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Stacked Dutch
design for a new Dutch embassy in Oslo

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Stacked Dutch
Design for a new Dutch embassy in Oslo

Patrick van Dodewaard - Structural Design elaboration
Stacked Dutch
Design for a new Dutch embassy in Oslo

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Stacked Dutch
Design for a new Dutch embassy in Oslo

Patrick van Dodewaard, 24 December 2015

Stacked Dutch makes a statement on how the Dutch embassy in Oslo can collaborate with commercial companies. A new Dutch embassy is designed in the centre of Oslo, Norway. By hosting Norwegian and Dutch start-ups the embassy becomes profitable and relevant by acting as a catalyst in-between both countries’ economies. The project becomes sustainable due to the main timber structure. Wood can be considered as a traditional material, but even more as an innovative material as the design proves. This duality adds to the ambitions of the embassy: innovating and preserving traditions.

The answer of sharing a secured institute with start-ups is found in the Dutch Identity. The Dutch Hofje (small courtyard with garden) form over 400 years a quiet, private and shared space for citizens. The properties of hofjes are used to form a contemporary office concept with small clusters of companies gathered around a shared central space. By scaling the principle of the urban street and hofjes down to building level, and by stacking them it is possible to create a public building with private areas without demanding visible security measures. With a maximum amount of 40 employees per hofje it is improving collaboration between companies and avoiding anonymity. The embassy-unit is a special volume in the centre of the building.

The structure is dominated by stacked timber trusses, a sustainable system creating strength, stability, and a certain flexibility on the floors. By placing the load-bearing system in the façades each office floor becomes very flexible. This makes the building more sustainable since the interior layer (also design in timber, since there is plenty of wood around in Norway) can be easily transformed every few years. At night the timber trusses show themselves through the translucent façade which gives the building an dynamic appearance. The connections of the timber truss connections are designed with steel tubes making an equilibrium in the nodes of the truss. A flexible connection suitable for all scenarios, hidden in the floor package. Only the steel bolts in the timber diagonals are visible, adding some structural beauty to the building.

The Structural Design of the building received special attention during the design process resulting in an integrated building design which cannot be seen as an architectonical project alone.

Stacked Dutch is a reinterpretation of the functioning of a Dutch embassy, with the Dutch Identity and sustainability as key elements for an innovative design, all together in a Norwegian context.
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1
Introduction
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Preface

The goal of the graduation studio No®Way is to design a sustainable Dutch embassy in Norway. The studio is set up by ir. Maarten Willems, together with dr.ir. Faas Moonen and dr.ir. Jos Bosman they form the evaluators for this studio.

The graduation project was divided in three parts. The first part, the M3 project, can be considered as the research part while the second part, M4, can be seen as the design part. The third part is the elaboration of the Structural Design calculations. The M3 research had a duration of four months and started in October 2014. The M4 project started in January 2015 with an excursion to Oslo and ended with an end colloquium in August 2015. Resulting in a nomination for the Dutch Archiprix 2016 competition. The Structural Design elaboration started in September 2015 till January 2016.

The studio gave me the opportunity to graduate a combined master, containing an architecture track and a structural design track. This means that the focus in the design is based on integrating structure and architecture. When graduating the architectural part, I finished the qualitative design phase of the structural design part. After this I quantified the structural design with the results stated in this thesis.

The thesis starts with an introduction in the design so it can be read without having read the Architectural thesis. Afterwards some elements of the design are checked to give a clear view on the final structure. Steel, concrete and timber calculations are done to test my structural design capacities. It will be no surprise that the pith of the matter will be in the timber elements.

It is important to notice that it was never the goal of the thesis to give a complete elaboration of the structural design. Only characteristic elements are pointed out and checked according to the Eurocode standards.
Introduction

No®Way: A sustainable Dutch embassy

The new designed embassy in Oslo is a study about integrating the Dutch Identity and sustainability into a building design. Where the embassy functions as a visible Dutch flagship in foreign countries the relation between the Dutch Identity and the design becomes clear. In former Dutch embassies architecture played a very dominant role to express the Dutch, in contemporary embassies the architecture is more modest. The representation of the Dutch is done with the interior of the building, which is a much cheaper tool. This is immediately the biggest problem/challenges of a contemporary embassy. They are (still too) expensive and not really relevant anymore due to the digital age. Therefore several ways to lower the costs are introduced, without losing the Dutch ‘quality’ image.

The new designed Dutch embassy in Oslo does not focus on the costs, but it focusses on the profit an embassy can make. Although an embassy is not allowed to make profit, it can be commercialized to balance the costs. The embassy is designed as a house for several small companies which can work together in a flexible building. In this way collaboration is stimulated and can stimulate the economic relationship between Norway and The Netherlands. The embassy supports small companies and can easily provide services to these companies. This Dutch habit of trading is stimulated and exploited in the building.

When creating a building for several companies security becomes a serious issue, especially when the embassy is involved in the building as well. The wish of the current embassy to be very open and to improve collaboration in contrast with the security which is needed to house several companies is solved by introducing small clusters. These clusters are inspired by Dutch hofjes, a semi-public urban typology with a very private feeling. The functioning and qualities of the Dutch hofje is described in the architectural thesis.

By looking at urban elements and translating them in the right way it becomes possible to play with themes such as privacy, openness, security, cooperation, visibility. But is important to notice that it is not the goal of this design to literally copy hofjes, streets and plazas, the design is about a translation of the classical hofjes concept towards a contemporary solution for privacy, openness, flexibility and cooperation. It is impossible to show the Norwegians that the embassy is build up by hofjes, since they do not know the typology and it is irrelevant in the first place. It is however a subtle tool to organize a multi company office building. The final result is pictured in Figure 1.1.
Figure 1.1
Design overview
Design overview

The design is a rather conceptual building and therefore the concept is explained in Figures 1.2 and 1.3. The hofjes used to form a multiple office building can be seen as the basis for the design.

In the Dutch embassy the hofjes are scaled down to building level and stacked upon each other to make a more compact volume. Other urban typologies are used to make the building clearer and easier to discuss, such as street and plaza. When introducing hofjes with companies there will be a routing system needed to enter all those office hofjes. This is designed as a street, a circling staircase atrium.

Where the hofjes are private and secured, the routing system has a public character. All employees can use the routing system to get to their office and it will also be used by clients of the embassy or citizens moving their way up towards the restaurant on the top floor. This public accessibility makes the Dutch embassy better known in Oslo, since it becomes more visible.

The street ends in the top of the building with the representative room. A space which is shared by the embassy with commercial parties. This room with great panoramic view will be one of the attractors of the building. The streets provide a visual relationship with the closed hofjes while a (translucent) façade is providing security, according to Ellin’s securing theories the hofje vs the street can be seen as a crusty space. This security measure is in fact not recognized as security (but as façade instead), and is therefore a very positive measure to use. These security issues form the basis of the concept, although it will be invisible when the building is rightly designed.

In the building three different privacy zones can be described, the public street, the semi-public hofjes and the private offices.

The hofjes are designed completely in wood. CLT panels combined with timber framing makes it possible to create a warm and cosy feeling with a sustainable layer. Most hofjes are one level in height.

**Figure 1.2**
Conventional embassy functions as an intermediate between Norwegian and Dutch companies.

**Figure 1.3**
Renewed embassy improves the catalyst function by housing companies, resulting in lower costs and better use.
Two concrete cores provide stability and function as a secondary and tertiary routing system.

Adding eight hofjes, the hofjes are partly self-supporting and partly supported by the cores. Twisting the volumes in a smart way makes the structural system easier and expresses individuality.

The embassy in the middle of the building is added. This embassy has a divergent appearance compared to the hofjes.

A routing is added to form the building. The spatial routing system holds the different hofjes together to form a complete composition.
corresponding to the low rise buildings which is common in hofjes, two hofjes do have an internal terrace, on top of offices, with a working area, this increases the diversity of the hofjes and creates a diverse atmosphere. Six hofjes and a representative room are designed.

The location and the desired dimensions of a hofje (not too big, not too small) resulted in two hofjes per floor. Each hofje is shifted horizontally and vertically, every hofje has therefore its own floor level, all to make the design more dynamic, visible in Figure 1.5. Similar to the original hofjes there is hardly any relation with other hofjes in the building, the relation between the employees of the hofjes can be strengthen on the lunch plaza at the top floor (Figure 1.6). When the hofjes would have a relation with each other it would have been too much interaction. The privacy would have been gone, including the social control and the possibility to work together on manageable scale.

The hofjes have a solid floor with flexible office units on top of it. Since office concepts last no longer than ten years they need to be flexible. A vertical distance between the hofjes is added to increase the individual appearance of each hofje. It corresponds to the ‘air as a roof’ element of the hofje. It also helps to control the daylight situation since larger and smaller openings can be created in this way. In the future it should be possible to create a local climate system for each hofje. Twisting and shifting the volumes also results in very useful terraces with great views. To access these terraces you have to cross a compete hofje, which makes it much more private and part of the hofje.
The embassy is a part of a much larger masterplan (Figure 1.8 and 1.9). This masterplan only mentioned briefly in this thesis since the thesis is focussed on the embassy alone. The architectural elaboration is stated in the architecture thesis, for now it is important to notice that the morphology research resulted in an embassy with a rectangle shape. Furthermore the structural grid for the complete project is derived from an efficient parking lot size, projected on the plot. In this way the embassy is depending on the masterplan.

The masterplan has a different structural plan compared to the embassy. Since the plan houses apartments, shops, public space and parking garage a more modest structure based of CLT slabs is designed. The concrete parking garage functions as a solid base for the apartment blocks. These blocks are completely designed in wood. By using CLT panels the building method is similar to a prefab concrete slab building method. The CLT however adds to the sustainability of the project.

Furthermore the volumes are based on the structural grid, resulting in a clear load distribution from roof to foundation without using cantilevers.
Introduction structural design

Overview

During the design phase of the project it was of great importance to integrate structural design and architecture. This integral approach should result in a relative simple elaboration of the structure. Since the first day of the project the architect and the structural engineer have cooperated (since it was one person, me) too develop a building in which the structure and architecture lean against each other.

This second part, the ‘calculation’ part is not mainly about improving the design, but about proving that the integral approach is good approach instead. The structural design is done already, only the verification is elaborated to prove that the design is realistic.

The design is literally a collection of stacked timber trusses (Figure 1.10). Each truss defines the façade of a hofje. By stacking the trusses the individualistic appearance of each hofje is strengthen, furthermore it creates a readable structural system. Most trusses are rotated a few degrees, besides creating a dynamic design this also allows the routing system to curve between the hofjes. This would not be possible with an orthogonal orientation of the trusses, a lack of supports will arise. All vertical loads of columns are directly transferred to diagonals in the timber trusses. Therefore all diagonals are designed under a different angle, this is very visible in Figure 1.10. In between the timber trusses large floor structures are used to create a span from truss to truss. By creating this large open floor the flexibility of the hofjes is guaranteed, this is a very important aspect of the sustainability part of the building.

Embassy

The structure of the embassy is different from the hofjes. The embassy is hanging at the steel trusses (Figure 1.11) in the roof with six tensile bars. The spaces around the embassy are not suited for large columns, therefore an indirect load distribution is chosen. Hanging it at the steel trusses makes the system clear, furthermore the symbolic difference of the embassy and hofjes is also visible in the structure. The embassy unit is not treated extensively in this thesis since the timber trusses and floor system of the hofjes are more interesting to calculate compared to the CLT slabs of the embassy.

Stability

The stability of the building is provided by the trusses and stiff floors, they distribute the loads to one of the two concrete cores which will distribute the loads to a pile foundation. The concrete cores work together since large steel trusses in the roof connect both cores.
Figure 1.10
Trusses with double chords
Figure 1.11
Simplified structural model scale 1:100. System of trusses and embassy is showed, structure of the street is left out to keep the model clear.
Figure 1.12
Technical section of the architectural design, strip of the south façade. Please notice that this is not the final design.
As mentioned earlier not every element is calculated in this project. The elements which are calculated are pictured in Figure 1.13 making the checks better understandable.

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<td>Glulam beam</td>
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<td>Concrete core A wall A</td>
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<td>7.2</td>
<td>Concrete core A wall B</td>
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<td>7.3</td>
<td>Concrete Retaining wall</td>
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<td>7.4</td>
<td>Concrete Basement floor</td>
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*Figure 1.13 Isometric drawing*
2
Structural Plans
Basement floor (Level -3000 mm)
  Level +1500
  Level +6000
  Level +10000
  Level +12000
  Level +17000
The basement is designed in concrete to form a stable and heavy ‘foot’ under the timber building (Figure 2.1). Furthermore it protects the Building Services, IT services and parking garage from explosions. The retaining walls and concrete floors are over-dimensioned due to moisture regulations in combination with crack control (Elaborated in chapter 7). With numbers some rooms are indicated.

The steel column in the Building services room distributes a load from the main staircase, this load is expected to be large and transferred directly to the concrete floor instead of the timber floors constructed on higher levels.

Figure 2.1
Structural floor plan
Level -3000 mm
Scale 1:200
Level +1500

Structural system on level +1500 and level +0 is elaborated (Figure 2.2). Load is transferred by timber sub-beams to main-beam to a truss and transferred to the basement walls or cores resulting in a load on the foundation. Furthermore the façade columns are pictured, transferring the wind load to the concrete basement.

The entrance of the building is a small cube with local stability measures, two steel crosses in combination with a solid roof plate provide stability in all directions.

Note: All structural elements are designed in timber unless mentioned different.
Level +6000

Structural system on level +6000 and level +5500 (Figure 2.3). The steel columns connecting the trusses to the trusses below are located. Truss 2.3 is penetrating the concrete core since this results in a beneficial mechanical scheme compared to a single cantilever truss. In the chosen method no difficult tensile connection has to be elaborated, however no design is made of the way the truss is crossing the concrete core. Therefore the lines are dotted.

Note: All structural elements are designed in timber unless mentioned different.

Figure 2.3
Structural floor plan
Level +6000 mm
Scale 1:200
The offices in Hofje B are build up by CLT walls. On top of these walls the opportunity of an extra floor is created. The simple floor structure of CLT panels on top of the CLT walls makes this possible. Stability is provided by connecting the rooms to a near truss. (Figure 2.4)

Furthermore steel tensile bars are introduced in the façade to reduce the desired beam height in the façade, this is important since no rejuvenation can be applied in the cantilever parts.

Note: All structural elements are designed in timber unless mentioned different.
The extra floor in hofje D is elaborated in a similar way compared to hofje E (Figure 2.5). Furthermore the Lunch Stand is hanged at the steel trusses in the roof. A steel structure is designed to support the floor. This structure is not elaborated further in this thesis, only an estimation of the final loads is given for calculating the steel trusses.

*Note: All structural elements are designed in timber unless mentioned different.*
The embassy floor is build up by a large CLT slab, combined with three beams to strengthen the floor, connected to six steel tensile columns. The columns are connected to the steel truss in the roof (Figure 2.6). The representation room and the facilitating hofje (toilets and restaurant) are designed similar to other hofjes.

*Note: All structural elements are designed in timber unless mentioned different.*
3

Loads
Wind loads

The wind load on different parts of the building is calculated after the NEN-EN 1991-1-4. Since the national annex of Norway was not available some Dutch parameters are used during the calculation of the wind load.

\[ F_w = c_v c_d c_i q_p(z_e) A_{ref} \]

Wind speed

\[ v_b = v_{b,0} \cdot c_{dir} \cdot c_{season} \]
\[ v_b = 29.5 \cdot 1.0 \cdot 1.0 = 29.5 \text{ m/s} \]

Oslo is located near the sea and therefore the Dutch value for \( V_{b,0} \) at wind area 1 is chosen. This is a conservative estimation.

Accumulation pressure (part 1)

According to the Dutch annex, the maximal accumulation pressure is given in chart NB table 5. For the embassy this in a high density area near the sea would result in a maximal accumulation pressure of:

\[ q_p(z_e) = q_p(25) = 1.16 \text{ kN/m}^2 \]

When this national annex value is ignored the accumulation pressure can be calculated as stated below:

\[ z_e = h = 25 \text{ m} \]
\[ z_{0,lv} = 1.0 \]
\[ z_{min} = 10 \]
\[ z_{max} = 200 \]

Terrain roughness:

\[ k_r = 0.19 \cdot \left( \frac{z_0}{z_{0,lv}} \right)^{0.07} = 0.19 \cdot \left( \frac{0.05}{0.05} \right)^{0.07} = 0.23 \]
\[ c_i(z) = k_r \cdot \ln \left( \frac{z}{z_0} \right) = 0.25 \cdot \ln \left( \frac{25}{1.0} \right) = 0.80 \]

Variation according the height:

\[ v_m(z) = c_i(z) \cdot c_d(z) \cdot v_b \]
\[ = 0.80 \cdot 1.0 \cdot 29.5 = 23.5 \text{ m/s} \]

Wind turbulence factor:

\[ \sigma_v = k_r \cdot v_b \cdot k_i = 0.23 \cdot 29.5 \cdot 1.0 = 6.79 \]
\[ l_v(z_e) = \frac{\sigma_v}{v_m(z_e)} = \frac{6.79}{23.5} = 0.29 \]

Resulting in an extreme accumulation pressure of:

\[ q_p(z_e) = (1+7 \cdot l_v(z_e)) \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z) = c_i(z_e) \cdot q_b \]
\[ q_p(z_e) = (1+7 \cdot 0.29) \cdot \frac{1}{2} \cdot 1.25 \cdot 23.5^2 = 1.045 \text{ kN/m}^2 \]
\[ \rho = 1.25 \text{ kg/m}^3 \]
\[ l_v(z_e) = 0.29 \]
\[ v_m = 23.5 \]

The calculated accumulation pressure is slightly smaller than the value stated in the Dutch annex, the calculated value is chosen.

Building factor \( c_c c_d \)

Since the building is higher than 15 meter and the height-depth ratio is not beneficial the value \( c_c c_d \) needs to be calculated according to the detailed method.

According to the Dutch Annex it is possible to split the values \( c_c \) and \( c_d \) since:

\[ h = 25 < 50 \text{m} \]
\[ \frac{h}{w} = \frac{25}{16} = 1.56 < 5 \]
\[ c_d = 1.0 \]

It is common to state \( c_c \) and therefore the \( c_c c_d \) will result in 1. This is however too conservative.
\[ h = 25 \text{ m} \\
z_s = 0.6 \cdot h = 0.6 \cdot 25 = 15 \text{ m} \]

Wind turbulence factor:

\[ L(z) = L_t \cdot \left( \frac{z}{z_t} \right)^\alpha = 300 \cdot \left( \frac{15}{200} \right)^{0.67} = 52.89 \]

\[ L_t = 300 \text{ m} \\
z = 15 \text{ m} \\
z_t = 200 \text{ m} \\
\alpha = 0.67 + 0.05 \ln(z_0) = 0.67 \]

\[ B^2 = \frac{1}{1 + 0.9 \left( \frac{b + h}{L(z)} \right)^{0.63}} = \frac{1}{1 + 0.9 \left( \frac{17.4 + 25}{52.89} \right)^{0.63}} = 0.55 \]

\[ b = 17.4 \text{ m} \\
h = 25 \text{ m} \\
L(z_s) = 52.89 \]

\[ I_v(z_s) = \frac{\sigma_v}{v_m(z)} = \frac{k_j}{c_s(z) \cdot \ln \left( \frac{z}{z_0} \right)} \]

\[ I_v(z_s) = \frac{1}{1.0 \cdot \ln \left( \frac{25}{10} \right)} = 0.31 \]

\[ c_s = \frac{1 + 7 \cdot I_v(z_s) \cdot \sqrt{B^2}}{1 + 7 \cdot I_v(z_s)} = \frac{1 + 7 \cdot 0.31 \cdot \sqrt{0.55}}{1 + 7 \cdot 0.31} = 0.82 \]

Resulting in a building factor of:

\[ c_s \cdot c_d = 0.82 \cdot 1.00 = 0.82 \]

Since the value is smaller than 0.85 it can not be used according to the Dutch Annex. 0.85 is used instead since it is more conservative.

**Compression coefficients**

\[ h = 25 \text{ m} \\
b = 17.4 \text{ m} \]

\[ b < h < 2b \]

The vertical façade of the building should be treated in two parts.

\[ upper = q_p(z) = q_p(h) \]

\[ lower = q_p(z) = q_p(b) \]

\[ h = \frac{25}{17.3} = 1.45 \]

\[ e = b \text{ or } e = 2h \]

\[ e = b = 58 \text{ m} \]

\[ e = 2h = 50 \text{ m} \]

\[ d = 17.4 \]

\[ e = 50 \]

\[ so: e > d \]

Resulting in the following façade coefficients:

\[ A \quad C_{pe,10} = -1.2 \quad C_{pe,1} = -1.4 \]

\[ B \quad C_{pe,10} = -0.8 \quad C_{pe,1} = -1.1 \]

\[ C \quad C_{pe,10} = -0.5 \quad C_{pe,1} = -0.5 \]

\[ D \quad C_{pe,10} = +0.8 \quad C_{pe,1} = +1.0 \]

\[ E \quad C_{pe,10} = -0.5 \quad C_{pe,1} = -0.5 \]

**Roof coefficients (for flat roof):**

\[ h_p = 0.30 \]

\[ h = 25 \]

\[ \frac{h_p}{h} = \frac{0.30}{25} = 0.012 < 0.025 \]

\[ F \quad C_{pe,10} = -1.6 \quad C_{pe,1} = -2.2 \]

\[ G \quad C_{pe,10} = -1.1 \quad C_{pe,1} = -1.8 \]

\[ H \quad C_{pe,10} = -0.7 \quad C_{pe,1} = -1.2 \]

\[ I \quad C_{pe,10} = +0.2 \quad C_{pe,1} = -0.2 \]
Accumulation pressure (part 2)

Since the building has two different accumulation pressure coefficients both have to be calculated, the method used is similar to accumulation pressure part 1.

\[ q_{p1} = q_p(b) \]
\[ q_{p2} = q_p(h) = 1,045 \text{ kN} / \text{m}^2 \]
\[ k_v = 0,23 \]
\[ \rho = 1.25 \text{ kg} / \text{m}^3 \]
\[ c_v(z) = k_v \cdot \ln\left(\frac{z}{z_0}\right) = 0,25 \cdot \ln\left(\frac{17,4}{10}\right) = 0,71 \]
\[ \nu_m(z) = c_v(z) \cdot c_s(z) \cdot \nu_b \]
\[ = 0,71 \cdot 1,0 \cdot 29,5 = 20,9 \text{ m/s} \]
\[ \sigma_v = k_v \cdot \nu_b \cdot k_s = 0,23 \cdot 29,5 \cdot 1,0 = 6,79 \]
\[ l_v(z_s) = \frac{\sigma_v}{\nu_m(z_s)} = \frac{6,79}{20,9} = 0,32 \]
\[ q_p(b) = (1 + 7 \cdot l_v(z_s)) \cdot \frac{1}{2} \cdot \rho \cdot \nu_m^2(z) \]
\[ q_{p1} = q_p(b) = (1 + 7 \cdot 0,32) \cdot \frac{1}{2} \cdot 1,25 \cdot 20,9^2 = 0,884 \text{ kN} / \text{m}^2 \]

Wind load

Now all coefficients are clear it is possible to calculate the wind load on all components of the building with the equation:

\[ F_w = c_s c_d \cdot c_i \cdot q_p(b) \cdot A_{ef} \]
\[ F_w = c_s c_d \cdot c_i \cdot q_p(h) \cdot A_{ef} \]

Since the volume of the building is complex the exact wind load calculation will be slightly different than to rectangle model used so far, however it is to be expected that the differences will be relatively small.

The wind loads acting on the hofjes are transferred mainly by the floors to the concrete cores which are assumed to be (almost) infinite stiff.

The wind load is calculated at two areas, one for the façade from level 0 to level +17 meter and one for level +17 to level +22 m. The schematization of the wind load is shown in figures 3.1 and 3.2. The calculation of the wind loads acting on the concrete cores is elaborated in the chapter 'Concrete cores'.

Lower part (zone 1)

Wind pressure coefficient (line load)

\[ F_{w1} = c_s c_d \cdot c_i \cdot q_p(b) \cdot A_{ef} \]
\[ q_{w1} = 0,884 \cdot 0,8 \cdot 0,85 = 0,6 \text{ kN} / \text{m}^2 \]

Wind suction coefficient (line load)

\[ F_{w1} = c_s c_d \cdot c_i \cdot q_p(b) \cdot A_{ef} \]
\[ q_{w1} = 0,884 \cdot -0,5 \cdot 0,85 = -0,38 \text{ kN} / \text{m}^2 \]

Upper part (zone 2)

Wind pressure coefficient (line load)

\[ F_{w1} = c_s c_d \cdot c_i \cdot q_p(b) \cdot A_{ef} \]
\[ q_{w1} = 0,884 \cdot 0,8 \cdot 1,045 = 0,71 \text{ kN} / \text{m}^2 \]

Wind suction coefficient (line load)

\[ F_{w1} = c_s c_d \cdot c_i \cdot q_p(b) \cdot A_{ef} \]
\[ q_{w1} = 0,884 \cdot -0,5 \cdot 1,045 = -0,44 \text{ kN} / \text{m}^2 \]
Figure 3.1
Zone 1 and 2 are pictured in the south façade. Wind pressure is acting on the façade when wind from south.

Figure 3.2
Zone 1 and 2 are pictured in the north façade. Wind suction is acting on the façade when wind from south.
The orange line in Figure 3.3 shows the façade with the governing wind-load direction. The south-west direction is used for checking structural elements on stability. This means wind pressure on the south side and wind suction on the north side.

When checking the complete building other wind direction need to be taken into account. Furthermore the dynamic wind loads caused by neighbouring buildings should be modelled. For this thesis only one simple wind direction suffices.
Snow load

The snow load is calculated after NEN-EN 1991-1-3.1.

\[ \text{\( s = \mu_1 \cdot C_a \cdot C_t \cdot s_k \)} \]

\[ \text{\( s = 0.8 \cdot 1.0 \cdot 1.0 \cdot 3.25 = 2.6 \text{ kN/m}^2 \)} \]

\[ \mu_1 = 0.8 \text{ roof angle between 0 and 30°} \]
\[ C_a = 1.0 \text{ normal environment} \]
\[ C_t = 1.0 \]

\[ s_k = 3.25 \text{ kN/m}^2 \]

The correct \( s_k \) factor for Norway is given by the Eurocode. A map illustrates that the snow load on the ground in Oslo is 3.25 kN/m\(^2\) (Figure 3.4). Since the altitude of Oslo is zero no further factors need to be taken into account.

Since the \( s_k \) factor in the Netherlands is no more than 0.7 kN/m\(^2\) instead of 3.25 kN/m\(^2\) it is to be expected that the roof structure will be thicker as proposed earlier since the Dutch rules of thumb do not suffices anymore.

Figure 3.4
Snow load on the ground in Norway
## Weight calculation

The permanent and variable loads acting in the building are estimated. All loads are assumed to spread under a certain surface. The façade factor for example has a low value, but it is spreading occurs the whole surface of the floor. This simplification is applied in order to keep the calculation clean. Since the building has several combined functions sometimes a conservative approach is chosen. The Lunch plaza for example is a transportation area, as well a catering area. This resulted in a conservative instantaneous factor of 1,0 instead of 0,25.

For an exact weight calculation a correct Revit model is required with all correct material properties. For this thesis a global weight calculation suffices. The loads are inspired by values of the Dutch *Tabellenboek*[^1] and NEN-EN 1991-1-1 art 6.3.1[^1].

All loads are multiplied by the surfaces retrieved from Revit. Afterwards the safety factors are introduced when using the loads in the ULS and SLS. For the weight calculation the roof is loaded instantaneous with a value of 0,7 which is assumed. Furthermore one floor is loaded extreme while all others are loaded with an instantaneous factor.

### Permanent loads

<table>
<thead>
<tr>
<th>Permanent loads</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof package including girders</td>
<td>2 kN/m²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof package including trusses</td>
<td>4 kN/m²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Façade</td>
<td>0,5 kN/m²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Floor street</td>
<td>1,5 kN/m²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Railings, lightning etc</td>
<td>0,7 kN/m²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total floor street load</td>
<td>2,7 kN/m²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Façade</td>
<td>1 kN/m²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Floor hofjes</td>
<td>1,2 kN/m²</td>
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<td></td>
</tr>
<tr>
<td>Ceiling, lightning etc</td>
<td>0,3 kN/m²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dividing walls</td>
<td>0,5 kN/m²</td>
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<tr>
<td>Total hofjes floor package</td>
<td>3 kN/m²</td>
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<tr>
<td>Floor basement</td>
<td>2,5 kN/m²</td>
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<tr>
<td>Retaining wall basement</td>
<td>2,5 kN/m²</td>
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<tr>
<td>Total basement floor including walls</td>
<td>5 kN/m²</td>
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<td>Building services</td>
<td>2 kN/m²</td>
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### Variable loads

<table>
<thead>
<tr>
<th>Variable loads</th>
<th></th>
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<tbody>
<tr>
<td>Snow</td>
<td>2,6 kN/m²</td>
</tr>
<tr>
<td>Office floor</td>
<td>2,5 kN/m²</td>
</tr>
<tr>
<td>Staircases</td>
<td>5 kN/m²</td>
</tr>
<tr>
<td>Representative room</td>
<td>5 kN/m²</td>
</tr>
<tr>
<td>Basement floor</td>
<td>3,5 kN/m²</td>
</tr>
<tr>
<td>Parking garage</td>
<td>2 kN/m²</td>
</tr>
<tr>
<td>Central routing</td>
<td>3 kN/m²</td>
</tr>
<tr>
<td>Lunch Plaza</td>
<td>5 kN/m²</td>
</tr>
</tbody>
</table>

\[ \psi \]
<table>
<thead>
<tr>
<th>Level</th>
<th>Room Description</th>
<th>Variable</th>
<th>$\psi$</th>
<th>Var-$\psi$</th>
<th>Permanent</th>
</tr>
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<tbody>
<tr>
<td><strong>Roof</strong></td>
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<td></td>
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<tr>
<td>Left of core A</td>
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<td>517</td>
<td>0,7</td>
<td>362</td>
<td>398 kN</td>
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<td>488</td>
<td>536 kN</td>
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<td>39 kN</td>
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<td>996</td>
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<td>2749</td>
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<td>1924</td>
<td>3209 kN</td>
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<td>Kitchen and Toilets</td>
<td>150 m$^2$</td>
<td>750</td>
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<td>750</td>
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<tr>
<td>Lunch plaza and stairs</td>
<td>189 m$^2$</td>
<td>945</td>
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<td>945</td>
<td>510 kN</td>
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<td>1000</td>
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<td>1000</td>
<td>600 kN</td>
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<td>60</td>
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<td>3125</td>
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<td>Embassy level 13</td>
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<td>1</td>
<td>120</td>
<td>65 kN</td>
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<td>Staircase hofje 12000</td>
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<td>1</td>
<td>100</td>
<td>54 kN</td>
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<td>Total</td>
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<td>220</td>
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<td>373</td>
<td>485 kN</td>
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<tr>
<td><strong>Level +12 m</strong></td>
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<td>Hofje 12</td>
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<td>Second floor hofje 10000</td>
<td>115 m$^2$</td>
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<td>0,5</td>
<td>144</td>
<td>345 kN</td>
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<tr>
<td>Balcony +12</td>
<td>36 m$^2$</td>
<td>94</td>
<td>0,7</td>
<td>66</td>
<td>108 kN</td>
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<td>Staircases</td>
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<td>200</td>
<td>1</td>
<td>200</td>
<td>108 kN</td>
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<td>Total</td>
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<td>1276</td>
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<td>757</td>
<td>1395 kN</td>
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<tr>
<td><strong>Level +10 m</strong></td>
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<td></td>
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<td></td>
</tr>
<tr>
<td>Hofje 10</td>
<td>200 m$^2$</td>
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<td>0,5</td>
<td>250</td>
<td>600 kN</td>
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<tr>
<td>Total</td>
<td></td>
<td>500</td>
<td></td>
<td>250</td>
<td>600 kN</td>
</tr>
<tr>
<td><strong>Level +8 m</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Balcony internal +8 m</td>
<td>211 m$^2$</td>
<td>528</td>
<td>0,5</td>
<td>264</td>
<td>633 kN</td>
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<tr>
<td>Balcony external</td>
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<td>66</td>
<td>1</td>
<td>68</td>
<td>78 kN</td>
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<tr>
<td>Exposition space</td>
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<td>0,25</td>
<td>133</td>
<td>286 kN</td>
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<td>35 m$^2$</td>
<td>175</td>
<td>1</td>
<td>175</td>
<td>95 kN</td>
</tr>
<tr>
<td>Total</td>
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<td>1300</td>
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<td>639</td>
<td>1092 kN</td>
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<td><strong>Level +6 m</strong></td>
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<tr>
<td>Hofje 6</td>
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<td>0,5</td>
<td>476</td>
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<td>425</td>
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<td>425</td>
<td>230 kN</td>
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<td>Total</td>
<td></td>
<td>2113</td>
<td></td>
<td>1269</td>
<td>2255 kN</td>
</tr>
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</table>

*Figure 3.5*  
Global loads and areas.
<table>
<thead>
<tr>
<th>Location</th>
<th>Area (m²)</th>
<th>Variable</th>
<th>Var.</th>
<th>Permanent (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Level 1.5 &amp; level 0</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hofje 1.5</td>
<td>537</td>
<td>1343</td>
<td>0.5</td>
<td>671</td>
</tr>
<tr>
<td>Main staircase</td>
<td>60</td>
<td>300</td>
<td>1</td>
<td>300</td>
</tr>
<tr>
<td>Landing</td>
<td>10</td>
<td>50</td>
<td>1</td>
<td>50</td>
</tr>
<tr>
<td>Entrance area</td>
<td>136</td>
<td>408</td>
<td>0.5</td>
<td>204</td>
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<td><strong>Total</strong></td>
<td></td>
<td>1643</td>
<td>971</td>
<td>1773</td>
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<tr>
<td>Level -3 m</td>
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<td>Building services/storage</td>
<td>548</td>
<td>1918</td>
<td>0.5</td>
<td>959</td>
</tr>
<tr>
<td>Parking garage</td>
<td>118</td>
<td>236</td>
<td>0.7</td>
<td>165</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td>2154</td>
<td>1124</td>
<td>4426</td>
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<tr>
<td><strong>Own weight Concrete cores</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Core A</td>
<td>144</td>
<td></td>
<td></td>
<td>3456</td>
</tr>
<tr>
<td>Core B</td>
<td>130</td>
<td></td>
<td></td>
<td>3110</td>
</tr>
<tr>
<td><strong>Total loads acting on the foundation</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Variable</td>
<td>Permanent</td>
<td>Total</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10461</td>
<td>26219</td>
<td>36680</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Safety factors 1.5 and 1.2</td>
<td>15691</td>
<td>31463</td>
<td>47155</td>
<td></td>
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<tr>
<td>Safety factor 1.35</td>
<td>14122</td>
<td>35396</td>
<td><strong>49518</strong></td>
<td></td>
</tr>
</tbody>
</table>

The governing weight calculation in the ULS is 49518 kN while for the SLS the total weight becomes 36680 kN. These values are important when calculating the two concrete cores.

Furthermore the values are used to make proper calculations. For example the timber trusses with connections, the point loads acting on the elements are estimated with help of the global weight calculations.
4

Roof structure
Structural roof plan
Load calculation lunch stand
Load calculation embassy
Girder
Gluelaminated beams
Steel Truss DD
Truss connection DD
Steel truss CC
Steel truss B
Modified roof section
The roof is divided in different zones. Left of core A (1 in Figure 4.1) there is a ‘floating’ timber element, supported by four steel columns. Therefore the roof is spanning in 4 directions in that area. On top of the steel trusses the roof is locally lifted since the trusses need space (zone 2). North and south of to the steel trusses (part 3 and 4) gluelaminated beams distribute the roof load to the steel trusses. Zone 5, east of core B is similar to the hofjes floor system with sub-beams and main-beams supported by timber trusses.

Note: All structural elements are designed in timber unless mentioned different.

1. Floating roof west of Core A
2. Lifted roof on top of steel trusses
3. North roof part
4. South roof part
5. East roof part (right of core B)

Figure 4.1
Structural roof plan
Level +21000 mm
Scale 1:200
Lunch stand load calculation

The lunch stand on level 13 to 16 m is schematized and the reaction forces of the tensile columns are calculated. The lunch stand is build up by CLT panels supported by a steel structure, this structure is not elaborated since it is not a visible key element of the design, the reaction forces however are important to get a more precise image of the forces acting on the steel trusses in the roof. The loads acting on the stand are schematized in two beams with supports acting as tensile bars. Beam 1 (Figure 4.2) is located at grid line 6 while the right bar is located 4,1 meter right to grid line 6. Both schemes intersect the tensile bars.

**Variable load**

\[ P_{rep} = 5,0 \text{ kN} / \text{m}^2 \]

\[ \psi = 1 \]

\[ P_{rep} \cdot \psi = 5,0 \text{ kN} / \text{m}^2 \]

**Permanent load**

<table>
<thead>
<tr>
<th>Component</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor</td>
<td>CLT</td>
<td>450 kg/m²</td>
</tr>
<tr>
<td>h</td>
<td></td>
<td>0,24 m</td>
</tr>
<tr>
<td>( Q_{floor} )</td>
<td></td>
<td>1,08 kN/m²</td>
</tr>
<tr>
<td>Ceiling</td>
<td></td>
<td>0,2 kN/m²</td>
</tr>
<tr>
<td>Finish</td>
<td></td>
<td>0,3 kN/m²</td>
</tr>
<tr>
<td>Steel structure</td>
<td></td>
<td>1,5 kN/m¹</td>
</tr>
</tbody>
</table>

**Scheme 1**

Since the shape of the floor is not rectangular the q load acting on the fictional beam varies in the right scheme (not pictured).

\[ w = 3,85 m \]

\[ q_{var,a} = 19,25 \text{ kN} / \text{m}^1 \]

\[ q_{per,a} = 7,66 \text{ kN} / \text{m}^1 \]

\[ q_{d1} = 1,5 \cdot q_{var,a} + 1,2 \cdot q_{per,a} = 38,07 \text{ kN} / \text{m}^1 \]

**Scheme 2**

\[ w = 2,65 m \]

\[ q_{var,a} = 13,25 \text{ kN} / \text{m}^1 \]

\[ q_{per,a} = 5,8 \text{ kN} / \text{m}^1 \]

\[ q_{d1} = 1,5 \cdot q_{var,a} + 1,2 \cdot q_{per,a} = 26,8 \text{ kN} / \text{m}^1 \]

\[ w = 3,35 m \]

\[ q_{var,b} = 16,75 \text{ kN} / \text{m}^1 \]

\[ q_{per,b} = 6,87 \text{ kN} / \text{m}^1 \]

\[ q_{d1} = 1,5 \cdot q_{var,a} + 1,2 \cdot q_{per,a} = 33,37 \text{ kN} / \text{m}^1 \]
Reaction forces tensile columns

B5A: 179 kN
B5B: 130 kN
DD5A: 30 kN (pressure)
DD5B: 22 kN (pressure)

Figure 4.2
Schematic of the lunch stand left bar with the four steel tensile columns.
Embassy load calculation

The loads of the embassy result in six tensile loads in bars connected to the steel trusses. In order to calculate the steel trusses the reaction forces are required. Therefore the embassy is (simply) schematized similar as the lunch stand with two beams. The reaction forces of the supports are translated to point loads on the steel truss.

The embassy houses two types of user functions. The first one, office and the second one, lobby. Both functions have different variable loads and different instantaneous factors resulting in an equal load:

\[
q_{\text{var, office}} = 2.5 \text{ kN/m}^2 \\
\psi_{0,\text{extreme}} = 1.0 \\
q_{\text{var, lobby}} = 5 \text{ kN/m}^2 \\
\psi_{0} = 0.5 \\
q_{\text{var}} = 2.5 \text{ kN/m}^2
\]

Including the safety factor of 1.5 the \(q_{\text{var}}\) becomes:

\[
q_{\text{var}} = 3.75 \text{ kN/m}^2
\]

Permanent loads acting on the embassy:

Note: Non load bearing timber frame walls are ignored since their weight is small and at that location no variable load can occur.

<table>
<thead>
<tr>
<th></th>
<th>level 13</th>
<th>level 17</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor</td>
<td>0.6 kN/m^2</td>
<td>0.6 kN/m^2</td>
</tr>
<tr>
<td>Ceiling</td>
<td>0.2 kN/m^2</td>
<td>0.2 kN/m^2</td>
</tr>
<tr>
<td>Including safety factor of 1.2</td>
<td>0.96 kN/m^2</td>
<td>0.96 kN/m^2</td>
</tr>
</tbody>
</table>

Extra loads caused by the own weight of the façades:

Façade load: 1.3 kN/m^1

(Timber frame structure with windows and aluminium cladding) Including safety factor of 1.2 results in a permanent load of the façade of 1.56 kN/m^1

South façade (point load in scheme): 7.80 kN
North façade (point load in scheme): 6.24 kN

The loads calculated are transformed from a surface to a line load according to Figure 4.3, the resulting loads can be seen in Figure 4.4, 4.5 and 4.6. The \(q_{\text{tot}}\) is applied in two MatrixFrame models of the embassy.

Calculated with MatrixFrame.

Scheme left:

\[
K2 = R_b = 460 \text{ kN} \\
K3 = R_{cc} = 67 \text{ kN} \\
K4 = R_{dd} = 540 \text{ kN}
\]

Scheme right:

\[
K2 = R_b = 670 \text{ kN} \\
K3 = R_{cc} = 25 \text{ kN} \\
K4 = R_{dd} = 411 \text{ kN}
\]

Figure 4.3

Two fictional beams (1 left and 2 right) and fictional locations of the tensile elements.
<table>
<thead>
<tr>
<th>Node</th>
<th>Length</th>
<th>Fictional width</th>
<th>Level 13 (m)</th>
<th>Fictional width</th>
<th>Level 17 (m)</th>
<th>$q_{\text{tot}}$</th>
<th>$q_{\text{var}}$</th>
<th>$q_{\text{per}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scheme left</td>
<td>K1</td>
<td>4,1</td>
<td>4</td>
<td>2,5</td>
<td>24,38</td>
<td>15,6</td>
<td>39,98</td>
<td>kN/m$^1$</td>
</tr>
<tr>
<td>K2</td>
<td>5,5</td>
<td>5</td>
<td>5</td>
<td>37,50</td>
<td>22,32</td>
<td>59,82</td>
<td>kN/m$^1$</td>
<td></td>
</tr>
<tr>
<td>K3</td>
<td>4,1</td>
<td>5</td>
<td>5</td>
<td>37,50</td>
<td>22,32</td>
<td>59,82</td>
<td>kN/m$^1$</td>
<td></td>
</tr>
<tr>
<td>K4</td>
<td>4,4</td>
<td>7</td>
<td>5</td>
<td>45,00</td>
<td>26,16</td>
<td>71,16</td>
<td>kN/m$^1$</td>
<td></td>
</tr>
<tr>
<td>K5</td>
<td>-</td>
<td>-</td>
<td>7</td>
<td>26,25</td>
<td>16,56</td>
<td>42,81</td>
<td>kN/m$^1$</td>
<td></td>
</tr>
<tr>
<td>Scheme right</td>
<td>K1</td>
<td>4,7</td>
<td>7</td>
<td>6,5</td>
<td>50,61</td>
<td>29,04</td>
<td>79,67</td>
<td>kN/m$^1$</td>
</tr>
<tr>
<td>K2</td>
<td>5,8</td>
<td>5</td>
<td>5</td>
<td>37,50</td>
<td>22,32</td>
<td>59,82</td>
<td>kN/m$^1$</td>
<td></td>
</tr>
<tr>
<td>K3</td>
<td>4,3</td>
<td>5</td>
<td>5</td>
<td>37,50</td>
<td>22,32</td>
<td>59,82</td>
<td>kN/m$^1$</td>
<td></td>
</tr>
<tr>
<td>K4</td>
<td>4,6</td>
<td>3,5</td>
<td>5</td>
<td>31,90</td>
<td>19,44</td>
<td>51,32</td>
<td>kN/m$^1$</td>
<td></td>
</tr>
<tr>
<td>K5</td>
<td>-</td>
<td>-</td>
<td>3,5</td>
<td>13,13</td>
<td>9,84</td>
<td>22,97</td>
<td>kN/m$^1$</td>
<td></td>
</tr>
</tbody>
</table>

**Figure 4.4**
Total loads translated from slab to line-load.

**Figure 4.5**
Scheme left (1) with applied loads.

**Figure 4.6**
Scheme right (2) with applied loads.
Girders

The girders supporting the roof are designed in sawn timber. The dimensions of the timber girders are designed according to NEN-EN 1995-1-1.

Core-to-core distance: 650 mm
Girder length: 5200 mm

**Permanent loads**

- EPDM layer, insulation and foils: 0.5 kN/m²
- Own weight timber girders: 0.3 kN/m³
- \( P_k \): 0.625 kN/m

**Variable loads**

\[ Q_{k,i} = 0.65 \cdot 1.0 = 0.65 \text{ kN} / \text{m}² \]

\[ s = (0.8 \cdot 1.0 \cdot 1.0 \cdot 3.25) \cdot 0.65 = 1.69 \text{ kN} / \text{m}² \]

Snow load is governing by far.

\[ q_d = 1.2 \cdot P_k + 1.5 \cdot Q_{snow} = \]

\[ q_d = 1.2 \cdot 0.63 + 1.5 \cdot 1.69 = 3.2 \text{ kN} / \text{m}³ \]

\[ q_{rep} = 1.0 \cdot P_k + 1.0 \cdot Q_{snow} = \]

\[ q_{rep} = 1.0 \cdot 0.63 + 1.0 \cdot 1.69 = 2.3 \text{ kN} / \text{m}³ \]

\[ M_{Ed} = 1/8 \cdot ql² \]

\[ M_{Ed} = 1/8 \cdot 3.2 \cdot 5.2² = 11.10 \text{ kNm} \]

\[ V_{Ed} = 1/2 \cdot ql \]

\[ V_{Ed} = 1/2 \cdot 3.2 \cdot 5.2 = 8.5 \text{ kN} \]

Bending strength sawn timber:

\[ f_{m;0;ud} = k_{mod} \cdot f_{m;0;rep} / \gamma_m \]

\[ f_{m;0;ud} = 0.8 \cdot 18 / 1.3 = 11.08 \text{ N} / \text{mm}² \]

With:

- \( k_{mod} = 0.8 \) average load duration
- \( f_{m;0;rep} = 18 \) C18
- \( \gamma_m = 1.3 \) NEN-EN 1995-1-1, art 2.4.1 (sawn timber)

\[ W_{req} = \frac{M_{Ed}}{f_{m;0;ud}} = \frac{1/6 \cdot bh²}{1/2 \cdot ql} \]

\[ W_{req} = \frac{11.10 \cdot 10^6}{11.08} = 10.02 \cdot 10^6 \text{ mm}³ \]

The largest common sawn timber dimensions, according to ‘houtwijzer’ 4.1 of Centrum Hout, has dimensions of 100x200 mm resulting in a:

\[ W = 1/6 \cdot bh² = 1/6 \cdot 100 \cdot 200² = 10.41 \cdot 10^6 \text{ mm}³ \]

This suffices \( W_{req} \) since:

\[ \frac{W_{req}}{W} = \frac{10.02 \cdot 10^6}{10.41 \cdot 10^6} = 0.96 < 1.0 \]

A check regarding deflection is done:

\[ u_{max} = \frac{l}{250} = \frac{5200}{250} = 20.8 \text{ mm} \]

\[ u_{def} = \frac{5}{384} \frac{ql^4}{EI} = \frac{5}{384} \frac{2.3 \cdot 5200^4}{9000 \cdot 1.30 \cdot 10^8} = 18.8 \text{ mm} \]

With:

- \( q_{rep} = 2.3 \text{ kN} / \text{m}³ \)
- \( l = 5200 \text{ mm} \)
- \( E = 9000 \)
- \( l = 1/12 \cdot bh³ = 1.30 \cdot 10^8 \)

The girder does suffices the maximum deflection, creep and a reduced E modulus are not taken into account and the deflection will be larger. The dimensions of the girder are however not increased since the deflection is not a problem. The roof structure may deflect a slightly more since the user will not experience this deflection, although the danger of cracks may occur.

Even more the total roof structure will be stiffer as calculated since the effect of the plywood panels connected to the girders is not taken into account. When this would have been taken into account the structural height and the moment of inertia will increase.
Glulam beam

The gluelaminated timber beams perpendicular to the steel trusses are checked in a similar way as the timber girders. These beams are connected to the steel trusses and have a maximal cantilever 4475 mm south of the truss. It is assumed that all snow load is taken by the girders, therefore the reaction forces of the girders create point loads on the glulam beams. Together with the own weight of the beam this makes the total load.

\[
L = 17560 \text{ mm} \\
V_{Ed,girder} = 8.5 \text{ kN}
\]

Frequency: point load acting every 650 mm

\[
q_{beam} = 0.66 \text{ kN/m} \\
q_d = 1.2 \cdot q_{beam} = 1.2 \cdot 0.66 = 0.79 \text{ kN/m}
\]

Point loads and q load are applied on the beam and the maximum bending moment is calculated with MatrixFrame (Figure 4.7)

\[
M_{Ed} = 140 \text{ kNm} \\
V_{Ed} = 63 \text{ kN}
\]

\[
f_{m:0:d} = k_{mod} \cdot f_{m:0:rep} / \gamma_m \\
f_{m:0:d} = 0.8 \cdot 26.5 / 125 = 17.02 \text{ N/mm}^2
\]

With:

\[
k_{mod} = 0.8 \quad \text{average load duration} \\
f_{m:0:rep} = 26.5 \quad \text{C28} \\
\gamma_m = 1.25 \quad \text{NEN-EN 1995-1-1, art 2.4.1 (laminated)}
\]

\[
W_{req} = \frac{M_{Ed}}{f_{m:0:d}} = 1/6 \cdot bh^2 \\
W_{req} = 140 \cdot 10^6 / 17.02 = 82.2 \cdot 10^5 \text{mm}^3
\]

The chosen beam profile suffices \( W_{req} \)

\[
W = 1/6 \cdot bh^2 = 1/6 \cdot 160 \cdot 600^2 = 96.00 \cdot 10^5 \text{mm}^3
\]

\[
\frac{W_{req}}{W} = \frac{82.2 \cdot 10^5}{96.00 \cdot 10^5} = 0.86 < 1.0
\]

Unity checks regarding the section 160x600 mm are calculated.

**Compression parallel to the grain ULS**

Since the beam is not loaded in compression (in the simplified model) the stress will be zero.

\[
\sigma_{c:0:d} = 0 \text{ N/mm}^2
\]

**Compression perpendicular to the grain ULS**

\[
\sigma_{c:90:d} = \frac{F_{c:90:d}}{A} = \frac{62.6 \cdot 10^3}{160 \cdot 600} = 0.65 \text{ N/mm}^2
\]

\[
f_{c:90:d} = k_{mod} \cdot \frac{f_{c:90:rep}}{\gamma_M} = 0.8 \cdot \frac{3.0}{125} = 2.16 \text{ N/mm}^2
\]

\[
\text{With:} \quad k_{mod} = 0.8 \quad \text{average load duration} \\
f_{c:90:rep} = 3.0 \quad \text{C28} \\
\gamma_m = 1.25 \quad \text{NEN-EN 1995-1-1, art 2.4.1 (glulam)}
\]

Results in the following check:

\[
\frac{c_{c:90:d}}{k_{90} \cdot f_{c:90:d}} \leq 10 \\
0.65 / 10 \cdot 2.16 = 0.30 < 1.0
\]

Beam satisfies for compression perpendicular to the grain.

Chosen beam profile, according to a lamina height of 40 mm:

b = 160 mm 

h = 600 mm
Bending moment ULS

Bending moment acting on the beam:

\[
\sigma_{m,0:d} = \frac{M}{W_y} = \frac{140 \cdot 10^6}{\frac{1}{6}bh^2} = \frac{140 \cdot 10^6}{\frac{1}{6} \cdot 160 \cdot 600^2} = 14,58 \text{ N/mm}^2
\]

Bending moment capacity:

\[
f_{m,0:d} = k_{mod} \cdot \frac{f_{m0:rep}}{\gamma_M} = 0,8 \cdot \frac{28}{1,25} = 17,92 \text{ N/mm}^2
\]

With:

- \(k_{mod} = 0,8\) average load duration
- \(f_{m0:rep} = 28\) C28
- \(\gamma_m = 1,25\) NEN-EN 1995-1-1, art 2.4.1 (glulam)

Check:

\[
\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1
\]

\[
k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1
\]

Since \(\sigma_{m,z,d} = \sigma_{m90,d} = 0\) and \(k_m = 0,7\) makes the first equation governing.

\[
\frac{\sigma_{m,y,d}}{f_{m,y,d}} + 0,7 \cdot 0 = 0,81 < 1
\]

14,58 + 0,7 · 0 = 0,81 < 1

Beam suffices for the bending moment criteria.

Shear stress check ULS

Shear stress on the beam section:

\[
\tau = \frac{F_{c390,d}}{A} = \frac{62,6 \cdot 10^3}{160 \cdot 600} = 0,65 \text{ N/mm}^2
\]

Ultimate shear force the beam section can transfer:

\[
f_{v,d} = k_{m,0:d} \frac{f_{v,0:rep}}{\gamma_M} = 0,8 \cdot \frac{3,2}{1,25} = 2,05 \text{ N/mm}^2
\]

Check:

\[
\frac{\tau_d}{f_{v,d}} \leq 1
\]

\[
0,65 \leq 2,05 = 0,31 \text{ N/mm}^2
\]

Beam suffices with ease.

Deflection SLS

The maximal deflection of the outer node in the serviceability state is 28 mm, calculated in MatrixFrame (Figure 4.7). Again creep is not included, but it is to be expected that the total deflection will be smaller when combining the girders, beams and multiplex plate into one stiffer roof package.

Check:

Distance from outer node to support: 3900 mm

\[
u_{\text{max}} = \frac{l}{250} = \frac{2 \cdot 3900}{250} = 31,2 \text{ mm}
\]

So the deflection (probably) satisfies.

Reaction forces supports

The reaction forces at the supports are important when calculating the steel trusses. These reaction forces are:

- B truss: 104 kN
- CC truss: 90,7 kN (tensile force)
- DD truss: 118 kN
Figure 4.7
Reaction forces at supports
Bending moment scheme
Deflection mode
Steel truss DD (DD3-DD9)

**Loads acting on truss DD**

An overview of the different loads acting on truss DD is given. (Figure 4.8)

1. **Roof**

Similar to the girder calculations before, the girders of the roof topping is calculated. This results in a line load acting on the truss. Since MatrixFrame does not allow line loads on trusses, all forces are converted to point loads on the nodes.

- Span roof CC-DD: 3.8 m
- C.t.c distance girders: 0.65 m
- EPDM, insulation, foils: 0.5 kN/m²
- Building services: 0.5 kN/m²
- Own weight girder: 0.3 kN/m¹
- \( P_k = 0.95 \text{ kN/m} \)

Snow load:
- \( Q_k = 2.6 \text{ kN/m}^2 \)
- \( q_d = 1.2 \cdot P_k + 1.5 \cdot Q_k \)
- \( q_s = 1.2 \cdot 0.95 + 1.5 \cdot 1.65 = 3.68 \text{ kN/m} \)

\( M_{Ed} = 1/8 \cdot q l^2 \)
\( M_{Ed} = 1/8 \cdot 3.68 \cdot 3.8^2 = 6.63 \text{ kNm} \)
\( V_{Ed} = 1/2 \cdot q l \)
\( V_{Ed} = 1/2 \cdot 3.68 \cdot 3.8 = 6.99 \text{ kN} \)

Now the point loads on the truss are known, since every 2.6 meter counts 4 girders the point loads on the knots are:

\( F_{root} = 6.69 \cdot 4 = 28 \text{ kN} \)

2. **Own weight truss**

Steel structure: 5 kN/m¹ (estimation)
Translated to point load on knots:
- \( q_d = 1.2 \cdot P_k \)
- \( q_s = 1.2 \cdot 5 = 6 \text{ kN/m} \)
- \( F_{own\text{-weight}} = 6 \cdot 2.6 = 15.6 \text{ kN} \)

3. **Timber glulam beams**

The timber beams connected with the steel trusses are calculated before. The resulted point loads acting every 5.2 meter on truss DD are: 118 kN

4. **Embassy point load**

Similar to the timber glulam beams reaction forces of the tensile columns of the embassy can be interpreted as point loads on the truss.

Therefore: \( K_6 = 540 \text{ kN} \)
\( K_9 = 411 \text{ kN} \)

Both loads are acting at an angle of 10.5°.

5. **Lunch Plaza point load**

The reaction forces of the Lunch stand are described earlier. The forces acting on the truss are (according to the MatrixFrame model) very small and compressional forces. This will not always be the case, since the platform will never be loaded everywhere at the same time (as calculated). Therefore the largest tensile force (acting on Truss B) is also used for calculating truss DD.

Therefore: \( K_5 = 178 \text{ kN} \)
\( K_6 = 130 \text{ kN} \)
Model of Truss DD

Length: 31.2 m
Height: 2.6 m
Steel: S355

The height of truss DD in the original design was 1.6 meter. It was however clear from the start of the calculation that this was not a realistic system height.

The truss is designed in a way it transfers loads mainly by tensional forces which is very common to do when designing in steel. Furthermore the steel class is relative high (with S355) since large forces are to be expected.

Result MatrixFrame

\[ N_{\text{max}} \text{ upper bar} = 3860 \text{ kN} \ (S32) \]
\[ N_{\text{max}} \text{ lower bar} = 3790 \text{ kN} \ (S36) \]
\[ N_{\text{max}} \text{ diagonal} = 1700 \text{ kN} \ (S18) \]

\[ R_{k_1} = 1225 \text{ kN} \]
\[ R_{k_2} = 1150 \text{ kN} \]

Figure 4.8
Overview of the MatrixFrame model used for calculating truss DD. The irregular numbering of nodes is the result of the modelling order in MatrixFrame. Also an overview of the applied loads is given.
**Upper bar compression strength**

The upper bar transfers compressional forces. Therefore buckling probably will be the governing factor. The load calculated with:

\[
N_{\text{max upper bar}} = N_{\text{ed}} = 3860 \text{ kN} \quad (S32)
\]

Steel = S355

System length = 31200 mm

Furthermore:

\[
\gamma_{m0} = 1,00 \\
\gamma_{m1} = 1,00 \\
\gamma_{m2} = 1,25
\]

A first approximation of the upper beam is done with a strength calculation. In this calculation the yield stress is decreased with 50% to reserve some room for the buckling problem.

\[
A_{\text{needed}} = \frac{N_{\text{ed}}}{0.5 \cdot f_{yd}}
\]

\[
A_{\text{needed}} = \frac{3860 \cdot 10^3}{0.5 \cdot 355} = 21746 \text{ mm}^2
\]

Chosen profile:

CFCHS 355,6 x 20 mm

\[
A = 21080 \text{ mm}^2 \\
I_{y} = 29792 \cdot 10^4 \text{mm}^4
\]

\[
\frac{N_{\text{Ed}}}{N_{\text{c, Rd}}} \leq 1,0
\]

\[
N_{\text{c, Rd}} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{21080 \cdot 355}{100} = 7483 \cdot 10^3 \text{ N}
\]

\[
N_{\text{Ed}} = \frac{3860 \cdot 10^3}{7483 \cdot 10^3} = 0.52 \leq 1.0
\]

The chosen profile fulfills the compression strength requirement with ease.

**Upper bar buckling**

Buckling can occur since the upper beam is loaded in compression.

Trusses can buckle as a whole when not connected, however not in this case since the three trusses are connected to each other every 10,4 meters. This connection is assumed to be stiff enough to carry the buckling problem. In this case the buckling length of the beam out of plane becomes 10,4 meters while the in-plane buckling length is equal to the beam-element length of 2,6 meters.

Check:

\[
N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}
\]

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

\[
\chi \leq 1,0
\]

\[
\Phi = 0.5 \cdot [1 + \alpha (\bar{\lambda} - 0.2) + \bar{\lambda}^2]
\]

\[
\bar{\lambda} = \frac{\sqrt{A \cdot f_y}}{N_{cr}}
\]

With alpha = 0,49 for cold formed profiles

\[
N_{cr} = \frac{\pi^2 \cdot E I}{l_{buc}^2} = \frac{\pi^2 \cdot 2,1 \cdot 10^5 \cdot 2,9 \cdot 10^8}{10400^2} = 5557 \text{ kN}
\]

\[
\bar{\lambda} = \frac{\sqrt{A \cdot f_y}}{N_{cr}} = \frac{21080 \cdot 355}{5557 \cdot 10^3} = 1.16
\]

\[
\Phi = 0.5 \cdot [1 + \alpha (\bar{\lambda} - 0.2) + \bar{\lambda}^2] = 1.41
\]

\[
\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{1.41 + \sqrt{1.41^2 - 1.16^2}} = 0.45
\]

\[
N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0.45 \cdot 21080 \cdot 355}{100} = 3367 \text{ kN}
\]

\[
N_{\text{Ed}} = \frac{3357}{3367} = 0.99 < 1.0
\]
The found profile fulfils the buckling test of the upper bar. However a new, rectangular profile is chosen. This can make the connections more economical, since the trusses are ‘hidden’ in the roof package no aesthetic arguments are leading.

HFRHS_I 400 x 400 x 16 mm  
\( h_i = 400 \text{ mm} \)  
\( A = 24300 \text{ mm}^2 \)  
\( l_y = 59336 \cdot 10^4 \text{ mm}^4 \)

Check on compression load

\[
\frac{N_{Ed}}{N_{c,Rd}} = 1,0  \\
N_{c,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{23400 \cdot 355}{100} = 8307 \cdot 10^3 \text{ N}  \\
N_{Ed} = 3860 \cdot 10^3  \\
N_{c,Rd} = 8307 \cdot 10^3  \\
\frac{N_{Ed}}{N_{c,Rd}} = 0.47 \leq 1.0
\]

As expected the profile fulfils with ease

Check on out-of-field buckling:

\[
N_{cr} = \frac{\pi^2 \cdot EI}{l_{buc}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 5.9 \cdot 10^8}{10400^2} = 11370 \text{ kN}  \\
\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \sqrt{\frac{23400 \cdot 355}{11370 \cdot 10^3}} = 0.87
\]

Check regarding the relative slenderness:

\[
\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \frac{l_{buc}}{i \cdot \lambda_1}  \\
i = 156,3 \text{ mm}  \\
\varepsilon = \frac{l_{buc}}{f_y} = \frac{235}{355} = 0.66
\]

\[
\lambda_1 = \pi \cdot \frac{\varepsilon}{f_y} = 93.9 \cdot 6.6 = 76.4  \\
\bar{\lambda} = \frac{l_{buc}}{i \cdot \lambda_1} = \frac{10400}{156,3 \cdot 76.4} = 0.87
\]

Note: Hot formed profile with alpha = 0.21

\[
\bar{\lambda} = 0.87  \\
\Phi = 0.5 \cdot \left[1 + \alpha \left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right]  \\
\Phi = 0.5 \cdot \left[1 + 0.21 \cdot (0.87 - 0.2) + 0.87^2\right] = 0.95  \\
\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{0.95 + \sqrt{0.95^2 - 0.87^2}} = 0.75  \\
N_{b,Rd} = \chi \cdot A \cdot f_y = \frac{0.75 \cdot 23400 \cdot 355}{100} = 6253.2 \text{ kN}  \\
N_{Ed} = 3860  \\
N_{b,Rd} = 6253,  \\
\frac{N_{Ed}}{N_{b,Rd}} = 0.62 < 1.0
\]

With the new profile the buckling problem is still not occurring.

**Lower beam axial tensile check**

The maximum tensile force found in the lower beam is 3300 kN. Since steel is very well capable of taking tensile forces this beam will have significantly smaller dimensions compared to the upper bar.

Chosen profile:  
HFRHS 300 x 300 x 12 mm  
\( h_i = 300 \text{ mm} \)  
\( A = 13660 \text{ mm}^2 \)

Check:

\[
\frac{N_{Ed}}{N_{t,Rd}} = 1,0  \\
N_{t,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{13660 \cdot 355}{100} = 4849 \cdot 10^3 \text{ N}  \\
N_{Ed} = 3790 \cdot 10^3  \\
N_{c,Rd} = 4849 \cdot 10^3  \\
\frac{N_{Ed}}{N_{c,Rd}} = 0.78 < 1.0
\]

The tensile bar satisfies.
**Vertical beam check**

Governing load in a vertical member, loaded under compression is beam S2 with \( N_{ed} = 1225 \, \text{kN} \).

Chosen profile (identical as lower bar):

HFRHS 300 x 300 x 12

\( h_t = 300 \, \text{mm} \)

\( A = 13660 \, \text{mm}^2 \)

Check:

\[
\frac{N_{Ed}}{N_{t,Rd}} \leq 1.0
\]

\[
N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{13660 \cdot 355}{100} = 4849 \cdot 10^3 \, \text{N}
\]

\[
N_{Ed} = \frac{1225 \cdot 10^3}{4849 \cdot 10^3} = 0.25 < 1.0
\]

The check satisfies.

The compressional resistance of the chosen profile is no problem, however buckling can occur in this vertical member. Therefore a buckling check is done.

\[
N_{cr} = \frac{\pi^2 \cdot EI}{l_{buc}^2} = \frac{\pi^2 \cdot 2 \cdot 1 \cdot 10^5 \cdot 1.88 \cdot 10^8}{2600^2} = 57641 \, \text{kN}
\]

\[
\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \sqrt{\frac{13660 \cdot 355}{57641 \cdot 10^3}} = 0.29
\]

\[
\bar{\lambda} = 0.29
\]

\[
\Phi = 0.5 \cdot \left[ 1 + \alpha \left( \bar{\lambda} - 0.2 \right) + \bar{\lambda}^2 \right]
\]

\[
\Phi = 0.5 \cdot \left[ 1 + 0.21 \left( 0.29 - 0.2 \right) + 0.29^2 \right] = 0.55
\]

\[
\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{0.55 + \sqrt{0.55^2 - 0.29^2}} = 0.97
\]

\[
N_{b,Rd} = \chi \cdot \frac{A \cdot f_y}{\gamma_{M1}} = \frac{0.97 \cdot 13660 \cdot 355}{10} = 4646,7 \, \text{kN}
\]

\[
\frac{N_{Ed}}{N_{b,Rd}} = \frac{1225}{4646} = 0.26 < 1.0
\]

**Diagonal beam check**

All diagonals are loaded with tensile strength in the used loading case. Only S34 has a compression load of 10 kN, which is so small that no buckling calculation is made for this beam.

The chosen profile is identical to the lower bar:

HFRHS 300 x 300 x 12 mm

\( h_t = 300 \, \text{mm} \)

\( A = 13660 \, \text{mm}^2 \)

Tensile check:

\[
\frac{N_{Ed}}{N_{t,Rd}} \leq 10
\]

\[
N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{13660 \cdot 355}{100} = 4849 \cdot 10^3 \, \text{N}
\]

\[
N_{Ed} = \frac{1710 \cdot 10^3}{4849 \cdot 10^3} = 0.35 \leq 1.0
\]

As expected the diagonal beam will not fail due to the tensile loads.

It is even possible to design the lower bar, the diagonals and the verticals in an even smaller profile. A 250x250x12 will also fulfil the required checks.

In the design it is not 100% clear which forces will act on the truss since the function of the lunch plaza is rather flexible, therefore some extra bearing capacity is reserved in the profile.
Vertical deflection

The maximal deflection of the complete truss is given by the equation:

$$u_{\text{max}} = \frac{l}{300} \times \frac{31200}{300} = 104\ mm$$

The calculated deflection in the ULS is 91 mm. The deflection is also calculated in the SLS resulting in a total deflection of 68 mm. This easily satisfies the maximal deflection rule (Figure 4.9).

When the lower bar, verticals and diagonals where designed in 250x250x12 mm profiles the deflection in the SLS would have become 75 mm and still satisfy.

Figure 4.9
The maximum deformation of Truss DD in the SLS is 68 mm.
Truss DD (extreme wind case)

An extra wind load is acting on truss DD causing an extra bending moment in the z direction. This situation increases the instability of the upper and lower bar. The wind from the south-east direction, determined in chapter 3 is used in this case (Figure 4.10).

The wind load acting on the upper bar:

\[ P_{w,d} = 0.8 \cdot 0.85 \cdot 1.045 \cdot 1.2 = 0.85 \text{ kN} / \text{m} \]

Furthermore their will be wind suction appearing at the other side of the truss:

\[ P_{w,d} = -0.5 \cdot 0.85 \cdot 1.045 \cdot 1.2 = -0.53 \text{ kN} / \text{m} \]

Resulting in the following load on the truss:

\[ P_{w,d,ext} = 1.5 \cdot (0.53 + 0.85) = 2.07 \text{ kN} / \text{m} \]

So an approximation of the maximal bending moment can be calculated:

\[ M_{Ed,z} = \frac{1}{8} ql^2 = \frac{1}{8} \cdot 2.07 \cdot 31.2^2 = 251.9 \text{ kNm} \]

The bending moment resistance of the upper bar:

\[ W_{z,Ed} \cdot f_y = \frac{2484 \cdot 10^3 \cdot 355}{100} = 1.24 \cdot 10^6 \text{Nmm} \]

The new stability check for the upper bar, with the local \( M_y \) neglected becomes:

\[
\begin{align*}
\frac{N_{Ed}}{X_y \cdot N_{Rk}} + k_{yz} & \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \cdot M_{z,Rk}} & + k_{yz} & \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \cdot M_{z,Rk}} & \leq 1 \\
\frac{N_{Ed}}{X_z \cdot N_{Rk}} + k_{zz} & \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \cdot M_{z,Rk}} & \leq 1
\end{align*}
\]

\[
\begin{align*}
M_{y,Ed} &= 0 \\
\Delta M_{y,Ed} &= 0
\end{align*}
\]

Resulting in simplified equations:

\[
\begin{align*}
\frac{N_{Ed}}{X_y \cdot N_{Rk}} + k_{yz} & \frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{z,Rk}} \leq 1 \\
\gamma_{M1} & \\
\frac{N_{Ed}}{X_z \cdot N_{Rk}} + k_{zz} & \frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{z,Rk}} \leq 1 \\
\gamma_{M1} & \\
\end{align*}
\]

First \( k_{yz} \) and \( k_{zz} \) are determined.

\[
k_{zz} = C_{nz} \left( 1 + \frac{0.87 - 0.2}{N_{Ed} / \chi_{z} \cdot N_{Rk} / \gamma_{M1}} \right) = 0.90
\]

From \( k_{zz} \), \( k_{yz} \) follows:

\[
k_{yz} = 0.6 \cdot k_{zz} = 0.54
\]

The stability check can now be elaborated:

\[
\begin{align*}
\frac{3860 \cdot 10^3}{0.74 \cdot 6253 \cdot 10^{-7}} + 0.54 & \leq 1 \\
0.83 + 0.11 &= 0.94 < 1 \\
\gamma_{M1} &
\end{align*}
\]

\[
\begin{align*}
\frac{3860 \cdot 10^3}{0.74 \cdot 6253 \cdot 10^{-7}} & + 0.9 \frac{2519 \cdot 10^6}{1237 \cdot 10^6} \leq 1 \\
0.83 + 0.17 &= 1.00 = 1
\end{align*}
\]

Although the \( M_z \) indeed has some influence on the stability of the upper bar it will not cause an instability failure.

Another remark with this calculation is that it is unlikely that an extreme wind-load and extreme snow-load act on the truss at the same time. Making the truss a structural safe element.
Figure 4.10
Simplified view of Truss DD loaded with wind from the south direction. The lower bar is below the main roof surface, therefore all wind is applied to the upper bar which is a product of the wind pressure and suction. Furthermore the springs indicate a stiffness gained by connecting truss DD to the other two trusses.
Truss DD K16 connection

Cross section classification.

K16 is chosen since the S18 bar is the diagonal with the largest tensile stresses in the truss. Therefore this is the governing connection (Figure 4.11)

Bar S18 - diagonal - loaded under tension
Bar S21 - vertical - loaded under compression
Bar S19-S20 - lower bar - loaded under tension

The bars have an overlap of 212 mm. This overlap is applied so the heart-line of the structure intersects in an elegant way. Only the overlapping chord needs to be checked. The efficiency of the overlapped vertical is equalized to the overlapping diagonal chord.

\[ \frac{b}{b_0} = \frac{300}{300} = 1.00 \geq 0.25 \]

All sections under compression are considered as Class 1 sections. For the tensile bar the slenderness of the sections are checked:

\[ \frac{b}{t_0} = \frac{h}{t_0} = \frac{300}{12} = 25 \leq 35 \]

So class 1. Furthermore:

\[ \frac{b}{t_0} = \frac{h}{t_0} = \frac{300}{12} = 25 < 35 \]

And:

\[ \frac{h}{b_0} = \frac{h}{b_0} = \frac{300}{300} = 1.0 > 0.5 \]

\[ \frac{h}{b_0} = \frac{h}{b_0} = \frac{300}{300} = 1.0 < 2.0 \]

So the section slenderness satisfies. Height-to-width ratio is not an issue since the square profiles used have a ratio of 1.0.

The overlapping part is checked and fulfils the following requirements:

\[ \lambda_{ov} = \frac{p}{q} \cdot 100\% = \frac{212}{424} \cdot 100 = 50\% \]

\[ 25\% \leq \lambda_{ov} \leq \lambda_{ov,lim} \]

\[ \lambda_{ov,lim} = 60\% \]
According to NEN-EN 1993-1-8 table 7.10 the connection needs to be checked on two scenarios, such as yielding of the lower bar flange and the diagonal chord failure.

**N-joint S18 - Yielding flange lower bar**

First the yielding flange check is elaborated.

\[
N_{i,Rd} = \frac{8.9 \cdot k_n \cdot f_y \cdot t_0^2 \sqrt{\gamma \left( b_1 + b_2 + h_1 + h_2 \right)}}{\sin \theta} \left/ \gamma_{M5} \right.
\]

With:

- \( k_n = 1.0 \)
- \( f_y = 355 \)
- \( t_0 = 12 \)
- \( \sin \theta = \sin(45) = 0.71 \)
- \( b_0 = 300 \)
- \( b_1 = b_2 = h_1 = h_2 = 300 \)
- \( \gamma_{M5} = 1.00 \)
- \( \gamma = \frac{b_0}{2 \cdot t_0} = \frac{300}{2 \cdot 12} = 12.5 \)

Results in the following resistance value:

\[
N_{i,Rd} = \frac{8.9 \cdot 1.0 \cdot 355 \cdot 12^2 \cdot \sqrt{12.5} \left( 300 + 300 + 300 + 300 \right)}{0.71} \left/ 4 \cdot 300 \right. \left/ \gamma_{M5} \right.
\]

\[ N_{i,Rd} = 2265 \text{ kN} \]

However failure of the diagonal is always the governing factor in these kind of connections.

**N-joint S18 - Diagonal chord failure**

The chord failure scenario with an overlap of 50% is checked with the following equation:

\[
N_{i,Rd} = f_y \cdot t_1 \left( b_{eff} + b_{e,ov} + 2h_1 - 4t_1 \right) / \gamma_{M5}
\]

With:

\[
b_{eff} = b_{e,ov} = \frac{10}{300/12} \cdot 355 \cdot 12 \cdot 300 = 120 \text{ mm}
\]

Results in a design resistance of:

\[
N_{i,Rd} = 355 \cdot 12 (120 + 120 + 2 \cdot 300 - 4 \cdot 12) / 1.0
\]

\[
N_{i,Rd} = 3374 \text{ kN}
\]

According to NEN-EN 1993-1-8 table 7.10 yielding of the lower bar' flange is only appearing when there is no overlap between the bars but a gap instead. Therefore the resistance is equal to 3374 kN.

Check with the diagonal normal force acting on the diagonal:

\[
\frac{N_{S18,Ed}}{N_{S18,Rd}} = \frac{1718}{3374} = 0.51 < 1.00
\]

Therefore the connection satisfies with ease. This was to be expected since there is no large difference in the dimensions of the diagonal, vertical and lower bar. Resulting in a very clear distribution of the loads.
Steel truss (CC3-CC9)

Truss CC is designed in a similar way compared to truss DD. The loads acting on the truss are slightly different since there is no lunch-plaza load acting on the truss. The load due to the embassy is smaller while the roof load is larger. Checks regarding strength and stability are done and the (important) resulting loads are calculated. The calculations are stated briefly.

Loads acting on truss CC

1. Roof

\[ V_{Ed,1} = \text{(Caused by the girders, spanning from truss DD to CC)} \] is equal to 6,99 kN. The girders spanning from truss CC to truss B result in the following \( V_{Ed,2} \).

\[
\begin{align*}
V_{Ed,1} &= 1/2 \cdot ql \\
V_{Ed,2} &= 1/2 \cdot 3,68 \cdot 5,2 = 9,57 \text{ kN} \\
V_{Ed} &= V_{Ed,1} + V_{Ed,2} = 6,99 + 9,57 = 16,56 \text{ kN} \\
F_{\text{roof}} &= V_{Ed} \cdot 4 = 16,56 \cdot 4 = 66,27 \text{ kN}
\end{align*}
\]

2. Own weight

The own weight is translated to point loads in the nodes with an equal value as truss DD: 15,6 kN.

3. Timber glulam beams

The timber beams connected with the steel trusses are calculated before. The resulted point loads acting every 5,2 meter on truss CC are: 118 kN.

4. Embassy point load

Similar to the timber glulam beams reaction forces of the tensile columns of the embassy can be interpreted as point loads on the truss.

\[ \begin{align*}
K6 &= 67 \text{ kN} \\
K9 &= 67 \text{ kN}
\end{align*} \]

Both loads are acting under an angle of 10,5°.

Result MatrixFrame

The loads are illustrated in Figure 4.12.

\[
\begin{align*}
N_{\text{max}} \quad \text{upper bar} &= 2700 \text{ kN} \quad (S32) \\
N_{\text{max}} \quad \text{lower bar} &= 2570 \text{ kN} \quad (S36) \\
N_{\text{max}} \quad \text{diagonal} &= 1100 \text{ kN} \quad (S18)
\end{align*}
\]

\[
\begin{align*}
R_{K1} &= 812 \text{ kN} \\
R_{K2} &= 863 \text{ kN}
\end{align*}
\]

Figure 4.12

Systematization of truss CC with the acting loads.
Since the loads are lower compared with truss DD a new profile is introduced to make the structure slightly more economical. Since the checks are similar as done with truss DD the explanation will be more briefly.

Upper bars: HFRHS 350 x 350 x 12  
Diagonal bars: HFRHS 250 x 250 x 10  
Lower bars: HFRHS 250 x 250 x 10

**Upper bar compression**

\[
N_{c,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{16060 \cdot 355}{100} = 5701 \cdot 10^3 \text{ N}
\]

\[
N_{Ed} = \frac{2700 \cdot 10^3}{5701 \cdot 10^3} = 0.47 < 1.0
\]

So the section satisfies the compression check.

**Upper bar buckling**

\[
N_{cr} = \frac{\pi^2 \cdot EI}{l_{buc}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 3,0 \cdot 10^8}{10400^2} = 5749 \text{ kN}
\]

\[
\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \sqrt{\frac{16060 \cdot 355}{5749 \cdot 10^3}} = 0,99
\]

\[
\Phi = 0.5 \cdot \left[1 + \alpha \left(\bar{\lambda} - 0,2\right) + \bar{\lambda}^2\right]
\]

\[
\Phi = 0.5 \cdot \left[1 + 0.21 \left(0.99 - 0.2\right) + 0.99^2\right] = 1,07
\]

\[
N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0.67 \cdot 16060 \cdot 355}{10} = 3835 \text{ kN}
\]

\[
N_{Ed} = \frac{1700}{3835} = 0,70 < 1,0
\]

The new proposed section for the upper bar fulfils the buckling and strength requirements.

**Lower beam tensile check**

\[
N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{9492 \cdot 355}{100} = 3370 \cdot 10^3 \text{ N}
\]

\[
N_{Ed} = \frac{2570 \cdot 10^3}{3370 \cdot 10^3} = 0.76 < 1,0
\]

So the section satisfies the tensile check.

**Diagonal bar tensile check**

It is clear that the diagonals will not fail since the tensile force acting on the bars is small compared to the lower truss while an equal profile is applied.

**Vertical beam buckling**

The compressional resistance of the chosen profile is no problem, however buckling can occur in this vertical member. Therefore a buckling check is done.

\[
N_{cr} = \frac{\pi^2 \cdot EI}{l_{buc}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 9.0 \cdot 10^7}{2600^2} = 27595 \text{ kN}
\]

\[
\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \sqrt{\frac{9492 \cdot 355}{27595 \cdot 10^3}} = 0,35
\]

\[
\Phi = 0.5 \cdot \left[1 + \alpha \left(\bar{\lambda} - 0,2\right) + \bar{\lambda}^2\right]
\]

\[
\Phi = 0.5 \cdot \left[1 + 0.21 \left(0.35 - 0.2\right) + 0.35^2\right] = 0,51
\]

\[
N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{100 \cdot 9492 \cdot 355}{10} = 3395 \text{ kN}
\]

\[
N_{Ed} = \frac{1100}{3395} = 0,32 < 1,0
\]

The vertical element will not buckle and therefore satisfies the required checks. Even with the reduced profile the section is conservative designed. Again this has to do with the uncertainty of the loads acting on the truss.
Steel truss (B4-B9)

Truss B is designed in a similar way compared to truss CC and DD (Figure 4.13). The loads acting on the truss are slightly higher compared to truss DD, however this truss is smaller since the north-east core is in the same grid. Truss B has a length of 26 meters.

**Loads acting on truss B**

1. **Roof**

   \[ F_{\text{roof}} = V_{\text{Ed}} \cdot 4 = 9.57 \cdot 4 = 38.28 \text{ kN} \]

   \[ F_{\text{roof}} = \frac{V_{\text{Ed}}}{4} = 9.57 \cdot 4 = 38.28 \text{ kN} \]

2. **Own weight**

   The own weight is translated to point loads in the knots with an equal value as truss DD: 15.6 kN.

3. **Timber glulam beams**

   The timber beams connected with the steel trusses are calculated before. The resulted point loads acting every 5,2 meter on truss B are: 118 kN

4. **Embassy point load**

   Similar to the timber glulam beams reaction forces of the tensile columns of the embassy can be interpreted as point loads on the truss.

   Therefore:  
   \[ K18 = 460 \text{ kN} \]
   \[ K8 = 670 \text{ kN} \]

   Both loads are acting at an angle of 10.5°.

5. **Lunch Plaza point load**

   The reaction forces of the Lunch-stand are described earlier. The forces acting on the truss are (according to the MatrixFrame model) very small and compression forces. This will not always be the case, since the platform will never be loaded everywhere as calculated. Therefore the largest tensile force (acting on Truss B) are used.

   Therefore:  
   \[ K5 = 178 \text{ kN} \]
   \[ K6 = 130 \text{ kN} \]

   **Note:** When calculating the tensile column the compression load case cannot be neglected.

**Result MatrixFrame**

\[
\begin{align*}
N_{\text{max}} \text{ upper bar} &= 3050 \text{ kN} \ (S32) \\
N_{\text{max}} \text{ lower bar} &= 2932 \text{ kN} \ (S36) \\
N_{\text{max}} \text{ diagonal} &= 1590 \text{ kN} \ (S18) \\
N_{\text{max}} \text{ vertical} &= 1210 \text{ kN} \ (S57) \\
R_{K1} &= 1135 \text{ kN} \\
R_{K2} &= 1342 \text{ kN}
\end{align*}
\]
Since the loads are lower compared with truss DD a new profile is introduced to make the structure slightly more economical. Since the checks are similar as done with truss DD the explanation will be briefly.

Upper bars: HFRHS 350 x 350 x 12,5  
Diagonal bars: HFRHS 250 x 250 x 10  
Lower bars: HFRHS 250 x 250 x 10

**Upper bar compression**

\[
N_{c,Rd} = \frac{A \cdot f_y \cdot 16700 \cdot 355}{\gamma_{M0} \cdot 100} = 5928 \cdot 10^3 \, N
\]

\[
\frac{N_{Ed}}{N_{c,Rd}} = \frac{3050 \cdot 10^3}{5928 \cdot 10^3} = 0,51 < 1,0
\]

So the section satisfies the compression check.

**Upper bar buckling**

\[
N_{cr} = \frac{\pi^2 \cdot E \cdot l}{l_{buc}^2} = \frac{\pi^2 \cdot 2,1 \cdot 10^5 \cdot 3,1 \cdot 10^8}{10400^2} = 6043 \, kN
\]

\[
\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \sqrt{\frac{16700 \cdot 355}{6043 \cdot 10^3}} = 0,99
\]

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

\[
\Phi = 0,5 \cdot \left[ 1 + 0,21 \left( 0,99 - 0,2 \right) + 0,99^2 \right] = 1,07
\]

\[
\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{1,07 + \sqrt{1,07^2 - 0,99^2}} = 0,67
\]

\[
N_{b,Rd} = \chi \cdot A \cdot f_y \cdot \frac{16700 \cdot 355}{\gamma_{M1}} = 3984 \, kN
\]

\[
\frac{N_{Ed}}{N_{b,Rd}} = \frac{3050}{3984} = 0,77 < 1,0
\]

The new proposed section for the upper bar fulfils the buckling and strength requirements.

**Lower beam tensile check**

\[
N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{9492 \cdot 355}{100} = 3370 \cdot 10^3 \, N
\]

\[
\frac{N_{Ed}}{N_{c,Rd}} = \frac{2932 \cdot 10^3}{3370 \cdot 10^3} = 0,87 < 1,0
\]

So the section satisfies the tensile check.

**Diagonal bar tensile check**

It is clear that the diagonal will not fail since the tensile force acting on the bars is small compared to the lower truss while an equal profile is applied.

**Vertical beam buckling**

The compressional resistance of the chosen profile is no problem, however buckling can occur in this vertical member. Therefore a buckling check is elaborated.

\[
N_{cr} = \frac{\pi^2 \cdot E \cdot l}{l_{buc}^2} = \frac{\pi^2 \cdot 2,1 \cdot 10^5 \cdot 9,0 \cdot 10^7}{2600^2} = 27595 \, kN
\]

\[
\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \sqrt{\frac{9492 \cdot 355}{27595 \cdot 10^3}} = 0,35
\]

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

\[
\Phi = 0,5 \cdot \left[ 1 + 0,21 \left( 0,35 - 0,2 \right) + 0,35^2 \right] = 0,51
\]

\[
\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{0,51 + \sqrt{0,51^2 - 0,35^2}} = 1,00
\]

\[
N_{b,Rd} = \chi \cdot A \cdot f_y \cdot \frac{100 \cdot 9492 \cdot 355}{\gamma_{M1}} = 3395 \, kN
\]

\[
\frac{N_{Ed}}{N_{b,Rd}} = \frac{1210}{3395} = 0,36 < 1,0
\]

The vertical element will not buckle and therefore satisfies the required checks. Even with the reduced profile the section is conservative designed. Again this has to do with the uncertainty of the loads acting on the truss.

---

**Figure 4.13**  
Systematisation of truss B with the acting loads.
Modified roof section

The calculated steel trusses have an increased height compared to the original designed trusses. Therefore the roof section is modified. The middle (lifted) part increased slightly with as positive result: more space for the Building Services below. Furthermore, the redefined profiles are applied in this section (Figure 4.14).

1. Truss B
2. Truss CC
3. Truss DD
4. Building services
5. Embassy
Figure 4.14
Technical section of the roof. The lifted part of the roof increased 800 mm in height. This has no negative influence on the appearance of the building design since the lifted part is hardly visible from street level.
5 Hofjes
Timber floor overview
Sub-beam calculation
Main-beam calculation
Timber truss 2.1
Truss connection timber-timber
Truss connection steel tube
3D analysis connection
Floor-truss connection
Truss-Truss connection
3D detail
Timber floor structure (overview)

A part of the structural floor plan level +6000 is pictured in Figure 5.3 with the 1: Sub beams and 2: main beams. The floor is separated in two elements, in reality the complete beam structure works as one complete system. The timber beams are designed as beams with a timber CLT slab on top, forming a T structure in both directions.
Introducing Kerto

The properties of Kerto are very beneficial when it comes to the bending strength. For the initial deflection mode however the modulus of elasticity is the contributing material property. The modulus of elasticity of Kerto however is almost equal to laminated timber (which is logical since it is both spine wood). The benefit is gained by activating the whole section. The section which is activated has a T shape. The strength of the upper flange is not disrupted by openings due to piping and building services.

When activating the section it is of great importance that the core-to-core distance of the sub beam parts of the T shape section is not too large. This means that the core-to-core distance is decreased from 2,6 meter in the preliminary design to 1 meter in the final design. This measure is taken to prevent the upper part of the beam to have a different deflection compared to the beam part of the section leading to an impossible connection.

The floor structure is build up by sub beams of 5,2 meters in length, supported by large main beams (up to 15,6 meters in length). This heavy floor structure is calculated in two 2D parts.

1. Check of the sub beams with the resulting loads as point loads for the main beams.

2. Check of the main beam.

It is to be expected that the deflection and shear force will cause problems since the span of the main beam is huge with a rejuvenation applied at the supports with the timber trusses.

A T beam system is chosen since the web will be visible from the floor below (Figure 5.1), creating a striped pattern in the ceiling. Practically seen the T beams are easier to combine with installations and lightning fixtures compared to I beams. The slab is from now on named flange while the beam is named web.

Figure 5.3
Floor section with sub beams (1), main beams (2) and aluminium ceiling (3) indicated.
The timber floor structure is designed with two types of Kerto LVL products of the manufacturer Metsawood. The applied products are Kerto-S for the web of the section and the Kerto-Q panels for the flange. Both products are best suitable for large spans combined with high loads. Where the Kerto-S panel have all veneer layers oriented parallel to the length of the beam the Kerto-Q is a cross-laminated product (Figures 5.4 and 5.5). In these plates the amount of cross laminated veneers is around 1:5th of the total veneer layers. It is possible to design a custom plate with fibres in the directions as preferred.

**Figure 5.4**  
Kerto-S All veneer layers glued in equal direction

**Figure 5.5**  
Characteristic values and physical properties for KERTO products.

<table>
<thead>
<tr>
<th>Property</th>
<th>Symbol</th>
<th>Kerto-S</th>
<th>Kerto-Q</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bending strength</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Edgewise (depth 300 mm)</td>
<td>$f_{m,0,\text{edge},k}$</td>
<td>44,0</td>
<td>36,0</td>
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<tr>
<td>Size effect parameter</td>
<td>$s$</td>
<td>0,12</td>
<td>0,12</td>
</tr>
<tr>
<td>Flatwise, parallel to grain</td>
<td>$f_{m,0,\text{flat},k}$</td>
<td>50</td>
<td>36</td>
</tr>
<tr>
<td>Flatwise, perpendicular to grain</td>
<td>$f_{m,90,\text{flat},k}$</td>
<td>-</td>
<td>8,0</td>
</tr>
<tr>
<td><strong>Tensile strength</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Parallel to grain (length 3000 mm)</td>
<td>$f_{0,k}$</td>
<td>35,0</td>
<td>26,0</td>
</tr>
<tr>
<td>Perpendicular to grain, edge</td>
<td>$f_{90,k}$</td>
<td>0,8</td>
<td>6,0</td>
</tr>
<tr>
<td><strong>Compressive strength</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Parallel to grain (length 3000 mm)</td>
<td>$f_{c,0,k}$</td>
<td>35,0</td>
<td>26,0</td>
</tr>
<tr>
<td>Perpendicular to grain, edge</td>
<td>$f_{c,90,\text{edge},k}$</td>
<td>6,0</td>
<td>9,0</td>
</tr>
<tr>
<td>Perpendicular to grain, flat</td>
<td>$f_{c,90,\text{flat},k}$</td>
<td>1,8</td>
<td>2,2</td>
</tr>
<tr>
<td><strong>Shear strength</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Edgewise</td>
<td>$f_{v,0,\text{edge},k}$</td>
<td>4,1</td>
<td>4,5</td>
</tr>
<tr>
<td>Flatwise, parallel to grain</td>
<td>$f_{v,0,\text{flat},k}$</td>
<td>2,3</td>
<td>1,3</td>
</tr>
<tr>
<td>Flatwise, perpendicular to grain</td>
<td>$f_{v,90,\text{flat},k}$</td>
<td>-</td>
<td>0,6</td>
</tr>
<tr>
<td><strong>Modulus of elasticity</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Parallel to grain, along</td>
<td>$E_{\text{0,mean}}$</td>
<td>13,800</td>
<td>10,500</td>
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<tr>
<td>Perpendicular to grain, edge</td>
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<tr>
<td>Perpendicular to grain, flat</td>
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<td>Parallel to grain, across</td>
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<td>600</td>
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<tr>
<td>Flatwise, parallel to grain</td>
<td>$G_{0,\text{flat,mean}}$</td>
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<td>120</td>
</tr>
<tr>
<td><strong>Other</strong></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Characteristic density</td>
<td>$\rho_k$</td>
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<td>480</td>
</tr>
<tr>
<td>Mean density</td>
<td>$\rho_{\text{mean}}$</td>
<td>510</td>
<td>510</td>
</tr>
<tr>
<td>Performance in fire, charring rate</td>
<td>$\beta_n$</td>
<td>0,7</td>
<td>0,7</td>
</tr>
</tbody>
</table>
Timber floor structure (sub-beam)

Profile (part 1)

Beams (300 mm) connected to a thick floor plate of 69 mm make a fictional T beam.

Center-to-center distance: 1300 mm
Length: 5200 mm

Web: Kerto S
h = 300 mm
b = 75 mm
A = 22500 mm²

Flange: Kerto Q
h = 69 mm
b = b_{eff}

Effective width is determined with use of the following image:

![Effective width diagram]

According to NEN-EN 9.1.2:

\[ b_{eff} = b_{c,eff} + b_w = 520 + 75 = 595 \, \text{mm} \]
\[ \min \left\{ b_{c,eff} = 0.1 \cdot l = 0.1 \cdot 5200 = 520 \, \text{mm} \right\} \]
\[ b_{c,eff} = 20 \cdot h_t = 20 \cdot 69 = 1380 \, \text{mm} \]
\[ b_w = 75 \, \text{mm} \]

Resulting: Kerto Q flange

h = 69 mm
b_{eff} = 595 mm
b_{c,t,c} = 1300 mm
A = 89700 mm²

Combined A of flange and web = 112200 mm²

Loads acting on the sub-beam

Permanent loads
Floor plate 0,2 kN/m²
Aluminium ceiling 0,1 kN/m²
Lightning fixtures 0,2 kN/m²
Kerto-Q flange
\[ A \cdot p_k = 89700 \cdot 480 \cdot 10^{-6} = 0,43 \, \text{kN/m}^1 \]
Kerto-S web
\[ A \cdot p_k = 22500 \cdot 480 \cdot 10^{-6} = 0,108 \, \text{kN/m}^1 \]
Total permanent load 1,18 kN/m¹

Note: Although the b_{eff} is smaller than b_{c,t,c} the last value is taken for calculating the loads acting on the profile.

Variable loads
Office floor 2,5 kN/m²
Total variable load 3,25 kN/m¹

Load combinations:

ULS
\[ q_d = 1.2 \cdot P_k + 15 \cdot Q = \]
\[ q_d = 1.2 \cdot 1,18 + 15 \cdot 3,25 = 6,3 \, \text{kN} / \text{m}^1 \]

SLS
\[ q_d = 1.0 \cdot P_k + 1.0 \cdot Q = \]
\[ q_d = 1.0 \cdot 1,18 + 1.0 \cdot 3,25 = 4,44 \, \text{kN} / \text{m}^1 \]

This q loads acts on a reduced length, since it is assumed that the main beam carries a part of the q load with a width of b_{eff}.
The $b_{eff}$ of the main beam is considered to be 1860 mm. Half of this length (960 mm) carries the $q$ load at the supports. This is visualised in the MatrixFrame model (Figure 5.6).

Similar calculations are done for the SLS resulting in the following values:

- $V_{Ed,\text{per}} = 2.0$ kN
- $V_{Ed,\text{var}} = 5.4$ kN
- $V_{Ed} = 7.4$ kN

The $V_{Ed}$ values are processed as point loads acting on the main beam. Nevertheless a check regarding the sub-beams need to be elaborated.
Profile (part 2)

The properties of the combined section are calculated.

Reduced values:
The final E modulus is reduces by a climatic factor taking the creep factor into account. In the case of climate class 1 this is 0.6. In the ULS the effect of the load may be reduced. In office buildings an value of 0.3 may be applied, in the SLS this is not the case. Also a design E modulus is determined.

ULS
Web:
\[ E_{\text{mean,fn}} = \frac{E_{\text{mean}}}{1 + \psi_2 \cdot k_{\text{def}}} = \frac{13800}{1 + 0.3 \cdot 0.6} = 11695 \text{ N/mm}^2 \]
\[ \psi_2 = 0.3 \]
\[ k_{\text{def}} = 0.6 \]
\[ E_d = E_{\text{mean}} \cdot \frac{13800}{1.2} = 11500 \text{ N/mm}^2 \]

Flange:
\[ E_{\text{mean,fn}} = \frac{E_{\text{mean}}}{1 + \psi_2 \cdot k_{\text{def}}} = \frac{10500}{1 + 0.3 \cdot 0.6} = 8898 \text{ N/mm}^2 \]
\[ \psi_2 = 0.3 \]
\[ k_{\text{def}} = 0.6 \]
\[ E_d = E_{\text{mean}} \cdot \frac{10500}{1.2} = 8750 \text{ N/mm}^2 \]

SLS
Web:
\[ E_{\text{mean,fn}} = \frac{E_{\text{mean}}}{1 + k_{\text{def}}} = \frac{13800}{1 + 0.6} = 8625 \text{ N/mm}^2 \]

Flange:
\[ E_{\text{mean,fn}} = \frac{E_{\text{mean}}}{1 + k_{\text{def}}} = \frac{10500}{1 + 0.6} = 6562 \text{ N/mm}^2 \]

For the strength calculations the design E modulus is used, for the deformations the \( E_{\text{mean}} \) is used, since the creep factor is taken into account at the deflection calculation.

The two parts of the section cooperate and therefore a combined EI needs to be calculated. \( E_0 \) = web and \( E_1 \) = flange.

\[ Z_0 = \frac{E_0 b_0 \left(h_v^2\right) + E_1 b_1 \left(2 \cdot h_v h + 2h_1 + h_1^2\right)}{2 \cdot (E_0 b_0 h_0 + E_1 b_h)} \]
\[ Z_0 = \frac{11500 \cdot 75 (300^2) + 8750 \cdot 595 (2 \cdot 300 \cdot 69 + 69^2)}{2 \cdot (11500 \cdot 75 \cdot 300 + 8750 \cdot 595 \cdot 69)} \]
\[ Z_o = 257,25 \text{ mm} \]
\[ Z_b = 300 + 69 - Z_0 = 111,75 \text{ mm} \]

\[ EI = \left[ \frac{E_0 b_0 (Z_0)^3}{3} + \frac{E_0 b_0 (Z_b - h)^3}{3} + \frac{E_1 b_1 (Z_b - h)^3}{2} \right] \]
\[ EI = \left[ \frac{11500 \cdot 75 (257,3)^3}{3} + \frac{11500 \cdot 75 (111,7 - 69)^3}{3} + \frac{8750 \cdot 595 (111,7 - 69) 69 (111,7 - 2 \cdot 69 + 69^2)}{3} \right] \]
\[ EI = 7,20 \cdot 10^{12} \]
\[ I = 6,8 \cdot 10^6 \]

A check is made with the Steiner method resulting in an equal EI as stated above. The \( Z_0 \) is smaller than with the regular method, since in the method used (presented by Jorissen)\(^5,3\) takes the difference in E moduli already taken into account.
Design strength

The characteristic value of the tensile strength is modified due to the material factor. The volume effect is found in the Kerto properties chart. The reduced characteristic strength is used for calculating the design strength of the profile according to the rule:

\[ R_d = k_{mod} \frac{R_k}{\gamma_M} \]

With a \( k_{mod} \) of 0,80 since the variable floor load is the shortest acting loads. The \( \gamma_M \) factor is a material property, in the case of LVL this is 1,2.

For the web:

\[ f_{1,0,k} = 35,0 \ N/mm^2 \]

\[ k_f = \min \left( \frac{11}{3000} \right)^{6/2} = \min \left( \frac{11}{3000} \right)^{0.12/2} = \min \left( \frac{11}{0,968} \right) \]

\[ f_{1,0,k} = 35 \cdot 0,968 = 33,9 \ N/mm^2 \]

\[ k_h = \min \left( \frac{300}{12} \right)^{0.12} = 1,0 \]

\[ f_{m,0,edge,k} = 44 \cdot 1 = 44 \ N/mm^2 \]

Kerto-S design values (web):

\[ f_{c,0,d} = k_{mod} \frac{f_{c,0,k}}{\gamma_M} = 0,8 \frac{35}{12} = 23,3 \ N/mm^2 \]

\[ f_{1,0,d} = k_{mod} \frac{f_{1,0,k}}{\gamma_M} = 0,8 \frac{33,8}{12} = 22,58 \ N/mm^2 \]

\[ f_{m,0,d} = k_{mod} \frac{f_{m,0,k}}{\gamma_M} = 0,8 \frac{44}{12} = 29,3 \ N/mm^2 \]

For the flange:

\[ f_{1,0,k} = 26,0 \ N/mm^2 \]

\[ k_f = \min \left( \frac{11}{3000} \right)^{6/2} = \min \left( \frac{11}{3000} \right)^{0.12/2} = \min \left( \frac{11}{0,968} \right) \]

\[ f_{1,0,k} = 0,968 \cdot 26 = 25,2 \ N/mm^2 \]

\[ k_p = \min \left( \frac{300}{12} \right)^{0.12} = 1,19 \]

\[ f_{m,0,k} = 1,19 \cdot 36 = 42,84 \ N/mm^2 \]

Kerto-Q design values (flange):

\[ f_{c,0,d} = k_{mod} \frac{f_{c,0,k}}{\gamma_M} = 0,8 \frac{26}{12} = 17,3 \ N/mm^2 \]

\[ f_{1,0,d} = k_{mod} \frac{f_{1,0,k}}{\gamma_M} = 0,8 \frac{25,2}{12} = 16,8 \ N/mm^2 \]

\[ f_{m,0,d} = k_{mod} \frac{f_{m,0,k}}{\gamma_M} = 0,8 \frac{42,84}{12} = 28,56 \ N/mm^2 \]
Bending stress check

The vertical loads acting on the section cause a bending moment. Since the profile is not a rectangular beam it is not possible to simply check the beam on the bending strength. The bending strength needs to be decomposed to compressive and tensile stresses acting on different areas in the profile.

1. The upper part of the flange.
2. The lower part of the flange.
3. The upper part of the web.
4. The lower part of the web.

Figure 5.7 illustrates the bending stresses acting on an I beam with fully connected flange to web. The critical points of the normal stress are similar to the T profile.

With Figure 5.7 into account the following normal stresses occur:

\[ \sigma_{01} = \frac{E_0 (Z_0 - h_0)}{EI} M \]
\[ = \frac{11500 (257.3)}{7.3 \cdot 10^{12}} \cdot 1.9 \cdot 10^7 = 7.80 \text{ N/mm}^2 \]

\[ \sigma_{02} = \frac{E_0 (Z_b - h_b)}{EI} M \]
\[ = \frac{11500 (1117 - 69)}{7.3 \cdot 10^{12}} \cdot 1.9 \cdot 10^7 = 1.30 \text{ N/mm}^2 \]

\[ \sigma_{21} = \frac{E_2 (Z_b - h_b)}{EI} M \]
\[ = \frac{8750 (1117 - 69)}{7.3 \cdot 10^{12}} \cdot 1.9 \cdot 10^7 = 0.99 \text{ N/mm}^2 \]

\[ \sigma_{20} = \frac{E_2 (Z_b)}{EI} M \]
\[ = \frac{8750 (1117)}{7.3 \cdot 10^{12}} \cdot 1.9 \cdot 10^7 = 2.58 \text{ N/mm}^2 \]

Now the bending stress check is elaborated:

\[ \frac{\sigma_{01}}{f_{m,0,d}} = \frac{7.80}{29.3} = 0.27 \leq 1.0 \]
\[ \frac{\sigma_{02}}{f_{m,0,d}} = \frac{1.30}{23.5} = 0.05 \leq 1.0 \]
\[ \frac{\sigma_{21}}{f_{m,0,d}} = \frac{0.98}{28.56} = 0.03 \leq 1.0 \]
\[ \frac{\sigma_{20}}{f_{m,0,d}} = \frac{2.58}{28.56} = 0.09 \leq 1.0 \]

The beam fulfils the normal stress requirements with ease. It is to be expected that the shear force between the flange and the web will be the governing factor. Therefore a check regarding shear forces is elaborated.
Shear stress check (simple method)

Similar to the normal stresses the shear stresses are calculated at different parts in the profile. The locations are displayed in Figure below.

1. In the neutral line (= median)
2. In the interface between the web and top flange
3. In the interface between the web and bottom flange (=0 since no bottom flange is present)
4. Top flange
5. Bottom flange (=0 in the T profile)

Position 1:
\[
\sigma_{v,1} = \frac{E_s (2Z_b - h_2) b_3 h_2 + E_0 (Z_b - h_2)^2 b_0}{2EI b_0} Q
\]
\[
\sigma_{v,1} = \frac{8750(2 \cdot 111,7 - 69)595 \cdot 69 + 11500(111,7 - 69)^2 \cdot 75}{2 \cdot 7,33 \cdot 10^{12} \cdot 75} \cdot 11,4 \cdot 10^3 = 0,61 \text{ N/mm}^2
\]

Position 2:
\[
\sigma_{v,2} = \frac{E_s (2Z_b - h_2) b_3 h_2 + E_0 (Z_b - h_2)^2 b_0}{2EI b_0} Q
\]
\[
\sigma_{v,2} = \frac{8750(2 \cdot 111,7 - 69)595 \cdot 69}{2 \cdot 7,33 \cdot 10^{12} \cdot 75} \cdot 11,4 \cdot 10^3 = 0,59 \text{ N/mm}^2
\]

Position 4:
\[
\sigma_{v,4} = \frac{E_s (2Z_b - h_2) (b_2 - b_0)}{4EI} Q
\]
\[
\sigma_{v,4} = \frac{8750(2 \cdot 111,7 - 69)(595 - 75)}{4 \cdot 7,33 \cdot 10^{12}} \cdot 11,4 \cdot 10^3 = 0,28 \text{ N/mm}^2
\]

The calculated shear stresses need to be checked with the design values for the shear strength of the timber structure. These values are determined in a similar way as the tensile and compressive stress strength.

Kerto-S (web):
\[
f_{v,0,edge,d} = k_{mod} f_{v,0,edge,k} \gamma_M = 0,8 \frac{4,1}{12} = 2,73 \text{ N/mm}^2
\]

Kerto-Q (flange):
\[
f_{v,90,flat,d} = k_{mod} f_{v,90,flat,k} \gamma_M = 0,8 \frac{0,6}{12} = 0,4 \text{ N/mm}^2
\]

Now the shear stress check is elaborated:

Position 1 (edgewise):
\[
\frac{\sigma_{v,1}}{f_{v,0,edge,d}} = 0,61 = 0,22 < 1,0
\]

Position 2 (flatwise):
\[
\frac{\sigma_{v,1}}{f_{v,90,flat,d}} = 0,59 = 1,46 > 1,0
\]

Position 4 (edgewise):
\[
\frac{\sigma_{v,1}}{f_{v,0,edge,d}} = \frac{0,28}{3,00} = 0,09 < 1,0
\]
With the simplified calculation method the shear stresses are not met. Therefore the flange will start ‘rolling’ over the web. A more detailed calculation is done for the flange to find the correct shear stresses, this calculation is elaborated on page 108.

First the shear stresses at position 2 are reduced due the fact that the flange is build up by different veneers, with different fibre directions. This results in a reduced E modulus for the flange since not every wood-fibre is oriented in the same direction.

Furthermore it is possible to play with the direction of the veneer layer connecting to the web. When the direction of the first layer connecting to the web is in the same direction of the fibres of the web, another shear capacity can be used. In this case:

\[ f_{v,0,\text{flat},d} = k_{\text{mod}} \frac{f_{v,0,\text{flat},k}}{\gamma_M} = 0.8 \cdot \frac{1.3}{12} = 0.88 \text{ N/mm}^2 \]

Resulting in a new check at position 2:

\[ \sigma_{v,1} \leq f_{v,0,\text{flat},d} \]

\[ \frac{\sigma_{v,1}}{f_{v,90,\text{flat},d}} = \frac{0.59}{0.87} = 0.68 < 10 \]

The connection fulfils.

However, the problem of rolling fibres is not eliminated, it is moved to the first veneer layer with a different fibre direction. A detailed Mathematica script is designed to calculate the shear stresses between all layers of the flange and with the web. After doing this a custom veneer layout is designed to fulfil the shear strength of the flange.

The detailed elaboration of the Mathematica script is shown at the main beam calculation of the floor at page 108.

**Deflections SLS**

The deflections of the sub-beam are expected to be small, however it is important to know them since it will be added to the deflection of the main-beams. In this calculation the effect of creep is taken into account with the instantaneous factor and the \( k_{\text{def}} \).

\[
\begin{align*}
u_{\text{in},G} &= u_{\text{in},G} + u_{\text{in},Q_1} + u_{\text{in},Q_2} \\
u_{\text{in},G} &= u_{\text{inst},G} \cdot (1 + \psi_{2,0} k_{\text{def}}) \\
u_{\text{in},Q_1} &= u_{\text{inst},Q_1} \cdot (1 + \psi_{2,0Q_1} k_{\text{def}}) \\
u_{\text{in},Q_1} &= u_{\text{inst},Q_1} \cdot (\psi_{Q_1} + \psi_{2,0} k_{\text{def}})
\end{align*}
\]

Furthermore the E modulus is not (!) reduced in the SLS as calculated earlier, otherwise the creep factor would have been doubled. With the found deflections caused by the permanent (2 mm) and variable load (6 mm):

\[
\begin{align*}
\sigma_{v,1} \leq f_{v,0,\text{flat},d} \\
u_{\text{in}} &= u_{\text{in},G} + u_{\text{in},Q_1} + u_{\text{in},Q_2} \\
u_{\text{in}} &= 3.2 + 7.08 = 10.28 \text{ mm} \\
u_{\text{in},G} &= u_{\text{inst},G} \cdot (1 + \psi_{2,0} k_{\text{def}}) \\
u_{\text{in},G} &= 2 \cdot (1+0.6) = 3.2 \text{ mm} \\
u_{\text{in},Q_1} &= u_{\text{inst},Q_1} \cdot (1 + \psi_{2,0Q_1} k_{\text{def}}) \\
u_{\text{in},Q_1} &= 6 \cdot (1+0.3 \cdot 0.6) = 7.08 \text{ mm}
\end{align*}
\]

The total deflection is small and this seems to be normal with the relatively low loads, short span and large profile.

Accepted deflection of the beam:

\[
\frac{u_{\text{max}}}{l} = \frac{5200}{250} = 20.8 \text{ mm}
\]
Figure 5.8
3D Section of hofje +6000
Timber floor structure (main-beam)

Profile (part 1)

Beams (900 mm) connected to a thick floor plate of 69 mm makes a fictional T beam.

Center-to-center distance: 5200 mm
Length (max): 15600 mm

Web: Kerto S
- \( h = 900 \text{ mm} \)
- \( b = 300 \text{ mm} \)
- \( A = 270000 \text{ mm}^2 \)

Flange: Kerto Q
- \( h = 69 \text{ mm} \)
- \( b = b_{\text{eff}} \)

Effective width is determined with use of the following image:

According to NEN-EN 9.1.2:\n\[
b_{\text{eff}} = b_{c,\text{eff}} + b_w = 1560 + 300 = 1860 \text{ mm}
\]
\[
\min \left\{ \begin{array}{l}
b_{c,\text{eff}} = 0.1 \cdot l = 0.1 \cdot 15600 = 1560 \text{ mm} \\
b_{c,\text{eff}} = 20 \cdot h_i = 20 \cdot 69 = 1725 \text{ mm}
\end{array} \right.
\]
\( b_w = 300 \text{ mm} \)

Resulting: Kerto Q flange
- \( h = 69 \text{ mm} \)
- \( b_{\text{eff}} = 1860 \text{ mm} \)
- \( b_{c,\text{eff}} = 5200 \text{ mm} \)
- \( A = 128340 \text{ mm}^2 \)

Combined A of flange and web = 398340 mm²

Loads acting on the main-beam

Permanent loads
- Floor plate: 0,2 kN/m²
- Aluminium ceiling: 0,1 kN/m²
- Lightning fixtures: 0,2 kN/m²
- Kerto-Q flange:
  \( A \cdot \rho_k = 128340 \cdot 480 \cdot 10^{-8} = 0,62 \text{ kN/m}^1 \)

Kerto-S web
\( A \cdot \rho_k = 270000 \cdot 480 \cdot 10^{-8} = 1,29 \text{ kN/m}^1 \)

Total permanent load = 2,84 kN/m¹

Note: Only the loads acting on \( b_{\text{eff}} \) are taken into account. All other loads are carried by the sub-beams and redirected to the main-beam with point loads.

Variable loads
- Office floor: 2,5 kN/m²
- Total variable load = 4,65 kN/m¹

Load combinations:

ULS
\( q_d = 1,2 \cdot P_k + 1,5 \cdot Q \) =
\( q_d = 1,2 \cdot 2,84 + 1,5 \cdot 4,65 = 10,4 \text{ kN / m}^1 \)

SLS
\( q_d = 1,0 \cdot P_k + 1,0 \cdot Q \) =
\( q_d = 1,0 \cdot 2,84 + 1,0 \cdot 4,65 = 7,49 \text{ kN / m}^1 \)

The loads of the sub-beams and the extra q load are applied to the beam resulting in Figure 5.9. The effect of the point loads is very visible in the \( V_{\text{Ed}} \) line. In reality it will be a more fluent line since the flange will distribute all loads.
Results ULS & SLS

\[ V_{Ed} = 195 \text{ kN} \]
\[ M_{Ed} = 802 \text{ kNm} \]

For the SLS:
\[ V_{Ed} = 146 \text{ kN} \]
\[ M_{Ed} = 602 \text{ kNm} \]

Profile (part 2)
The properties of the combined section are calculated. Reduced design values, similar as the sub-beam:

**ULS**
Web:
\[ E_{\text{mean,fn}} = \frac{E_{\text{mean}}}{(1+\psi_2\cdot k_{\text{def}})} = \frac{13800}{1+0.3\cdot0.6} = 11695 \text{ N/mm}^2 \]
\[ E_d = \frac{E_{\text{mean}}}{\gamma_M} = \frac{13800}{1.2} = 11500 \text{ N/mm}^2 \]
Flange:
\[ E_{\text{mean,fn}} = \frac{E_{\text{mean}}}{(1+\psi_2\cdot k_{\text{def}})} = \frac{10500}{1+0.3\cdot0.6} = 8898 \text{ N/mm}^2 \]
\[ E_d = \frac{E_{\text{mean}}}{\gamma_M} = \frac{10500}{1.2} = 8750 \text{ N/mm}^2 \]

**SLS**
Web:
\[ E_{\text{mean,fn}} = \frac{E_{\text{mean}}}{(1+k_{\text{def}})} = \frac{13800}{1+0.6} = 8625 \text{ N/mm}^2 \]
Flange:
\[ E_{\text{mean,fn}} = \frac{E_{\text{mean}}}{(1+k_{\text{def}})} = \frac{10500}{1+0.6} = 6562 \text{ N/mm}^2 \]
The two parts of the section cooperate and therefore a combined EI needs to be calculated. $E_0 = \text{web}$ and $E_1 = \text{flange}$.

$$Z_0 = \frac{E_0 b_0 \left(h_0^2\right) + E_1 b_1 \left(2 \cdot h_0 h_1 + 2h_1 + h_1^2\right)}{2 \cdot \left(E_0 b_0 h_0 + E_1 b_1 h_1\right)}$$

$$Z_0 = \frac{11500 \cdot 300 \left(900^2\right) + 8750 \cdot 1860 \left(2 \cdot 900 \cdot 69 + 69^2\right)}{2 \cdot \left(11500 \cdot 300 \cdot 900 + 8750 \cdot 1860 \cdot 69\right)}$$

$$Z_0 = 578,69 \text{ mm}$$

$$Z_0 = 900 + 69 - Z_0 = 390,31 \text{ mm}$$

$$ EI = \left[ \frac{E_0 b_0 \left(Z_0\right)^3}{3} + \frac{E_0 b_0 \left(Z_0 - h_0\right)^3}{3} + \frac{E_1 b_1 \left(Z_0 - h_1\right) h_1 \left(Z_0 - \frac{1}{2} h_1\right) + E_1 b_1 h_1^2}{2} \left(Z_0 - \frac{1}{3} h_1\right)\right]$$

$$EI = \left[ \frac{11500 \cdot 300 \left(578,6\right)^3}{3} + \frac{11500 \cdot 300 \left(390,31 - 69\right)^3}{3} + \frac{8750 \cdot 1860 \left(390,31 - 69\right) \left(390,31 - \frac{1}{2} \cdot 69\right)}{2} \right]$$

$$EI = 4,03 \cdot 10^{14}$$

$$l \approx 3,8 \cdot 10^{10}$$

A check is made with the Steiner method resulting in an equal EI as stated above. The $Z_0$ is smaller than with the regular method, since in the method used presented by Jorissen the difference in $E$ is already taken into account.

### Design strength

The characteristic value of the tensile strength is modified due to the material factor. The volume effect is found in the Kerto properties chart.

For the web:

$$f_{1,0,k} = 35,0 \text{ N} / \text{mm}^2$$

$$k_i = \min\left(\frac{3000}{l}\right)^{0.12} = \min\left(\frac{3000}{15600}\right)^{0.12} = \min\left(\frac{1,1}{0,91}\right) = 0,91$$

$$f_{1,0,k} = 35 \cdot 0,91 = 31,7 \text{ N} / \text{mm}^2$$

$$k_p = \min\left(\frac{300}{h}\right)^{0.12} = \min\left(\frac{300}{900}\right) = 0,88$$

$$f_{m,0,\text{edge},k} = 44 \cdot 0,88 = 38,72 \text{ N} / \text{mm}^2$$

For the flange:

$$f_{1,0,k} = 26,0 \text{ N} / \text{mm}^2$$

$$k_i = \min\left(\frac{3000}{l}\right)^{0.12} = \min\left(\frac{3000}{15600}\right)^{0.12} = \min\left(\frac{1,1}{0,91}\right) = 0,91$$

$$f_{1,0,k} = 0,91 \cdot 26 = 23,5 \text{ N} / \text{mm}^2$$

$$k_p = \min\left(\frac{300}{h}\right)^{0.12} = \min\left(\frac{300}{69}\right) = 1,19$$

$$f_{m,0,k} = 1,19 \cdot 36 = 42,84 \text{ N} / \text{mm}^2$$
Bending stress check

The vertical loads acting on the section cause a bending moment. Since the profile is not a rectangular beam it is not possible to simple check the beam on the bending strength. The bending strength needs to be decomposed to compressive and tensile stresses acting on different areas in the profile.

1. The upper part of the flange.
2. The lower part of the flange.
3. The upper part of the web.
4. The lower part of the web.

Figure 5.10 illustrates the normal forces acting on an I beam with fully connected flange to web. The critical points of the normal stress are similar to the T profile.

This reduces characteristic strength is used for calculating the design strength of the profile according to the rule:

\[ R_d = k_{\text{mod}} \frac{R_k}{\gamma_M} \]

With a \( k_{\text{mod}} \) of 0.80 since the variable floor load is the shortest acting loads. The \( \gamma_m \) factor is a material property, in the case of LVL this is 1.2.

Kerto-S (web):

\[
\begin{align*}
    f_{c,0,d} &= k_{\text{mod}} \frac{f_{c,0,k}}{\gamma_M} = 0,8 \frac{35}{12} = 23,3 \text{ N/mm}^2 \\
    f_{1,0,d} &= k_{\text{mod}} \frac{f_{1,0,k}}{\gamma_M} = 0,8 \frac{31,7}{12} = 21,1 \text{ N/mm}^2 \\
    f_{m,0,d} &= k_{\text{mod}} \frac{f_{m,0,k}}{\gamma_M} = 0,8 \frac{38,7}{12} = 25,8 \text{ N/mm}^2 
\end{align*}
\]

Kerto-Q (flange):

\[
\begin{align*}
    f_{c,0,d} &= k_{\text{mod}} \frac{f_{c,0,k}}{\gamma_M} = 0,8 \frac{26}{12} = 17,3 \text{ N/mm}^2 \\
    f_{1,0,d} &= k_{\text{mod}} \frac{f_{1,0,k}}{\gamma_M} = 0,8 \frac{23,5}{12} = 15,7 \text{ N/mm}^2 \\
    f_{m,0,d} &= k_{\text{mod}} \frac{f_{m,0,k}}{\gamma_M} = 0,8 \frac{42,84}{12} = 28,6 \text{ N/mm}^2 
\end{align*}
\]
With Figure 5.10 into account the following normal stresses occur:

\[
\sigma_{01} = \frac{E_0 (Z_b)}{EI} M = \frac{11500 \times (578.68)}{4.03 \times 10^{14}} \cdot 80,1 \cdot 10^7 = 13,20 \, \text{N} / \text{mm}^2
\]

\[
\sigma_{02} = \frac{E_0 (Z_b - h_2)}{EI} M = \frac{11500 \times (390.3 - 69)}{4.03 \times 10^{14}} \cdot 80,1 \cdot 10^7 = 7,33 \, \text{N} / \text{mm}^2
\]

\[
\sigma_{21} = \frac{E_2 (Z_b - h_2)}{EI} M = \frac{8750 \times (390.3 - 69)}{4.03 \times 10^{14}} \cdot 80,1 \cdot 10^7 = 5,58 \, \text{N} / \text{mm}^2
\]

\[
\sigma_{20} = \frac{E_2 (Z_b)}{EI} M = \frac{8750 \times (390.3)}{4.03 \times 10^{14}} \cdot 80,1 \cdot 10^7 = 6,77 \, \text{N} / \text{mm}^2
\]

Now the normal stress check is elaborated:

\[
\sigma_{1,0,d} \leq f_{m,0,d}
\]

\[
\sigma_{v,0,d} \leq f_{m,0,d}
\]

\[
\frac{\sigma_{01}}{f_{m,0,d}} = \frac{13,20}{25,8} = 0,51 \leq 1,0
\]

\[
\frac{\sigma_{02}}{f_{m,0,d}} = \frac{7,33}{25,8} = 0,28 \leq 1,0
\]

\[
\frac{\sigma_{21}}{f_{m,0,d}} = \frac{5,41}{28,6} = 0,19 \leq 1,0
\]

\[
\frac{\sigma_{20}}{f_{m,0,d}} = \frac{6,77}{28,6} = 0,24 \leq 1,0
\]

The extra normal load caused by the wind-load is not taken into account. This will be elaborated further on in a different load-case.

---

**Shear stress check (simple method)**

Similar to the normal stresses the shear stresses are calculated at different locations in the profile. The locations are displayed in the figure below. This simple method is similar to the sub-beam.

1. In the neutral line (= median)
2. In the interface between the web and top flange
3. In the interface between the web and bottom flange (=0 since no bottom flange is present)
4. Top flange
5. Bottom flange (=0 in the T profile)

### Position 1:

\[
\sigma_v = \frac{E_2 (2Z_b - h_2) b_2 h_2 + E_2 (Z_b - h_2)^2 b_0 Q}{2EI} Q
\]

\[
\sigma_{v,1} = \frac{8750(2 \cdot 390.3 - 69) \cdot 1860 \cdot 65 + 11500(390.3 - 69)^2 \cdot 300}{2 \cdot 4.03 \cdot 10^{14} \cdot 300} \cdot 195 \cdot 10^3 = 0,93 \, \text{N} / \text{mm}^2
\]

### Position 2:

\[
\sigma_{v,2} = \frac{E_2 (2Z_b - h_2) b_2 h_2 Q}{2EI} Q
\]

\[
\sigma_{v,2} = \frac{8750(2 \cdot 390.3 - 69) \cdot 1860 \cdot 69}{2 \cdot 4.03 \cdot 10^{14} \cdot 300} \cdot 195 \cdot 10^3 = 0,64 \, \text{N} / \text{mm}^2
\]
Position 4:
\[
\sigma_{v,4} = \frac{E_z (2Z_b - h_b) (b_2 - b_0)}{4EI} \cdot Q \\
\sigma_{v,4} = \frac{8750 (2 \cdot 390,3 - 69) (1860 - 300)}{4 \cdot 4,03 \cdot 10^{14}} \cdot 195 \cdot 10^3 \\
\sigma_{v,4} = 1,17 \text{ N/mm}^2
\]

The calculated shear stresses need to be checked with the design values for the shear strength of the timber structure. These values are determined in a similar way as the tensile and compressive stress strength.

Kerto-S (web):
\[
f_{v,0,\text{edge},d} = k_{\text{mod}} f_{v,0,\text{edge},k} = 0,8 \cdot \frac{4,1}{12} = 0,273 \text{ N/mm}^2
\]

Kerto-Q (flange):
\[
f_{v,0,\text{edge},d} = k_{\text{mod}} f_{v,0,\text{edge},k} = 0,8 \cdot \frac{4,5}{12} = 0,3 \text{ N/mm}^2
\]
\[
f_{v,90,\text{flat},d} = k_{\text{mod}} f_{v,90,\text{flat},k} = 0,8 \cdot \frac{0,6}{12} = 0,4 \text{ N/mm}^2
\]

Now the shear stress check is elaborated:

Position 1 (edgewise):
\[
\frac{\sigma_{v,1}}{f_{v,0,\text{edge},d}} \leq 0,93 \\
\frac{\sigma_{v,1}}{2,73} = 0,34 < 1,0
\]

Position 2 (flatwise):
\[
\frac{\sigma_{v,1}}{f_{v,90,\text{flat},d}} \leq 0,64 \\
\frac{\sigma_{v,1}}{0,40} = 1,61 > 1,0
\]

Position 4 (edgewise):
\[
\frac{\sigma_{v,1}}{f_{v,0,\text{edge},d}} \leq 0,93 \\
\frac{\sigma_{v,1}}{3,00} = 0,33 < 1,0
\]

With the simple calculation method the shear stresses are not met at position 2. Therefore the flange will start rolling over the web. Check:
\[
\tau_{\text{mean},d} \leq \begin{cases} 
    f_{v,90,d} & \text{if } b_w \leq 8 \cdot h_i \\
    f_{v,90,d} \left( \frac{8h_i}{h_i} \right) & \text{if } b_w > 8 \cdot h_i 
\end{cases}
\]
\[
8 \cdot h_i = 8 \cdot 69 = 552 > b_w
\]

A more detailed calculation is done for the flange to find the correct shear stresses.

In the book Timber structures 35.3 and Structural Timber Design to Eurocode5.4 methods are described to calculate the shear stresses in a multi layered plate. The so called panel shear will occur between the first layer which is perpendicular oriented to the beam span. The method prescribes a flange of three layers and one of seven layers.

The Kerto-Q flange of 69 mm in height has 23 layers and therefore the example is extended with help of Mathematica. Furthermore the example shows an E1,E2,E1 structure of the layers, while the Kerto-Q panel has a different build up.

| = Parallel to beam direction 
- = Perpendicular to beam direction 

Kerto-Q panel: 18x parallel, 5x perpendicular. 
||-||-||-||-||-||-||-||-||-||-|

The theory of panel shear and the designed script is explained on the next pages followed by the calculations of the shear stress.
Three layered flange example

Situation A (Figure 5.11):
Fibre direction top veneers = span direction
Rigid in the span direction.
Flexible in the direction perpendicular to the panel

Situation B (Figure 5.11):
Fibre direction perpendicular to the span
Flexible in the span direction
Rigid perpendicular to the span direction

In situation B the flange will start rolling ($f_{v,90,d}$) over
the web, while in situation A panel shear occurs
between flange and web ($f_{v,0,d}$). The rolling shear will
occur between layer 1 and 2. However, the surface
($b_{eff}$) of the shear force acting between layer 1 and 2
is larger than the width of the web, assumed that the
shear forces spreads under an angle of 45 degrees.
Situation A is more beneficial for the shear distribution.
However it needs to be checked of the panel shear is
activated. This can be checked with the shear stress
ratio. The following steps are elaborated:

1. Calculate strain
2. Define stresses
3. Equate shear stress layer 1-2
4. Equate shear stress layer 1-web
5. Check the ratio of the stresses and choose the
determining factor.
First the interface between layer 1 and layer 2 is checked: Shear force spreads over a width of:

\[ b_{\text{eff}} = b_{\text{web}} + 2 \cdot \frac{1}{3} h \]

\[ \sigma_{v,1-2} = \frac{F}{b_0 + \frac{2}{3} h} = \frac{\left(2Z - \frac{1}{3} h\right)E_1 + (2Z - h)E_2}{6EI\left(b_0 + \frac{2}{3} h\right)} \cdot bh \cdot Q \]

Interface between layer 1 and web:

\[ \sigma_{v,1-2} = \frac{F}{b_0} = \frac{(4Z - 2h)E_1 + (2Z - h)E_2}{6EI(b_0)} \cdot bh \cdot Q \]

The ratio \( \frac{\sigma_{v,1-2}}{\sigma_{v,\text{web}}} \) in relation to the ratio \( \frac{f_{v,90,d}}{f_{v,0,d}} \) indicates if the rolling shear of the panel shear is governing. Most common the rolling shear between layer 1 and 2 is governing.

\[ \frac{\sigma_{v,1-2}}{\sigma_{v,\text{web}}} = \frac{E_1\left(2Z - \frac{1}{3} h\right) + E_2(2Z - h)}{E_1(4Z - 2h) + E_2(2Z - h)} \cdot \frac{b_0}{b_0 + \frac{2}{3} h} \]

For a seven layer flange this equation becomes:

\[ \frac{\sigma_{v,1-2}}{\sigma_{v,\text{web}}} = \frac{E_1\left(6Z - \frac{15}{7} h\right) + E_2(6Z - 3h)}{E_1(8Z - 4h) + E_2(6Z - 3h)} \cdot \frac{b_0}{b_0 + \frac{2}{7} h} \]
Shear stress check (Advanced method)

The theory of the 3 layer plate is translated to a model with 23 layers. First the E modulus of both veneer directions is assumed to be equal. In this way it becomes possible to verify the results of the model.

\[
\begin{align*}
Z & = 390.31 \text{ mm} \\
h & = 69 \text{ mm (height flange)} \\
b_w & = 300 \text{ mm} \\
b & = 1860 \text{ mm (b_{eff})} \\
a & = 23 \text{ (amount of layers)} \\
E_1 & = 8750 \text{ N/mm}^2 \\
E_2 & = 8750 \text{ N/mm}^2 \text{ (fictional)} \\
EI & = 4,10 \cdot 10^{14} \text{ Nmm}^2 \\
M & = 801 \cdot 10^6 \text{ Nmm} \\
Q & = 195000 \text{ N}
\end{align*}
\]

The strains of the element are organised in relation with the amount of veneer layers, a short description is showed.

\[
\begin{align*}
\varepsilon_0 &= \frac{MZ}{EI} \\
\varepsilon_1 &= \frac{Z - \frac{(a-0)}{a}}{Z} \varepsilon_0 \\
\varepsilon_2 &= \frac{Z - \frac{(a-1)}{a}}{Z} \varepsilon_0 \\
\varepsilon_3 &= \frac{Z - \frac{(a-2)}{a}}{Z} \varepsilon_0 \\
\ldots \varepsilon_{23} &= \frac{Z - \frac{(a-23)}{a}}{Z} \varepsilon_0
\end{align*}
\]

The stresses are calculated similar as the 3 layer example, although \(E_2 = E_1\) in this case.

\[
\begin{align*}
\sigma_1 &= \varepsilon_1 \cdot E_1 \\
\sigma_2 &= \varepsilon_2 \cdot E_1 \\
\sigma_3 &= \varepsilon_2 \cdot E_2 \\
\sigma_4 &= \varepsilon_3 \cdot E_2 \\
\ldots \sigma_{45} &= \varepsilon_{23} \cdot E_1 \\
\ldots \sigma_{46} &= \varepsilon_{24} \cdot E_1
\end{align*}
\]

In the model an extra step is introduced to make the force acting between the layers visible. In the example this step is directly executed in the last step.

\[
\begin{align*}
l_{1-2} &= \frac{(\sigma_1 + \sigma_2) \cdot 1}{2} \cdot \frac{h}{a} \\
l_{2-3} &= \frac{(\sigma_2 + \sigma_3) \cdot 1}{2} \cdot \frac{h}{a} \\
l_{22-23} &= \frac{(\sigma_{43} + \sigma_{44}) \cdot 1}{2} \cdot \frac{h}{a} \\
l_{23-24} &= \frac{(\sigma_{46} + \sigma_{45}) \cdot 1}{2} \cdot \frac{h}{a}
\end{align*}
\]

Now the stresses are calculated, similar to the example:

\[
\begin{align*}
\sigma_{1-2} &= \frac{\left( \sum_{i=1}^{23} l_{i-i+1} l_{i-2} \right)}{M} \cdot Q = 0.60 \text{ N/mm}^2 \\
\sigma_{web-1} &= \frac{\left( \sum_{i=1}^{23} l_{i-i+1} \right)}{b_w} \cdot Q = 0.64 \text{ N/mm}^2 \\
\frac{\sigma_{1-2}}{\sigma_{web-1}} &= \frac{0.60}{0.64} = 0.94
\end{align*}
\]
The shear force acting between the first veneer layer and the web is equal to the shear force found with the simple method earlier. The model is also checked with numeric values used in the Timber Structures 3 book.

Furthermore it is noticed that the shear force decreased to 94% from the first to the second veneer layer.

Now the values of the model are adjusted to the standard Kerto-Q 69 mm veneer orientation, making the contact area between veneer 2 and 3 critical.

Shear stresses are calculated:

\[
\sigma_{2-3} = \left( \frac{\sum_{i=1}^{23} l_{i-\{i+1\}} - l_{1-2} - l_{2-3}}{M} \right) \cdot Q = 0.46 \text{ N/mm}^2
\]

\[
\sigma_{\text{web}-1} = \frac{b_w + \left( 2 \cdot 2 \cdot h \right)}{a} \cdot Q = 0.53 \text{ N/mm}^2
\]

\[
\frac{\sigma_{2-3}}{\sigma_{\text{web}-1}} = 0.46 \cdot 0.86 = 0.39
\]

The shear stresses did indeed decrease.

Check regarding the shear strength of the profile:

\[
\frac{f_{v,90,d}}{f_{v,0,d}} = \frac{0.4}{0.87} = 0.46
\]

0.86 > 0.46 and the rolling shear between layer 2 and layer 3 is therefore governing.

\[
\frac{\sigma_{2-3}}{f_{v,90,d}} = \frac{0.46}{0.40} = 1.15 > 1.0
\]

Does still not fulfil.

A new (custom) veneer structure is introduced with more veneer layers parallel to the beam direction at the edges of the flange, resulting in a larger height over which the shear stresses can spread:

Resulting in a new shear stress calculation, presented on the next page.
The different pattern is visible in calculating the normal stresses in the flange.

\[
\begin{align*}
\text{layer 1:} & \quad \sigma_1 = \epsilon_1 \cdot E_1 \\
& \quad \sigma_2 = \epsilon_2 \cdot E_1 \\
\text{layer 2:} & \quad \sigma_3 = \epsilon_2 \cdot E_1 \\
& \quad \sigma_4 = \epsilon_3 \cdot E_1 \\
\text{layer 3:} & \quad \sigma_5 = \epsilon_3 \cdot E_1 \\
& \quad \sigma_6 = \epsilon_4 \cdot E_1 \\
\text{layer 4:} & \quad \sigma_7 = \epsilon_4 \cdot E_1 \\
& \quad \sigma_8 = \epsilon_5 \cdot E_1 \\
\text{layer 5:} & \quad \sigma_9 = \epsilon_5 \cdot E_2 \\
& \quad \sigma_{10} = \epsilon_6 \cdot E_2 \\
\text{layer 6:} & \quad \sigma_{11} = \epsilon_6 \cdot E_1 \\
& \quad \sigma_{12} = \epsilon_7 \cdot E_1 \\
\text{etc} & \\
\end{align*}
\]

Resulting in the following shear stresses:

\[
\sigma_{4-5} = \frac{\sum_{i=1}^{23} l_{i-3(i+1)} - l_{i-2} - l_{i-3} - l_{i-4} - l_{i-5}}{M} \cdot Q = 0.39 \text{ N/mm}^2
\]

\[
\sigma_{\text{web}-1} = \frac{\sum_{i=1}^{23} l_{i-3(i+1)}}{b_w} \cdot Q = 0.53 \text{ N/mm}^2
\]

\[
\frac{\sigma_{2-3}}{\sigma_{\text{web}-1}} = \frac{0.39}{0.53} = 0.74
\]

Check regarding the shear strength of the profile: 0.74 > 0.46 and the rolling shear between layer 4 and layer 5 is therefore governing.

\[
\sigma_{4-5} < \frac{f_{v,90,d}}{0.40} = 0.98 < 1.0
\]

Does fulfill. So the new layer system of: ||||-||-|||-|||-||-|||| fulfills. This is understandable since the shear force has more height to spread across the profile. Making a flange with less (but larger) veneer layers would also improve the properties, however this is not possible with the Kerto products.

**Shear check sub-beam**

The new flange is also applied at the sub-beam (same flange) and therefore the check is also elaborated for the sub-beam.

\[
\begin{align*}
Z & = 111.7 \text{ mm} \\
h & = 69 \text{ mm (height flange)} \\
b_w & = 75 \text{ mm} \\
b & = 595 \text{ mm (b eff)} \\
a & = 23 \text{ (amount of layers)} \\
E_1 & = 8750 \text{ N/mm}^2 \\
E_2 & = 1667 \text{ N/mm}^2 \\
E_1 & = 7.33 \times 10^{12} \text{ N/m}^2 \\
Q & = 11400 \text{ N}
\end{align*}
\]

Resulting in the following shear stresses:

\[
\sigma_{4-5} = \frac{\sum_{i=1}^{23} l_{i-3(i+1)} - l_{i-2} - l_{i-3} - l_{i-4} - l_{i-5}}{M} \cdot Q = 0.31 \text{ N/mm}^2
\]

\[
\sigma_{\text{web}-1} = \frac{\sum_{i=1}^{23} l_{i-3(i+1)}}{b_w} \cdot Q = 0.48 \text{ N/mm}^2
\]

\[
\frac{\sigma_{2-3}}{\sigma_{\text{web}-1}} = \frac{0.31}{0.48} = 0.65
\]
\[
\sigma_{4-5} < f_{v,90,d} \\
\frac{\sigma_{4-5}}{f_{v,90,d}} = 0.31 = 0.78 < 1.0
\]

Does fulfil and therefore the sub-beam and the main-beam sections fulfil. However there is a rejuvenation applied at the end of the main-beams, they have be checked again.

**Rejuvenation main beam**

The main beams have a rejuvenation at each ending, this has aesthetic as well practical reasons. The beam seems to disappear in the ceiling, adding mystery to the structure, the structure becomes (visually) much lighter and advanced. Furthermore the rejuvenation is critical considering the detailing of the façade (Figure 5.13). The maximal beam (web) height at the end of the beam is 490 mm to maintain the dimensions of the white line in the façade. The rejuvenation reduces the web from 900 to 490 mm.
First a check is done to prove that the concentrated forces at the rejuvenation can be neglected. Known: tensile and compressive forces act parallel to the grain.

According the Eurocode the angle of the rejuvenation can not be steeper than:

\[
\frac{1}{i} = \frac{1}{10}
\]

Where \( i \) is the distance between the maximal and the minimal beam height.

Max height: 969 mm (extreme case)
Min height: 490 mm
Difference: 390 mm
Length \( i \) (minimal): 4800 mm

The rejuvenation is not larger than \( Z_0 \), prescribing that all changes happen in the tensile zone of the profile.

The Eurocode prescribes rules for the shear forces at the supports for rectangle beams with a rejuvenation. This is checked for the T beam to give an indication of the shear force problem at the supports.

\[
\tau_d = \frac{1.5 \cdot V}{b h_{\text{eff}}} \leq k_v \cdot f_{v,d}
\]

With the following values:

\( b = 300 \ \text{mm} \)
\( h_{\text{eff}} = 510 \ \text{mm} \)
\( V = 195000 \ \text{N} \)
\( f_{v,0,\text{edge,d}} = \frac{k_{\text{mod}}}{\gamma_M} f_{v,0,\text{edge,k}} = 0.8 \cdot \frac{4.5}{1.2} = 2.73 \)

\[
k_v = \min \left\{ \frac{1}{k_n \left( 1 + \frac{1.1 \cdot i^{1/5}}{\sqrt{h}} \right)}, \frac{1}{\sqrt{h} \left( \alpha (1 - \alpha) + 0.8 \frac{x}{h} \sqrt{\frac{1}{\alpha - \alpha^2}} \right)} \right\}
\]

\[
\alpha = \frac{h_{\text{eff}}}{h} = \frac{510}{969} = 0.52
\]
\( k_n = 4.5 \) (LVL)
\( x = 300 \)

Resulting in an \( k_v \) of

\[
k_v = \min \left\{ \frac{1}{4.5 \left( 1 + \frac{1.1 \cdot 4800^{1/5}}{\sqrt{969}} \right)}, \frac{1}{\sqrt{969} \left( 0.52 (1 - 0.52) + 0.8 \frac{300}{969} \frac{1}{0.52} - 0.52^2 \right)} \right\}
\]

\[
k_v = \min \left\{ \frac{1}{0.56} \right\}
\]

The shear force check of the rectangle example is done:

\[
\tau_d = \frac{1.5 \cdot V}{b h_{\text{eff}}} \leq k_v \cdot f_{v,d}
\]

\[
\tau_d = \frac{1.5 \cdot 195 \cdot 10^3}{300 \cdot 510} = 1.91 \ \text{N/mm}^2
\]

\[
k_v \cdot f_{v,d} = 0.56 \cdot 2.73 = 1.53 \ \text{N/mm}^2
\]

\[\tau_d < k_v \cdot f_{v,d}\]

The check does not suffice for the rejuvenation of a rectangle beam. The T profile has a larger EI, making the rejuvenation possible. However the rolling shear and panel shear of the flange is problematic. Therefore a detailed check is elaborated, similar to the main-beam full section. First the new EI is calculated. A normal stress check is not necessary since close to the supports the bending moment acting on the beam is rather small.
The two parts of the section cooperate and therefore a combined EI needs to be calculated. \( E_0 = \text{web} \) and \( E_1 = \text{flange} \).

\[
Z_0 = \frac{E_0 b_0 (h_0^2) + E_b (2 \cdot h_1 h_i + 2h_i^2)}{2 \cdot (E_0 b_0 h_0 + E_b h_i)}
\]

\[
Z_0 = 11500 \cdot 300 \left( \frac{441^2}{441^2 + 2 \cdot 441 \cdot 69 + 69^2} \right) + 8750 \cdot 1860 \left( \frac{2 \cdot 441 \cdot 69 + 69^2}{2 \cdot (11500 \cdot 300 \cdot 441 + 8750 \cdot 1860 \cdot 69)} \right)
\]

\[
Z_0 = 328,33 \text{ mm}
\]

\[
Z_b = 441 + 69 - Z_0 = 1812 \text{ mm}
\]

\[
EI = \left[ \frac{E_0 b_0 (Z_0)^3}{3} + \frac{E_b (Z_b - h_i) (h_i (Z_b - \frac{1}{2} h_i) + \frac{E_b h_i^2}{2} (Z_b - \frac{1}{3} h_i))}{3} \right] + \left[ \frac{11500 \cdot 300 (328,78)^3}{3} + \frac{11500 \cdot 300 (18121 - 69)^3}{3} + \left[ 8750 \cdot 1860 (18121 - 69) (18121 - \frac{1}{2} \cdot 69) + \frac{8750 \cdot 1860 \cdot 69^2}{2} (18121 - \frac{1}{3} \cdot 69) \right] \right]
\]

\[
EI = 6.71 \cdot 10^{13}
\]

\[
l \approx 6.4 \cdot 10^9
\]

**Shear check analysis simple method**

Three positions are checked. Web, web-flange and flange beginning with position 1 (web):

\[
\sigma_{v1} = \frac{E_2 (2Z_b - h_2) b_2 h_2 + E_0 (Z_b - h_2)^2 b_0 Q}{2EI \cdot b_0}
\]

\[
\sigma_{v1} = \frac{8750 (2 \cdot 18121 - 69) 1860 \cdot 69 + 11500 (18121 - 69)^2 300}{2 \cdot 6.71 \cdot 10^{13} \cdot 300} \cdot 195 \cdot 10^3 = 1.81 N / mm^2
\]

Position 2:

\[
\sigma_{v2} = \frac{E_2 (2Z_b - h_2) b_2 h_2 Q}{2EI \cdot b_0}
\]

\[
\sigma_{v2} = \frac{8750 (2 \cdot 18121 - 69) 1860 \cdot 69}{2 \cdot 6.71 \cdot 10^{13} \cdot 300} \cdot 195 \cdot 10^3 = 1.59 N / mm^2
\]

Position 4:

\[
\sigma_{v4} = \frac{E_2 (2Z_b - h_2) (b_2 - b_0) Q}{4EI}
\]

\[
\sigma_{v4} = \frac{8750 (2 \cdot 18121 - 69) (1860 - 300)}{4 \cdot 6.71 \cdot 10^{13}} \cdot 195 \cdot 10^3 = 2.91 N / mm^2
\]

Now the shear stress check is elaborated:

Position 1 (edgewise):

\[
\sigma_{v1} \leq \frac{f_{v,0,\text{edge},d}}{f_{v,0,\text{edge},d}} \quad \frac{\sigma_{v1}}{f_{v,0,\text{edge},d}} = 0.66 < 10
\]

Position 2 (flatwise):

\[
\sigma_{v1} \leq \frac{f_{v,0,\text{flat},d}}{f_{v,90,\text{flat},d}} \quad \frac{\sigma_{v1}}{f_{v,90,\text{flat},d}} = 1.84 > 10
\]

Position 4 (edgewise):

\[
\sigma_{v1} \leq \frac{f_{v,0,\text{edge},d}}{f_{v,0,\text{edge},d}} \quad \frac{\sigma_{v1}}{f_{v,0,\text{edge},d}} = 0.96 < 10
\]

The beam does not suffice in the interface between web and flange. Besides the fact that the rolling shear within the flange needs to be checked, the \( f_{v,0} \) value does not suffice either.
When calculating with the advanced method the connection does not satisfy either. The shear stresses found at the connection between flange and web is 1,31 N/mm². The resistance of 0,87 N/mm² is clearly not enough to transfer the loads.

An alternative section is proposed in which extra timber is added to the sides of the rejuvenated beam. The new maximal width is found by an iterative process with Mathematica, resulting in a web-width of 640 mm at most (Figure 5.14).

\[
\begin{align*}
Z &= 223 \text{ mm} \\
h &= 69 \text{ mm} \\
b_w &= 640 \text{ mm} \\
b &= 1860 \text{ mm} \\
a &= 23 \\
E_1 &= 8750 \text{ N/mm}^2 \\
E_2 &= 1667 \text{ N/mm}^2 \\
EI &= 1.07 \cdot 10^{14} \text{ Nmm}^2 \\
Q &= 195000 \text{ N}
\end{align*}
\]

With this new value the shear stress between the flange and web becomes:

\[
\frac{\sigma_{v1}}{f_{v,90,flat,d}} = \frac{0.49}{0.87} = 0.56 < 1.0
\]

And the rolling shear stress between layer 4 and 5 of the flange becomes:

\[
\frac{\sigma_{4-5}}{f_{v,90,d}} = \frac{0.39}{0.40} = 0.98 < 1.0
\]

So the beam suffices with a rejuvenation since the width of the beam increases locally.

Figure 5.14
Rejuvenated beam, new section isometric drawing. By enlarging the width at the supports the beam is capable of transferring the shear loads.
**Vibrations floor**

The Eurocode prescribes rules for vibrations in floors for housing. The hofjes floor is checked according to those rules to provide an indication. First the natural frequency of the floor is determined:

\[
f_l = \frac{\pi}{2 \cdot f^2} \sqrt{\frac{(EI)}{m}} = \frac{\pi}{2 \cdot 15.6^2} \sqrt{\frac{4.10 \cdot 10^7}{102}} = 12.89 \text{ Hz}
\]

\[(EI)_l = 4.10 \cdot 10^7 \text{ Nm}^2 / \text{m}
\]

\[m = 102 \text{ kg} / \text{m}^2\]

A fictional point load is placed on the floor and the deflection due to that point load is calculated. The point load of 20 kN results in a \(w_{\text{fin}}\) of 9.6 mm.

\[\frac{w}{F} \leq a\]

\[\frac{w}{F} = \frac{9.6}{20} = 0.48 < a\]

\[a = 1\]

\[b = 120\]

\(a\) and \(b\) are given in the Dutch nation annex. The first check fulfills, however a second check needs to be done.

\[\nu \leq b^{(\xi - 1)}/l\]

\[\xi = 0.01, f_l = 12.89\]

\[b^{(\xi - 1)} = 0.015\]

\(V\) is calculated with the following equation:

\[V = \frac{4 (0.4 + 0.6 \cdot n_{40})}{m \cdot b \cdot l + 200} = \frac{4 (0.4 + 0.6 \cdot 1)}{102 \cdot 35 \cdot 15.6 + 200} = 7.1 \cdot 10^{-5}\]

\[n_{40} = 1\]

So: \(\nu \leq b^{(\xi - 1)}\) and vibrations will therefore not cause problems in the floor. The vibrations will even be smaller since the floor surface and spans are assumed conservative.

**Deflections SLS**

The deflections of the main-beam can be calculated with the following equations:

\[u_{\text{fin},G} = u_{\text{inst},G} + u_{\text{fin},Q_i} + u_{\text{fin},Q_i}\]

\[u_{\text{fin},G} = u_{\text{inst},G} \cdot (1 + \psi_{2,G} k_{\text{def}})\]

\[u_{\text{fin},Q_i} = u_{\text{inst},Q_i} \cdot (1 + \psi_{2,Q_i} k_{\text{def}})\]

Resulting in the following equations:

\[u_{\text{fin},G} = u_{\text{inst},G} \cdot (1 + \psi_{2,G} k_{\text{def}})\]

\[u_{\text{fin},Q_i} = u_{\text{inst},Q_i} \cdot (1 + \psi_{2,Q_i} k_{\text{def}})\]

\[u_{\text{fin},Q_i} = 38 \cdot (1 + 0,3 \cdot 0,6) = 44,8 \text{ mm}\]

An extra deflection is caused by the deflection of the sub-beams. Furthermore camber (Dutch: zeeg) is applied to the main beam with the height of the deflection caused by the permanent load. In this case 33.6 mm.

\[w_{\text{net.fin}} = (w_{\text{inst.per}} + w_{\text{inst.var}}) + w_{\text{creep}} - w_c\]

\[w_{\text{net.fin}} = 21 + 38 + 19.4 - 33.6 = 44,8 \text{ mm}\]

\[w_{\text{net.fin,sub}} = 44,8 + 10,3 = 55,1 \text{ mm}\]

Maximal allowed deflection:

\[w_{\text{max}} = \frac{l}{250} = \frac{15600}{250} = 62,4 \text{ mm}\]

So the beam suffices with the applied sheer.
Wind load on floor

The floors of the hofjes are exposed to a horizontal wind load. It is assumed that the wind load is 100% transferred by the floor to the infinite stiff concrete core. It is assumed that the 69 mm height flange of the floor will take the complete wind load and that it is prevented from buckling due to the timber webs of the floor. This means that the floor is much stiffer in reality than calculated so far and no buckling check is required.

The influence of the wind load on the stresses in the floor of hofje +12 m is checked. Two methods are used to check the floor: A fictional beam method in Matrixframe, and a more precise 2D plate model in SCiA. Both are loaded with a line load.

Wind load

Wind pressure coefficient (line load)

\[
F_{w1} = c_p c_d c_r q_p (b) \cdot A_{ref}
\]

\[
q_{w1} = 0.884 \cdot 0.8 \cdot 0.85 \cdot 5 = 2.99 \text{ kN/m}^1
\]

Wind suction coefficient (line load)

\[
F_{w1} = c_p c_d c_r q_p (b) \cdot A_{ref}
\]

\[
q_{w1} = 0.884 \cdot 0.5 \cdot 0.85 \cdot 5 = -1.87 \text{ kN/m}^1
\]

These values are summarized and multiplied by a factor 1.5.

\[
Q_{w1} = 1.5 \cdot 2.99 = 4.49 \text{ kN/m}^1
\]

\[
Q_{w2} = 1.5 \cdot 1.87 = 2.80 \text{ kN/m}^1
\]

\[
Q_{w2} = 4.49 + 2.80 = 7.29 \text{ kN/m}^1
\]

These values are applied to the beam and plate model.

Matrix model approach

The floor is simplified to a beam on two supports, representing the connections with core A (Figure 5.15). The MatrixFrame model results in a maximal horizontal force of 97.67 kN at the left wall support. The normal stresses are calculated with a strip of 1 meters in width.

\[
\sigma_{wind} = \frac{F}{A} = \frac{97670}{69 \cdot 1000} = 1.41 \text{ N/mm}^2
\]

This extra stress do not form a problem since the normal stresses are by far not governing. (The shear force in the flanges are governing). Therefore no extra checks are executed.

SCIA model approach

To get a better view on the stresses in the floor due to wind loads the floor is modelled in SCIA Engineer (Figure 5.16). The loads of the suction and pressure are both applied for wind from the south direction as well for the wind from the west direction. It turned out that the maximum stresses appear at the connection between the floor and the left wall of the core. This was to be expected since a fictional rotation point is formed at this point (1). The core is modelled as a stiff line connection, resulting in a sigma y of 0.35 N/mm² (pressure). Furthermore a sigma x of 0,6 N/mm² (pressure) appears at the north side of the left wall (2). For the wind load case from the north a tensile stress of 0,6 N/mm² appears in point 2.

The deflection is showed to indicate that the model is correct since the deflection makes sense.

Again the stresses are small, that the results are much smaller compared to the line model. This can be explained since in the SCIA model the slab is activated and the stresses are better spread. Nevertheless a check regarding the normal stresses is irrelevant.
Figure 5.15
Line model of the floor performed in Matrixframe.

Figure 5.16
(Deformed) model of the SCIA slab check loaded with wind from the south direction. Left the stresses in the y direction, right the stresses in the x direction. Both show the critical stress areas.
First, the truss is calculated according to the original design (Figure 5.17) to get an indication of the required dimensions and connections. This means that the truss has equal upper and lower bars with a height of 490 mm and a width of 240 mm each. The diagonals used to be square elements of 350x350 mm. However these dimensions are adjusted to 340x360 mm. This has to do with the minimal and maximal distance between bolts, creating a more suitable connection.

Figure 5.17
Truss 2.1 is indicated in the isometric drawing with:
1. Upper bar 2x (240x490 mm)
2. Lower bar 2x (240x490 mm)
3. Diagonal 1x (340x360 mm)
4. Vertical 1x (340x240 mm)
The truss section is pictured left (Figure 5.18), please notice that the drawing does not contain connections between the bars. These connections where not elaborated in the architectural design. The upper and lower bar are both separated in two bars. Since the bars act as normal compression or tensile bars it is not the height (compared to a regular beam) which matters but it is the surface. Therefore it is beneficial to split the beam in two wider, but lower beams. Resulting in a structure suitable to fit in the floor package. Another great advantage of this multi-layered truss is the stronger and symmetrical connection properties between the upper bar and diagonal it is creating.

The connections of the diagonals with the upper and lower bars are proposed in the floor package and therefore invisible from the inside. A connection calculation is made further on to test if this is possible or an alternative connection/truss is required.

**Figure 5.18**
Truss 2.1 section
**Loads acting on truss**

An overview of the different loads acting on truss DD is given. Three different types of loads are applied:
1. Point loads from the upper floors and roof.
2. Reaction forces of the floor.
3. Wind loads in the x and y direction.

**Point loads**

The loads acting on the truss are occasionally reduced, this means that another part of the building transfers the remaining part of the load (primarily the concrete cores). Furthermore the instantaneous factors are used, this is allowed since the floor of hofje +6000 m is loaded extreme. A safety factor of 1,35 is used for all loads except for the resulting loads of the steel trusses, the safety factor of this load is taken into account earlier.

<table>
<thead>
<tr>
<th>Type</th>
<th>Load</th>
<th>Reduction</th>
<th>Ψ</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel trusses reaction loads</td>
<td>2037</td>
<td>0,5</td>
<td>0,7</td>
<td>713 kN</td>
</tr>
<tr>
<td>Roof (west of core A)</td>
<td>760</td>
<td>1</td>
<td>0,7</td>
<td>532 kN</td>
</tr>
<tr>
<td>Level +16000</td>
<td>1200</td>
<td>1</td>
<td>0,5</td>
<td>600 kN</td>
</tr>
<tr>
<td>Level +16000 routing</td>
<td>1455</td>
<td>0,5</td>
<td>0,5</td>
<td>363 kN</td>
</tr>
<tr>
<td>Level +12000</td>
<td>1181,5</td>
<td>1</td>
<td>0,5</td>
<td>590 kN</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>2799 kN</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Total including safety factor</strong></td>
<td>3529 kN</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Total divided by four</strong></td>
<td>882 kN</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>West (left) 120%</td>
<td></td>
<td></td>
<td></td>
<td>1059 kN</td>
</tr>
<tr>
<td>East (right) 80%</td>
<td></td>
<td></td>
<td></td>
<td>706 kN</td>
</tr>
</tbody>
</table>

**Floor load**

The reaction loads of a 15.6 m beam are calculated earlier with a value of 195 kN each support. This value is reduced linear for smaller beams.
**Wind load**

The wind load acting on the truss will increase the instability of the truss and needs therefore to be taken into account. However the wind load is by far smaller than the normal loads.

\[ P_{w,d} = 0.8 \cdot 0.85 \cdot 0.88 \cdot 4.0 = 2.39 \text{ kN/m} \]

This wind load is applied on a simplified 2D beam system, resulting in a maximal bending moment of 77 kNm at the left support.

**MatrixFrame input and results**

The truss is 2D modelled in MatrixFrame (Figure 5.19). The model is checked with a more advanced 3D SCIA model afterwards. The 2D truss results in the following input loads:

Matrix results are stated below and pictured on the next page (Figure 5.20).

<table>
<thead>
<tr>
<th>Load Type</th>
<th>lower bar</th>
<th>upper bar</th>
<th>diagonal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression</td>
<td>1329 kN</td>
<td>130 kN</td>
<td>1681 kN</td>
</tr>
<tr>
<td>Tensile</td>
<td>64 kN</td>
<td>2135 kN</td>
<td>1681 kN</td>
</tr>
</tbody>
</table>
Figure 5.20
Expected resulting forces acting on the bars. Results obtained with MatrixFrame.

Figure 5.21
Expected deformation gives more insight in the acting normal (tension or compression) forces in the bars.
Material properties

The timber truss is designed with Kerto-S LVL bars. The material properties are briefly elaborated below, a more detailed calculation of the material properties is done with the timber floor structure. The characteristic material properties are transformed to design properties:

Material volume factor Kerto = s = 0,12

\[
k_n = \min \left( \frac{300}{h} \right)^s = \min \left( \frac{300}{490} \right)^{0.12} = 0.94
\]

\[
k_i = \min \left( \frac{3000}{l} \right)^{s/2} = \min \left( \frac{300}{23800} \right)^{0.12/2} = 0.88
\]

Furthermore:

\[k_{\text{mod}} = 0.8 \text{ (mid-term loading)}\]

\[k_{\text{def}} = 0.6 \text{ (climate class 1)}\]

\[Y_m = 1.2 \text{ (LVL)}\]

\[\psi_2 = 0.3 \text{ (office)}\]

Resulting in the following design material properties for the ULS:

\[E_d = \frac{E_{\text{mean}}}{\gamma_M} = \frac{13800}{1.2} = 11500 \text{ N/mm}^2\]

\[G_{\text{mean,fn}} = \frac{G_{\text{mean}}}{\gamma_M} = \frac{600}{1.2} = 500 \text{ N/mm}^2\]

The design values are calculated with the corresponding \(k_{\text{mod}}\) \(Y_m\) and \(k_i\) or \(k_n\):

\[f_{1,0,k} = 35.00 \text{ N/mm}^2\]

\[f_{1,0,d} = 20.60 \text{ N/mm}^2\]

\[f_{m,0,k} = 35.00 \text{ N/mm}^2\]

\[f_{m,0,d} = 27.65 \text{ N/mm}^2\]

\[f_{c,0,k} = 35.00 \text{ N/mm}^2\]

\[f_{c,0,d} = 23.33 \text{ N/mm}^2\]

\[f_{v,0,\text{edge},k} = 4.1 \text{ N/mm}^2\]

\[f_{v,0,\text{edge},d} = 2.733 \text{ N/mm}^2\]

\[f_{v,0,k} = 2.3 \text{ N/mm}^2\]

\[f_{v,0,\text{lat},d} = 1.53 \text{ N/mm}^2\]

\[f_{c,90,k} = 6 \text{ N/mm}^2\]

\[f_{c,90,d} = 4 \text{ N/mm}^2\]

\[f_{c,90,k} = 0.8 \text{ N/mm}^2\]

\[f_{c,90,d} = 0.53 \text{ N/mm}^2\]

Profile properties

Upper and lower bar, existing of two beams:

\[h = 490 \text{ mm}\]

\[b = 240 \text{ mm}\]

\[b_{\text{total}} = 490 \text{ mm}\]

\[A_1 = 117600 \text{ mm}^2\]

\[A_{\text{total}} = 235200 \text{ mm}^2\]

Total width = 830 mm

\[I_y = \frac{1}{12} \cdot b \cdot h^3 = \frac{1}{12} \cdot 480 \cdot 480^3 = 4.7 \cdot 10^9 \text{ mm}^4\]

\[E I_y = 5.42 \cdot 10^{13} \text{ N/mm}^2\]

The calculation of \(I_y\) is more complex since the section contains two bars with a empty element in the middle. The method is similar to the method used with the T beam timber floor and therefore only the results of the calculation is displayed here:

\[A_1 = 117600 \text{ mm}^2\]

\[A_2 = 117600 \text{ mm}^2\]

\[y_1 = 120 \text{ mm}\]

\[y_2 = 710 \text{ mm}\]

\[s_1 = 14112000\]

\[s_2 = 83496000\]

\[I_1 = 564480000 \text{ mm}^4\]

\[I_2 = 564480000 \text{ mm}^4\]

\[s_{\text{tot}} = 97608000\]

\[A_{\text{tot}} = 235200 \text{ mm}^2\]

\[Z_o = 415 \text{ mm}^2\]

\[Z_b = 415 \text{ mm}^2\]

\[I_z = 2.16 \cdot 10^{10} \text{ mm}^4\]

Diagonal:

\[h = 360 \text{ mm}\]

\[b = 340 \text{ mm}\]

\[A = 122400 \text{ mm}^2\]

\[I = 1.3 \cdot 10^8 \text{ mm}^4\]

\[E I = 1.52 \cdot 10^{13} \text{ mm}^4\]
Overview stresses

Lower bar:
\[
\sigma_{t,0,d} = \frac{F}{A} = \frac{64 \cdot 10^3}{235200} = 0.27 \text{ N/mm}^2
\]
\[
\sigma_{c,0,d} = \frac{F}{A} = \frac{1329 \cdot 10^3}{235200} = 5.65 \text{ N/mm}^2
\]

Upper bar (similar approach):
\[
\sigma_{t,0,d} = \frac{2135 \cdot 10^3}{490 \cdot 240} = 9.08 \text{ N/mm}^2
\]
\[
\sigma_{c,0,d} = \frac{127 \cdot 10^3}{490 \cdot 240} = 0.55 \text{ N/mm}^2
\]

Diagonal (similar approach):
\[
\sigma_{t,0,d} = \frac{1680 \cdot 10^3}{340 \cdot 360} = 13.73 \text{ N/mm}^2
\]
\[
\sigma_{c,0,d} = \frac{1680 \cdot 10^3}{340 \cdot 360} = 13.73 \text{ N/mm}^2
\]

Normal force checks

Lower bar:
Tensile and compressive load check:
\[
\frac{\sigma_{t,0,d}}{f_{t,0,d}} = \frac{0.27}{20.60} = 0.01 < 1.00
\]
\[
\frac{\sigma_{c,0,d}}{f_{c,0,d}} = \frac{5.65}{23.33} = 0.24 < 1.00
\]

Upper bar:
Tensile and compressive load check:
\[
\frac{\sigma_{t,0,d}}{f_{t,0,d}} = \frac{9.08}{20.60} = 0.44 < 1.00
\]
\[
\frac{\sigma_{c,0,d}}{f_{c,0,d}} = \frac{0.55}{23.33} = 0.02 < 1.00
\]

Diagonal:
Tensile and compressive load check:
\[
\frac{\sigma_{t,0,d}}{f_{t,0,d}} = \frac{13.73}{20.60} = 0.67 < 1.00
\]
\[
\frac{\sigma_{c,0,d}}{f_{c,0,d}} = \frac{13.73}{23.33} = 0.59 < 1.00
\]

Bending moment due to wind load

The bending moment \( M_z \) acting on the upper bar of the truss is checked. It is a conservative assumption that the complete bending moment is taken by the upper bar.

Furthermore \( M_y \) is assumed to be 0. Which is the case in the ideal situation. In the real situation \( M_y \) will not be zero, the 2D model used however cannot extract the \( M_y \) results and are therefore neglected. This is a safe assumption since the moment \( M_y \) will be very small compared to \( M_z \) and the normal forces.

\[
\sigma_{m,z,d} = \frac{M}{1\frac{1}{6}bh^2} = \frac{77 \cdot 10^3}{1\frac{1}{6} \cdot 830 \cdot 490^2} = 2.31 \text{ N/mm}^2
\]
\[
k_m = 0.7 \text{ (LVL)}
\]

The bending moment check becomes:
\[
\frac{\sigma_{y,z,d}}{f_{m,y,d}} + k_m \cdot \frac{\sigma_{m,z,d}}{f_{m,z,d}} < 1.00
\]
\[
k_m \cdot \frac{\sigma_{y,z,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} < 1.00
\]
\[
0 + \frac{2.31}{27.66} = 0.08 < 1.00
\]
\[
0 + 0.7 \cdot \frac{2.31}{27.66} = 0.06 < 1.00
\]

So the check fulfils with ease.
Combination of bending moment & normal forces

The checks are elaborated, assuming that the bending moment will act on the upper and lower bar, which is conservative again. The following equations are used for the tensile and compressive check:

\[
\frac{\sigma_{x,d}}{f_{x,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \cdot \frac{\sigma_{m,z,d}}{f_{m,z,d}} < 1,00
\]

\[
\frac{\sigma_{c,d}}{f_{c,d}} + k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} < 1,00
\]

And:

\[
\left(\frac{\sigma_{c,d}}{f_{c,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \cdot \frac{\sigma_{m,z,d}}{f_{m,z,d}} < 1,00
\]

\[
\left(\frac{\sigma_{c,d}}{f_{c,d}}\right)^2 + k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} < 1,00
\]

Upper bar checks:
0.44 + 0.06 = 0.50 < 1.00
0.44 + 0.08 = 0.52 < 1.00
0.02^2 + 0.06 = 0.06 < 1.00
0.02^2 + 0.08 = 0.08 < 1.00

Lower bar checks:
0.01 + 0.06 = 0.07 < 1.00
0.01 + 0.08 = 0.09 < 1.00
0.24^2 + 0.08 = 0.12 < 1.00
0.24^2 + 0.08 = 0.33 < 1.00

As expected the extra bending moment caused by the wind load has small impact on the strength of the trusses. This can be explained by the relative small surface of the wind load acting on one truss. When calculating the concrete cores the wind load will become a serious member.

Stability of the truss elements

Compression forces are present in the upper bar, lower bar and in certain diagonals. Therefore the buckling stability of all elements need to be checked. It is of great importance that the two elements of the lower and upper bars cooperate. Therefore they are connected on a frequent base.

Lower bar check:
\[ l_{\text{bc}} = 15,6 m \]
\[ E_{0.05} = 11600 \text{ N/m}^2 \]
\[ f_{c,0,k} = 35 \text{ N/mm}^2 \]
\[ i_z = \sqrt{\frac{I_z}{A}} = \sqrt{\frac{2,15 \cdot 10^6}{2,35 \cdot 10^5}} = 303,03 \]
\[ i_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{4,7 \cdot 10^9}{2,35 \cdot 10^5}} = 141,45 \]
\[ \lambda_z = \frac{l_{\text{bc}}}{i_z} = \frac{15600}{303,03} = 51,58 \]
\[ \lambda_y = \frac{l_{\text{bc}}}{i_y} = \frac{15600}{110,29} = 110,29 \]

Resulting in a relative lambda’s of:
\[ \lambda_{rel,z} = \frac{\lambda_z}{\pi \sqrt{E_{0.05}}} = \frac{51,58 \sqrt{\frac{35}{11600}}}{\pi} = 0,90 \]
\[ \lambda_{rel,y} = \frac{\lambda_y}{\pi \sqrt{E_{0.05}}} = \frac{110,29 \sqrt{\frac{35}{11600}}}{\pi} = 1,93 \]

The values are both larger than 0.3, resulting in the following stability checks:

\[ \frac{\sigma_{c,d}}{k_{c,y} \cdot f_{c,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \cdot \frac{\sigma_{m,z,d}}{f_{m,z,d}} < 1,00 \]
\[ \frac{\sigma_{c,d}}{k_{c,z} \cdot f_{c,d}} + k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} < 1,00 \]

For this equation some extra reduction factors need to be elaborated.
\[ \beta_c = 0.1 \quad \text{(LVL)} \]

\[ k_z = 0.5 \left( 1 + \beta_c \left( \lambda_{\text{rel},z} - 0.3 \right) + \lambda_{\text{rel},z}^2 \right) \]

\[ k_y = 0.5 \left( 1 + \beta_c \left( \lambda_{\text{rel},y} - 0.3 \right) + \lambda_{\text{rel},y}^2 \right) = 0.93 \]

\[ k_y = 0.5 \left( 1 + 0.1(0.90 - 0.3) + 0.90^2 \right) = 0.93 \]

\[ k_y = 0.5 \left( 1 + 0.1(0.129 - 0.3) + 0.129^2 \right) = 1.38 \]

\[ k_y = 0.5 \left( 1 + 0.1(2.76 - 0.3) + 2.76^2 \right) = 4.42 \]

\[ k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{\text{rel},y}^2}} = \frac{1}{4.42 + \sqrt{4.42^2 - 2.76^2}} = 0.13 \]

\[ k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{\text{rel},z}^2}} = \frac{1}{1.38 + \sqrt{1.38^2 - 0.93^2}} = 0.54 \]

The check can now be elaborated:

\[ \frac{0.55}{0.13 \cdot 23.3} + 0 + \frac{0.7}{27.66} = 0.24 < 1.00 \]

\[ \frac{0.55}{0.53 \cdot 23.3} + 0 + \frac{2.31}{27.66} = 0.13 < 1.00 \]

No buckling will occur in the upper bar of the truss. Furthermore the floor will prevent the bar from buckling.

**Upper bar check:**

\[ l_{\text{buc}} = 22.3 \, m \]

\[ E_{0.05} = 11600 \, N / \text{mm}^2 \]

\[ f_{\text{c.o.k}} = 35 \, N / \text{mm}^2 \]

\[ i_z = 303.03 \]

\[ i_y = 141.45 \]

\[ \lambda_z = \frac{l_{\text{buc}}}{i_z} = \frac{22300}{303.03} = 73.59 \]

\[ \lambda_y = \frac{l_{\text{buc}}}{i_y} = \frac{22300}{110.29} = 157.65 \]

\[ \lambda_{\text{rel},z} = \frac{\lambda_z}{E_{0.05}} = \frac{73.59}{11600} = 0.0063 \]

\[ \lambda_{\text{rel},y} = \frac{\lambda_y}{E_{0.05}} = \frac{157.65}{11600} = 0.0134 \]

\[ k_z = k_y = 0.5 \left( 1 + 0.1(0.73 - 0.3) + 0.73^2 \right) = 0.79 \]

\[ k_{c,y} = k_{c,z} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{\text{rel},y}^2}} = \frac{1}{0.79 + \sqrt{0.79^2 - 0.73^2}} = 0.92 \]
The check for the diagonal can now be elaborated:

\[
\frac{13.74}{0.92 \cdot 23.3} + 0 + 0.7 \cdot \frac{2.31}{27.66} = 0.69 < 1.00
\]

\[
\frac{13.74}{0.92 \cdot 23.3} + 0 + \frac{2.31}{27.66} = 0.72 < 1.00
\]

No buckling will occur in the diagonal bar of the truss.

**Deflection Truss 2.1**

The deflections of truss 2.1 can be calculated with the following equation, given by NEN-EN-1995-1-1:

\[
u_{\text{fin}} = u_{\text{fin,G}} + u_{\text{fin,Q1}} + u_{\text{fin,Q2}}
\]

In which the total deflection is a summation of the deflections caused by the permanent load, the dominant variable loads and the instantaneous variable loads. In these sub formulas the creep factor of the timber structure is taken into account.

\[
u_{\text{fin,G}} = u_{\text{inst,G}} \cdot (1 + \psi_{2.0,C} k_{\text{def}})
\]

\[
u_{\text{fin,Q1}} = u_{\text{inst,Q1}} \cdot (1 + \psi_{2.0,C} k_{\text{def}})
\]

\[
u_{\text{fin,Q2}} = u_{\text{inst,Q2}} \cdot (\psi_{2.0} + \psi_{2.0} k_{\text{def}})
\]

**SCIA model verification**

The loads acting on truss DD are split to give a better insight view on the deflection. The deflections are obtained by SCIA engineer and may differ slightly compared to the MatrixFrame model. The element load results of the SCIA are comparable with MatrixFrame with a maximal of 2167 kN in the upper bar and 1713 kN in the diagonal (Figure 5.22). The difference can be explained by the different way of modelling the own weight of the truss.

In the ULS of the entire loads give a maximal deflection of 37 mm in Matrixframe, and a maximal deflection of 33 mm in SCIA. The difference are again small, so the SCIA model is approved and to calculate the total deflection in the SLS.
The found deflections in SCIA in the SLS are based on a simplification of the load distribution. Since the loads are not well diverged in different types the following distinctions are made:

1. Own weight of the truss structure and floor structure.
2. Variable loads on the floor.
3. Instantaneous loads of the upper floors.

Resulting in the following immediate deflections (Figure 5.23):

\[
\begin{align*}
    u_{in,0} &= u_{inst,0} \cdot (1 + \psi_{2,0} k_{def}) \\
    u_{in,0} &= 8,2 \cdot (1 + 0,6) = 13,12 \text{ mm} \\
    u_{in,Q,1} &= u_{inst,Q,1} \cdot (1 + \psi_{2,Q} k_{def}) \\
    u_{in,Q,1} &= 3,4 \cdot (1 + 0,3 \cdot 0,6) = 4 \text{ mm} \\
    u_{in,Q,i} &= u_{inst,Q,1} \cdot (0,5 + \psi_{2,Q} k_{def}) \\
    u_{in,Q,i} &= 12,2 \cdot (0,7 + 0,3 \cdot 0,6) = 10,7 \text{ mm}
\end{align*}
\]

Note: In the SCIA model \( E_{\text{mean}} \) is used since the creep factor is already applied to the deflections.

Result in a total deflection of:
\[
    u_{fin} = 13,12 + 4 + 10,7 = 27,82 \text{ mm}
\]

A check with the maximal allowed deflection in the cantilever part:
\[
\begin{align*}
    w_{inst} &= 8,2 + 3,4 + 12,2 = 23,8 \text{ mm} \\
    w_{inst,max} &= \frac{2 \cdot l}{300} = \frac{16400}{300} = 54,66 \text{ mm} \\
    w_{net} &= 27,82 \text{ mm} \\
    w_{net,max} &= \frac{2 \cdot l}{250} = \frac{16400}{250} = 65,6 \text{ mm}
\end{align*}
\]

The deflection is rather small, this may be caused by the extreme dimensions of the timber trusses. The extreme dimensions are however required for the connections. Furthermore the maximal allowed deflection are relatively large in my opinion, therefore the result of 27,82 mm satisfies.

Figure 5.23
Deformed truss 2.1 with the maximal deformation of 27,82 mm in the lower left part.
Truss connection (timber-timber)

The connection K7 between the diagonal and the lower truss is calculated first with a timber-timber connection in which the loads are transferred by bolts only.

Forces acting on the node (Figure 5.24):

\[
\begin{align*}
F_{s15} & = 1246 \text{ kN} & f_{s15,h} & = -948,66 \\
F_{s14} & = 983 \text{ kN} & f_{s15,v} & = -1017,31 \\
F_{s7} & = 1330 \text{ kN} & f_{s14,h} & = -750,20 \\
F_{s8} & = 40 \text{ kN} & f_{s14,v} & = -804,49 \\
\end{align*}
\]

Angle \( s15 = 47 \) degrees
Angle \( s14 = 47 \) degrees

Forces acting on the lower beam:

\[
F_{\text{tot,low}} = F_{s7} + F_{s8} = 1330 + 40 = 1370 \text{ kN}
\]

Bolt properties

M20 type 8.8 \( d = 20 \text{ mm} \)
\( A = 244 \text{ mm}^2 \)

Strength bolt \( f_{u,k} = 660 \text{ N/mm}^2 \)

Characteristic yield moment bolt:

\[
M_{f,k,R} = 0,3f_{u,k} \cdot d^{2.6} = 0,3 \cdot 660 \cdot 20^{2.6} = 0,47 \cdot 10^6 \text{ Nmm}
\]

Tensile strength bolt:
\[
F_{Ax,Rd} = A \cdot f_{u,k} = 244 \cdot 660 = 161040 \text{ N}
\]

Tensile strength washer:

Diameter washer = 60 mm

Bearing capacity of the washer becomes:

\[
F_{2Ax,Rd} = 3 \cdot f_{c,90,k} \cdot \frac{\pi}{4} \left( d_w^2 - (d + 1)^2 \right) \\
F_{2Ax,Rd} = 3 \cdot 6 \cdot \frac{\pi}{4} (60^2 - (20 + 1)^2) = 44659 \text{ N}
\]

\[
F_{Ax,Rd} = 44659 \text{ N}
\]

The bearing capacity of the washer is governing and is taken into account for the failure scenarios.
Timber properties

Kerto S  
\( f_{c,90,k} = 6 \text{ N/mm}^2 \)
\( p_k = 480 \text{ kg/m}^3 \)

Embedment factor timber:

\[ k_{90} = 1,30 + 0,015 \cdot d = 1,30 + 0,015 \cdot 20 = 1,6 \]

Safety factors

| Y_m | connection | 1,2 |
| Y_m | material    | 1,2 |
| k_mod |           | 0,8 |

Compressive strain timber connection:

\[ f_{n,0,k} = 0,082 (1 - 0,01 \cdot d) \rho_k \]
\[ f_{n,0,k} = 0,082 (1 - 0,01 - 20) \cdot 480 = 31,68 \text{ N} / \text{mm}^2 \]

Since the angle of the lower bar is 0 degrees the compressive strain \( f_{h,0,k} \) is equal to the compressive strain in the lower bar. For the diagonal:

\[ f_{n,u,k} = \frac{f_{h,0,k}}{k_{90} \sin^2 \alpha + \cos^2 \alpha} \]
\[ f_{n,47,k} = \frac{31,49}{1,6 \cdot \sin^2 (47) + \cos^2 (47)} = 23,84 \text{ N} / \text{mm}^2 \]

Strength assessment

The connection is now tested with the Johanssen failure criteria (Figure 5.25). The lowest of the four scenarios is considered as the governing strength.

\[ \beta_{2,1} = \frac{f_{n,47,k}}{f_{h,0,k}} = \frac{23,84}{31,49} = 0,76 \]

Failure g

\[ F_{v,P_k,g} = f_{h,0,k} \cdot t_4 \cdot d \]
\[ F_{v,P_k,g} = 31,49 \cdot 240 \cdot 20 = 151142 \text{ N} \]

Failure h

\[ F_{v,P_k,h} = 0,5 \cdot f_{h,2,k} \cdot t_2 \cdot d \]
\[ F_{v,P_k,h} = 0,5 \cdot 23,84 \cdot 340 \cdot 20 = 81056 \text{ N} \]

Figure 5.25

Johansen failure criteria for a double shear timber-timber connection.

Failure j

\[ F_{v,P_k,j} = \frac{F_{A_v,P_k}}{4} + 1,05 \cdot \frac{f_{h,1,k} \cdot t_1 \cdot d}{2 + \beta} \left[ \frac{2 \beta (1 + \beta) \sqrt{2 \cdot 0,47 \cdot 10^6 \cdot 31,39 \cdot 20}}{2 + 0,76} \cdot \frac{4 \cdot 0,76 (2 + 0,76) \cdot 0,44 \cdot 10^6}{31,49 \cdot 340^2} \right] \]
\[ \beta = 0,76 \]
\[ F_{v,P_k,j} = 81140 \text{ N} \]

Failure k

\[ F_{v,P_k,k} = \frac{F_{A_v,P_k}}{4} + 1,15 \cdot \frac{2 \beta}{1 + \beta} \cdot \sqrt{2 \cdot 0,47 \cdot 10^6 \cdot 31,39 \cdot 20} \cdot f_{h,1,k} \cdot d \]
\[ F_{v,P_k,k} = 44659 \text{ N} + 36726 \text{ N} \]

Check failure j based on 25% Johansen load

\[ F_{v,P_k,j} = 1,25 \cdot \frac{f_{h,1,k} \cdot t_1 \cdot d}{2 + \beta} \]
\[ \beta = 0,76 \]
\[ F_{v,P_k,j} = 87469 \text{ N} \]
The minimal characteristic resistance of the connection is failure mode h: 83.4 kN.

\[ F_{V,R,d} = k_{mod} \frac{F_{V,R,k}}{\gamma_M} = 0.8 \cdot \frac{36.7}{1.2} = 24 \text{ kN} \]

Now the resistance of one bolt is known the effective amount of bolts is determined:

- Rows of bolts: 4
- Bolts per row (n): 5
- \( a_1: 100 \text{ mm} \)
- \( a_2: 80 \text{ mm} \)

\[ n_{ef} = \min \left\{ n, n^{0.9} \frac{a_1}{\sqrt{13d}} = 5^{0.9} \frac{100}{\sqrt{13 \cdot 20}} = 3.35 \right\} \]

The complete resistance of the connection therefore becomes:

\[ F_{V,ef,R,k} = n_{ef} \cdot F_{V,R,k} = 3.35 \cdot 4 \cdot 24 = 321 \text{ kN} \]

\[ F_V = 1391 \]

\[ \frac{1391}{321} = 4.33 > 1.00 \]

The connection does not fulfill by far. A larger surface is necessary to place more bolts, a surface which is not available. Another option is to apply steel plates in the diagonals.

**Splitting capacity of the timber (lower beam)**

Besides the strength problem which is solvable there is another, more complex problem with this timber-timber connection. Since the loads of the diagonal result in a horizontal and a vertical vector in the lower beam the lower beam starts to split due to the vertical vector. The splitting of the lower beam is checked, it is however to be expected that splitting is governing with these kind of loads. The vertical vector:

\[ F_{V,90,d} = \sin(47) \cdot 1245 = 910 \text{ kN} \]

The splitting capacity can be calculated according to the following rule:

\[ F_{90,Rd} = 14 \cdot bw \frac{h_o}{\sqrt{1 - \frac{h_o}{h}}} \]

\[ F_{90,Rd} = 14 \cdot 240 \cdot \frac{390}{\sqrt{1 - \frac{390}{490}}} = 146.9 \text{ kN} \]

Resulting in:

\[ \frac{F_{V,90}}{F_{90,Rd}} = \frac{910}{146.9} = 6.19 > 1.00 \]

The timber of the lower bar will start to split. The splitting capacity of wooden structures can be increased by adding steel rings, for this massive structure this is not to be advised since it would damage the lower bar.

Therefore an alternative connection is elaborated. Instead of a timber-timber connection a timber-steel-timber connection is introduced, making it possible to transfer only normal loads through the timber and no shear forces.
Truss connection (steel tube)

The previous connection did not satisfy. The major problem was the perpendicular forces working on the lower beam. A new connection with a steel tube connecting all bars is proposed. The steel element makes equilibrium and therefore only normal loads are working on the timber elements. The stresses in the tube itself will become very complicated, however this can be solved by choosing the right tube diameter and thickness.

The load distribution of node 7 is drawn in Figure 5.26 to give an overview of the detail. Another great advantage of this detail is the independence of the diagonal angle, making it very suitable for multiple connections.

\[
\begin{align*}
F_{s15} &= 1246 \text{ kN (tensile)} \\
F_{s14} &= 983 \text{ kN (compression)} \\
F_{s7} &= 1330 \text{ kN (compression)} \\
F_{s8} &= 40 \text{ kN (tensile)}
\end{align*}
\]

Angle \(s15 = 47\) degrees
Angle \(s14 = 47\) degrees

**Geometric properties**

<table>
<thead>
<tr>
<th>Member</th>
<th>(t)</th>
<th>Width (h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.240 mm</td>
<td>490 mm</td>
</tr>
<tr>
<td>2</td>
<td>340 mm</td>
<td>360 mm</td>
</tr>
</tbody>
</table>

The diagonal is increased to 360 mm since it is a more convenient size when applying bolts. With the minimal spacing requirements taken into account an extra row of bolts can be added.

**Figure 5.26**
Equilibrium of the node.
**Safety factors**

\[ Y_m \text{ connection} = 1.2 \]
\[ Y_m \text{ material} = 1.2 \]
\[ k_{\text{mod}} = 0.8 \]

**Bolt properties**

M20 type 8.8  
\[ d = 20 \text{ mm} \]
\[ A = 244 \text{ mm}^2 \]
Strength bolt  \[ f_{u,k} = 660 \text{ N/mm}^2 \]
Characterictic yield moment bolt:
\[ M_{dN} = 0.3 f_{u,k} \cdot d^{2.6} = 0.3 \cdot 660 \cdot 20^{2.6} = 0.47 \cdot 10^6 \text{ Nmm} \]

Tensile strength bolt:
\[ F_{Ax,Rd} = A \cdot f_{u,k} = 244 \cdot 660 = 161040 \text{ N} \]

Tensile strength washer:

Diameter washer = 60 mm (minimal 3d)

Bearing capacity of the washer becomes:
\[ F_{v,Rk} = \frac{\pi}{4} \left( d_w^2 - (d + 1)^2 \right) \]
\[ F_{2,Ax,Rd} = 3 \cdot \left( f_{c,90,k} \cdot \frac{\pi}{4} \left( d_w^2 - (d + 1)^2 \right) \right) \]
\[ F_{Ax,Rd} = 44659 \text{ N} \]

The bearing capacity of the washer is governing and is taken into account for the failure scenarios.

**Timber properties**

Kerto S  
\[ f_{c,90,k} = 6 \text{ N/mm}^2 \]
\[ p_{k} = 480 \text{ kg/m}^3 \]

Embedment factor timber:
\[ k_{90} = 1.30 + 0.015 \cdot d = 1.30 + 0.015 \cdot 20 = 1.6 \]

**Check S15 single steel plate**

First the diagonal S15 is checked with a single internal steel plate. For aesthetic reasons an internal steel plate is chosen, the amount of bolts needed to transfer the load from the timber beam to the steel plate is calculated, followed by a tensile load check on the internal steel plate.

Amount of plates: 1
Plate thickness: 12 mm
\[ f_{y} = 355 \text{ N/mm}^2 \]
\[ t_n = 340 \text{ mm} \]
\[ t_1 = 360/2-12 = 174 \text{ mm} \]
Load angle: 0 degrees

The failure mechanisms for timber-steel connections with two shear planes are researched. The smallest resistance is valid and can be multiplied since two shear planes are present.

\[ F_{v,Rk} = n_{df} \cdot F_{v,Rk} \]
\[ F_{v,Rk} = \min \left[ f_{h,t,k} \cdot t_1 \cdot d \left( 2 + \frac{4 M_{f,Rk}}{f_{h,t,k} d \cdot t_1^2} - 1 \right) + \frac{F_{Ax,Rd}}{4} \right] \]
\[ 2.3 \sqrt{2 M_{f,Rk} \cdot f_{h,t,k} d + \frac{F_{Ax,Rd}}{4}} \]

Failure f
\[ F_{v,Rk} = f_{h,t,k} \cdot t_1 \cdot d \]
\[ F_{v,Rk} = 3168 \cdot 174 \cdot 20 = 109578 \text{ N} \]

Failure g
\[ F_{v,Rk} = f_{h,t,k} \cdot t_1 \cdot d \left( 2 + \frac{4 M_{f,Rk}}{f_{h,t,k} d \cdot t_1^2} - 1 \right) + \frac{F_{Ax,Rd}}{4} \]
\[ 3148 \cdot 174 \cdot 20 \left( 2 + \frac{4 \cdot 0.47 \cdot 10^8}{3148 \cdot 20 \cdot 174^2} - 1 \right) + \frac{F_{Ax,Rd}}{4} \]
\[ F_{v,Rk} = 3148 \cdot 174 \cdot 20 \left[ 2 + \frac{4 \cdot 0.47 \cdot 10^8}{3148 \cdot 20 \cdot 174^2} - 1 \right] + \frac{44659}{4} = 60316 \text{ N} \]
The smallest failure mechanism is failure h+h.

\[ F_{v,Rk} = 2,3 \sqrt{2M_{y,Rk} \cdot f_{h,tk} \cdot d + \frac{F_{Ax,Rk}}{4}} \]

\[ F_{v,Rk} = 2,3 \sqrt{2 \cdot 0,47 \cdot 10^6 \cdot 31,48 \cdot 20 + \frac{44659}{4}} = 67118 \text{ N} \]

The resistance of 120 kN per bolt is governing. The effective amount of bolts is now calculated:

- Amount of bolts in a row: 8
- Amount of rows: 4
- Total number of bolts: 32
- a1 (minimal 3d): 120 mm
- a2 (minimal 3d): 70 mm
- a4 (minimal 3d): 75 mm

\[ n_{ef} = \min \left\{ n = 8 \right\} \sqrt[3]{\frac{a_1}{13d}} = 32 \sqrt[3]{\frac{120}{13 \cdot 20}} = 5,36 \]

The total resistance of the connection becomes:

\[ F_{v,ef,Rk} = F_{v,Rk} \cdot n_{ef} \cdot a = 120 \cdot 5,36 \cdot 4 = 2784 \text{ kN} \]

\[ F_{v,ef,d} = k_{mod} \cdot \frac{F_{v,ef,Rk}}{\gamma_M} = 0,8 \cdot \frac{2784}{1,2} = 1856 \text{ kN} \]

\[ \frac{F_{Ed}}{F_{v,ef,d}} = \frac{1245}{1856} = 0,67 < 1,0 \]

Therefore the timber-bolt-steel plate connection seems to fulfill. However the steel plate needs to be checked on its tensile strength.

Steel plate tensile check:

\[ \frac{N_{Ed}}{N_{t,Rd}} < 1,0 \]

\[ N_{t,Rd} = \frac{A \cdot f_y}{\gamma_M} = \frac{4320 \cdot 355}{1,00} = 1533 \text{ kN} \]

\[ \frac{1395}{1533} = 0,91 < 1,0 \]

The steel plate succeeds the check.

However the calculations gives no satisfying results. It is to be expected that the steel plate fails at the connection with the steel tube, or the steel tube will fail through punching shear. The surface is reduced in this area. It can be solved by locally increasing the steel plate, this is however very inconvenient. Furthermore the area of the bolts is not taken into account, resulting in a slightly smaller effective section and a smaller tensile strength.

Although the problems with the steel plate can be solved within the connection, the amount of bolts needed to transfer the load is large. This has aesthetic consequences and large assembly costs.

A new option, with two inner steel plates is proposed in member S15. It is to be expected that this will increase the connection resistance per bolt and therefore leads to less bolts. Furthermore the loads are better spread across loading to a smaller shear load on the steel tube.
Check S15 double steel plate

Amount of plates: 2
Plate thickness: 10 mm
\( f_y \): 355 N/mm²
\( t_1 \): 340 mm
\( t_2 \): 100 mm
\( t_3 \): 140 mm
Load angle: 0 degrees

The failure mechanisms for timber-steel connections with two shear planes are investigated (Figure 5.27). The smallest resistance is valid and can be checked, the mechanism is multiplied afterwards since four shear planes are present with the double steel plate.

Total resistance of the connection:

\[
F_{v,Rk} = \frac{n_{df} \cdot F_{v,Rk}}{F_{n,Rk}}
\]

\[
F_{v,Rk} = \min \left\{ f_{h,1k} \cdot t_1 \cdot d, 0.5 \cdot f_{h,1k} \cdot t_2 \cdot d \right. \]

\[
2.3 \sqrt{M_{y,Rk} \cdot f_{h,1k} \cdot d} + \frac{F_{Ax,Rk}}{4}
\]

A document provided by Metsäwood prescribes the calculation applied to Kerto-S5.6. The equations of the dowelled connection are similar except for certain constants, which are slightly more beneficial in the Metsäwood equations. The equations given by the Eurocode and Jorissen5.3 are used for the connection.

Failure 1
\[
F_{v,Rk} = f_{h,1k} \cdot t_1 \cdot d
\]
\[
F_{v,Rk} = 31.48 \cdot 100 \cdot 20 = 62976 \text{ N}
\]

Failure 2
\[
F_{v,Rk} = 0.5 \cdot f_{h,1k} \cdot t_2 \cdot d
\]
\[
F_{v,Rk} = 0.5 \cdot 31.48 \cdot 140 \cdot 20 = 44083 \text{ N}
\]

Failure 3
\[
F_{v,Rk} = f_{h,1k} \cdot t_1 \cdot d \left[ 2 + \frac{4M_{y,Rk}}{f_{h,1k} \cdot d \cdot t_1^2} - 1 \right] + \frac{F_{Ax,Rk}}{4}
\]
\[
F_{v,Rk} = 31.48 \cdot 100 \cdot 20 \left[ 2 + \frac{4 \cdot 0.47 \cdot 10^6}{31.48 \cdot 20 \cdot 100^2} - 1 \right] + \frac{F_{Ax,Rk}}{4}
\]
\[
\frac{44659}{4} = 43659 \text{ N}
\]

Failure 4
\[
F_{v,Rk} = 2.3 \sqrt{2M_{y,Rk} \cdot f_{h,1k} \cdot d} + \frac{F_{Ax,Rk}}{4}
\]
\[
F_{v,Rk} = 2.3 \sqrt{2 \cdot 0.47 \cdot 10^6 \cdot 31.48 \cdot 20} + \frac{44659}{4} = 67118 \text{ N}
\]
The smallest failure mechanism is failure:

\[
F_{Rd} = \min \begin{cases} F_1 + F_2 & 63 + 44 = 107 \text{ kN} \\ F_3 + F_4 & 63 + 67 = 130 \text{ kN} \\ F_2 + F_3 & 44 + 44 = 88 \text{ kN} \\ F_2 + F_4 & 44 + 67 = 111 \text{ kN} \\ F_3 + F_4 & 44 + 67 = 111 \text{ kN} \\ F_4 + F_4 & 67 + 67 = 134 \text{ kN} \end{cases}
\]

The resistance of 88 kN per bolt is governing. Since two steel plates are applied the resistance is doubled to 176 kN. The effective amount of bolts is now calculated:

Amount of bolts in a row: 6
Amount of rows: 3
Total number of bolts: 18
a1 (minimal 3d): 120 mm
a2 (minimal 3d): 90 mm
a3 (maximal 80mm): 150 mm
a4 (minimal 3d): 90 mm

The maximal value \(a_3\) of 80 mm seems peculiar since it is rather close to the edge of the timber element. Especially with the large diameter bolts used.

\[
n_{\text{ef}} = \min \left\{ \frac{n^{0.9}}{\sqrt{13d}} \left( \frac{a_1}{130} \right) = 6^{0.9} \sqrt{\frac{120}{13 \cdot 90}} = 4.23 \right. \\
n_{\text{ef, KERTO}} = n^{0.9} \sqrt{\frac{2t_1}{50 \cdot d^2}} = 6^{0.9} \sqrt{\frac{120 \cdot 140}{50 \cdot 90^2}} = 4.80
\]

The effective number of bolts is described in the Eurocode (upper equation) and by the Metsäwood documentation. The Eurocode is more conservative and chosen to equate the total connection resistance.

\[
F_{v,\text{ef},Rk} = F_{v,Rk} \cdot n_{\text{ef}} \cdot a = 176 \cdot 4.23 \cdot 3 = 2233 \text{ kN}
\]
\[
F_{v,\text{ef},d} = F_{v,\text{ef},Rk} \cdot k_{\text{mod}} \cdot \frac{F_{v,\text{ef},Rk}}{Y_M} = 0.8 \cdot \frac{2233}{1.2} = 1489 \text{ kN}
\]
\[
F_{\text{Ed}} = \frac{F_{v,\text{ef},d}}{1489} = 0.83 < 1.0
\]

The beam-bolts-steel connection seems to fulfil. The steel plate is checked again, similar to the example with one single plate. The new plates are thinner and smaller.

Steel plate thickness: 10 mm
Section height: 340 mm
Section surface: 3400 mm²
\(Y_{m1}\): 1,00
Steel: S355

\[
\frac{N_{\text{Ed}}}{N_{f,\text{Ed}}} < 1.0
\]
\[
N_{f,\text{Ed}} = \frac{A \cdot f_y}{\gamma_M} = \frac{3400 \cdot 355}{1.00} = 1171 \text{ kN}
\]
\[
\frac{1246}{1171 \cdot 2} = 0.53 < 1.0
\]

The two steel plates fulfil the test with ease and therefore the decision is made to design the connection with two steel plates.
S15 Block shear check (Metsäwood approach)

The Kerto documentation prescribes a block shear failure method which is far more extensive compared to the Eurocode equations. This method is used for calculating the block shear failure and schematized in Figure 5.28.

\[ F_{0,Ed} \leq F_{0,Rd} \]
\[ F_{0,Rd} = k_{\text{mod}} \cdot \frac{F_{0,Rk}}{\gamma_M} \]

The characteristic timber failure capacity of the joint area where:

\[ F_{0,Rk} = \sum_{i=1}^{m} F_{i,0,Rk} \]

And:

\[ F_{i,0,Rk} = F_{i,p,Rk} + F_{i,p,Rk} \]

Metsäwood considers two parts of the block shear failure. The inner lamellas part and the outer lamellas parts capacities. First the capacity of the middle lamellas is determined.

Figure 5.28 Definition of symbols for the inner parts of lamellas
Capacity of inner parts lamellas (side lamellas)

The capacity of the inner lamellas for a bar with tensile load:

\[
F_{i,0,k} = \min \left\{ \frac{A_{h,ip} \cdot f_{h,0,k}}{F_{tv,k}}, \frac{A_{f} \cdot f_{iR} \cdot k_{th,ph} \cdot k_{tv}}{F_{tk,tv,k}} \right\}
\]

\[
d = 20 \text{ mm}
\]
\[
n = 18
\]
\[
n_1 = 6
\]
\[
n_2 = 3
\]
\[
t_1 = 100 \text{ mm}
\]
\[
t_2 = 140 \text{ mm}
\]
\[
f_y = 660 \text{ N/mm}^2
\]
\[
f_{i,0,k} = 35 \text{ N/mm}^2 \text{ Kerto-S}
\]
\[
f_{v,\text{edge},k} = 4.1 \text{ N/mm}^2 \text{ Kerto-S}
\]
\[
f_{i,30,k} = 0.8 \text{ N/mm}^2 \text{ Kerto-S}
\]
\[
k_v = 0.7 \text{ Kerto-S}
\]

Makes:

\[
A_{h,ip} = (n - n_1) \cdot d \cdot t_i = (18 - 6) \cdot 20 \cdot 100 = 24000 \text{ mm}^2
\]

And:

\[
F_{cv,k} = F_{v,k} + (n_2 - 1) \cdot d \cdot t_{ef,j} \cdot f_{h,0,k}
\]

\[
F_{tv,k} = \begin{cases} 
F_{t,k} & \text{when } F_{t,k} \leq F_{v,k} \\
F_{v,k} \left( 1 - 0.3 \frac{F_{t,k}}{F_{v,k}} \right) & \text{when } F_{t,k} > F_{v,k}
\end{cases}
\]

With the following values for the side lamellas:

\[
t_{ef,j} = \min \left\{ \begin{array}{l}
t_i \\
0.68 \cdot d \cdot \frac{f_y}{f_{h,0,k}} \\
100 \text{ mm}
\end{array} \right\}
\]

\[
t_{ef,j} = \min \left\{ \begin{array}{l}
0.68 \cdot 20 \cdot \sqrt[1 / 3]{\frac{660}{31.48}} = 62 \text{ mm}
\end{array} \right\}
\]

\[
A_{x,ip} = 2 \cdot (n_2 - 1) \cdot \left( \frac{(n_1 - 1) \cdot a_i + a_g}{t_{ef,j}} \right)
\]

\[
A_{x,p} = (n_2 - 1) \cdot (a_2 - d) \cdot t_j
\]

\[
A_{x,ip} = 2 \cdot (3 - 1) \cdot \left( \frac{(6 - 1) \cdot 120 + 150}{62} \right) = 186000 \text{ mm}^2
\]

\[
A_{x,p} = (3 - 1) \cdot (90 - 20) \cdot 100 = 14000 \text{ mm}^2
\]

Now the resistances can be calculated:

\[
F_{t,k} = 1.7 \cdot n_1^{-0.1} \cdot A_{x,ip} \cdot f_{h,0,k}
\]

\[
F_{v,k} = k_v \cdot n_1^{-0.1} \cdot A_{x,ip} \cdot f_{v,k}
\]

\[
F_{t,k} = 1.7 \cdot 6^{-0.1} \cdot 14000 \cdot 31.48 = 696353 \text{ N}
\]

\[
F_{v,k} = 0.7 \cdot 6^{-0.1} \cdot 186000 \cdot 4.1 = 446251 \text{ N}
\]

And:

\[
F_{t,k} > F_{v,k}
\]

\[
F_{br,k} = 446251 \left( 1 - 0.3 \frac{446251}{696353} \right) = 360458 \text{ N}
\]

The governing strength of the inner part lamellas becomes:

\[
F_{i,0,k} = \min \left\{ \frac{A_{h,ip} \cdot f_{h,0,k}}{F_{tv,k}} \right\} = \min \left\{ \frac{24000 \cdot 31.48}{360.5} = 755.7 \text{ kN} \right\}
\]

Capacity of the edge part of lamellas (side lamellas)

Almost similar as calculating the inner part lamellas the edge part contributing to the resistance is calculated. Figure 5.29 prescribes the symbols used for this calculation. The total capacity of the edge part of lamellas (tension):

\[
F_{e,0,k} = \min \left\{ \frac{A_{h,ip} \cdot f_{h,0,k}}{F_{tv,k}} \right\} = \min \left\{ \frac{24000 \cdot 31.48}{360.5} = 755.7 \text{ kN} \right\}
\]
Where:

\[ A_{\text{rup}} = n_1 \cdot d \cdot t_1 = 6 \cdot 20 \cdot 100 = 12000 \text{ mm}^2 \]

And:

First \( F_{\text{se},k} \) is determined with help of the following equations:

\[
\begin{align*}
F_{\text{se},k} &= \left\{ \begin{array}{ll}
F_{s,k} \left(1 - 0,3 \cdot \frac{F_{s,k}}{F_{v,k}} \right) & \text{when } F_{s,k} \leq F_{v,k} \\
F_{v,k} \left(1 - 0,3 \cdot \frac{F_{v,k}}{F_{v,k}} \right) & \text{when } F_{v,k} < F_{s,k}
\end{array} \right.
\]

\[
F_{v,k} = \left\{ \begin{array}{ll}
F_{t,k} \left(1 - 0,3 \cdot \frac{F_{t,k}}{F_{v,k}} \right) & \text{when } F_{t,k} \leq F_{v,k} \\
F_{v,k} \left(1 - 0,3 \cdot \frac{F_{v,k}}{F_{t,k}} \right) & \text{when } F_{t,k} > F_{v,k}
\end{array} \right.
\]

\[
F_{\text{se},k} = \frac{14 \cdot n_1^{0.9}}{s_{\text{end}}} \cdot t_{\text{ef},2} \cdot (a_3 - 0,5d) \cdot f_{t,90,k}
\]

Resulting in the following splitting capacities:

\[
F_{\text{se},k} = \frac{14 \cdot n_1^{0.9}}{s_{\text{end}}} \cdot t_{\text{ef},1} \cdot (a_3 - 0,5d) \cdot f_{t,90,k} = 206820 \text{ N}
\]

\[
F_{\text{se},k} = \frac{14 \cdot 6^{0.9}}{2,6} \cdot 68,55 \cdot (150 - 0,5 \cdot 20) \cdot 0,8 = 206820 \text{ N}
\]

And:

\[
F_{s,k} = \frac{14 \cdot n_1^{0.9}}{s_{\text{hole}}} \cdot t_{\text{ef},1} \cdot (a_3 - 0,5d) \cdot f_{t,90,k}
\]

\[
F_{s,k} = \frac{14 \cdot 6^{0.9}}{1,08} \cdot 68,55 \cdot (150 - 0,5 \cdot 20) \cdot 0,8 = 497659 \text{ N}
\]
Furthermore:

\[
A_{v,\text{ep}} = k_{t,\text{ep}} \cdot A_{v,\text{ep}}
\]

\[
A_{v,\text{ep}} = A_{v,\text{ep}}
\]

Where:

\[
A_{v,\text{ep}} = (2a_t - d) \cdot t_i
\]

\[
A_{v,\text{ep}} = 2 \cdot ((n_i - 1) \cdot a_t + a_3) \cdot t_{\text{ef},1}
\]

\[
A_{v,\text{ep}} = (2 \cdot 90 - 20) \cdot 100 = 16000 \text{ mm}^2
\]

\[
A_{v,\text{ep}} = 2 \cdot ((6 - 1) \cdot 120 + 150) \cdot 68,55 = 102826 \text{ mm}^2
\]

This results in the following \( k_{t,\text{ep}} \) factor:

\[
k_{t,\text{ep}} = \frac{1}{1 + \frac{A_{v,\text{ep}}}{A_{v,\text{ep}}}} = \frac{1}{1 + \frac{16000}{102826}} = 0.87
\]

\[
A_{v,\text{ep}} = k_{t,\text{ep}} \cdot A_{v,\text{ep}} = 0.87 \cdot 16000 = 13845,6 \text{ mm}^2
\]

Now \( F_{i,k} \) and \( F_{v,k} \) can be calculated:

\[
F_{i,k} = 1.7 \cdot n_i^{-0.1} \cdot k_{t,\text{ep}} \cdot A_{v,\text{ep}} \cdot f_{h,0,k}
\]

\[
F_{v,k} = k_v \cdot n_i^{-0.1} \cdot A_{v,\text{ep}} \cdot f_{v,k}
\]

\[
F_{i,k} = 1.7 \cdot 6^{-0.1} \cdot 13846 \cdot 31,48 = 688673 \text{ N}
\]

\[
F_{v,k} = 0.7 \cdot 6^{-0.1} \cdot 102826 \cdot 4,1 = 246700 \text{ N}
\]

Resulting in:

\[
F_{i,k} > F_{v,k}
\]

\[
F_{v,k} = 246700 \left( 1 - 0.3 \cdot \frac{246700}{688673} \right) = 220188 \text{ N}
\]

and:

\[
F_{v,k} < F_{s,k}
\]

\[
F_{s,v,k} = F_{v,k} \left( 1 - 0.3 \cdot \frac{F_{v,k}}{F_{s,k}} \right)
\]

\[
F_{s,v,k} = 246700 \left( 1 - 0.3 \cdot \frac{246700}{497658} \right) = 210012 \text{ N}
\]

The minimal value:

\[
F_{e,p,k} = \min \left\{ \begin{array}{l}
A_{h,\text{ep}} \cdot f_{h,0,k} = 12000 \cdot 31,48 = 377,8 \text{ kN} \\
F_{t,v,k} = 220,1 \text{ kN} \\
F_{s,v,k} = 210,0 \text{ kN} \\
F_{s,e,k} = 206,8 \text{ kN}
\end{array} \right.
\]

Makes the minimal resistance 206,8 kN.

**Capacity of inner lamellas**

Now the shear block resistance of the two outer parts is calculated. However, in order to calculate the complete block shear resistance, the resistance of the middle part of the element (between the two steel plates) has to be calculated. This is done in a similar way as the outer lamellas. The elaboration of the inner part is therefore stated briefly:

\[
t_{\text{ef},2} = \min \left\{ \frac{120 \text{ mm}}{1.63 \cdot 20 \cdot \sqrt{\frac{660}{31,48}}} = 149 \text{ mm} \right. 
\]

Therefore the original width \( t_2 \) is used for the calculations.

\[
A_{v,\text{ep}} = 2 \cdot (3 - 1) \cdot ((6 - 1) \cdot 120 + 150) \cdot 120 = 360000 \text{ mm}^2
\]

\[
A_{v,\text{ep}} = (3 - 1) \cdot (90 - 20) \cdot 120 = 16800 \text{ mm}^2
\]

\[
F_{i,k} = 1.7 \cdot 6^{-0.1} \cdot 16800 \cdot 31,48 = 835624 \text{ N}
\]

\[
F_{v,k} = 0.7 \cdot 6^{-0.1} \cdot 360000 \cdot 4,1 = 863712 \text{ N}
\]

Resulting in:

\[
F_{i,k} < F_{v,k}
\]

\[
F_{v,k} = 835624 \left( 1 - 0.3 \cdot \frac{835624}{863712} \right) = 593090 \text{ N}
\]

\[
A_{h,\text{ep}} = (n - n_i) \cdot d \cdot t_i = (18 - 6) \cdot 20 \cdot 120 = 28800 \text{ mm}^2
\]

So:

\[
F_{i,0,p,k} = \min \left\{ \begin{array}{l}
A_{h,\text{ep}} \cdot f_{h,0,k} = 28800 \cdot 31,48 = 906,8 \text{ kN} \\
F_{t,v,k} = 593,0 \text{ kN}
\end{array} \right.
\]
The edge component of the middle lamellas can be calculated in a similar way:

\[ s_{\text{end}} = 2.6 \]
\[ s_{\text{hole}} = 1.08 \]

\[ F_{se,k} = \frac{14 \cdot 6^{0.9}}{2.6} \cdot 120 \cdot (150 - 0.5 \cdot 20) \cdot 0.8 = 362044 \, N \]

\[ F_{s,k} = \frac{14 \cdot 6^{0.9}}{1.08} \cdot 120 \cdot (150 - 0.5 \cdot 20) \cdot 0.8 = 871167 \, N \]

Furthermore:

\[ A_{t,ep} = (2 \cdot 90 - 20) \cdot 120 = 19200 \, mm^2 \]
\[ A_{v,ep} = 2 \cdot (6 - 1) \cdot 120 + 150 \cdot 120 = 180000 \, mm^2 \]

\[ k_{t,ep} = \frac{1}{1 + \frac{A_{t,ep}}{A_{v,ep}}} = 0.90 \]
\[ A_{t,ip} = k_{t,ep} \cdot A_{t,ep} = 0.90 \cdot 19200 = 17349.4 \, mm^2 \]

Now \( F_{v,k} \) and \( F_{t,k} \) can be calculated:

\[ F_{t,k} = 17 \cdot 6^{0.1} \cdot 17349.4 \cdot 31.48 = 862951 \, N \]
\[ F_{v,k} = 0.7 \cdot 6^{0.1} \cdot 180000 \cdot 4.1 = 431856 \, N \]

Resulting in:

\[ F_{t,k} > F_{v,k} \]

\[ F_{v,k} = 431856 \left(1 - 0.3 \cdot \frac{431856}{862951}\right) = 367020 \, N \]

And:

\[ F_{s,k} > F_{v,k} \]

\[ F_{sv,k} = 431856 \left(1 - 0.3 \cdot \frac{431856}{871166}\right) = 367632 \, N \]

\[ A_{n,ep} = n_1 \cdot d \cdot t_1 = 6 \cdot 20 \cdot 120 = 14400 \, mm^2 \]

The minimal value:

\[ F_{se,k} = \min \left\{ A_{n,ep} \cdot f_{v,0,0} = 14400 \cdot 31.48 = 453.4 \, kN \right. \]
\[ F_{tv,k} = 367,0 \, kN \]
\[ F_{sv,k} = 367,6 \, kN \]
\[ F_{se,k} = 362,0 \, kN \]

**Total block shear resistance**

Now all parts are known an easy summation can be made to calculate the total resistance of the connection:

\[ F_{t,0,0} = F_{tp,0} + F_{ep,0} \]
\[ F_{t,0,0} = F_{tp,0} + F_{ep,0} = 360,5 + 206,8 = 567,3 \, kN \]
\[ F_{2,0,0} = F_{tp,0} + F_{ep,0} = 593,1 + 362,0 = 955,1 \, kN \]
\[ F_{3,0,0} = F_{tp,0} + F_{ep,0} = 360,5 + 206,8 = 567,3 \, kN \]

The total resistance of the three parts (including edge and inner parts) therefore becomes:

\[ F_{tot,0} = \sum_{i=1}^{m} F_{i,0,0} = 567,3 + 955,1 + 567,3 = 2090 \, kN \]

Including the safety factors the design strength of the connection is:

\[ F_{tot,0,\gamma} = k_{mod} \cdot \frac{F_{tot,0}}{\gamma_{mod}} = 0.8 \cdot \frac{2090}{1.2} = 1393 \, kN \]

Check:

\[ \frac{F_{Ed}}{F_{tot,0,\gamma}} = \frac{1246}{1393} = 0.89 < 1.00 \]

So the block shear resistance of the connection is large enough to satisfy the check.
Figure 5.30
3D illustration of the tubular connection. Amount of bolts is indicative only.
Check S14 double steel plate

The diagonal S14 is checked in a similar way as S15. Checks are made regarding the bolt failure modes, the strength of the steel plate and the block shear failure. Since the methods which are used are equal to the methods of diagonal S15 only the results are stated below.

Load: \(983\) kN (compression)
Amount of plates: 2
Plate thickness: 10 mm
\(f_y\): 355 N/mm\(^2\)
\(t_h\): 340 mm
\(t_1\): 80 mm
\(t_2\): 160 mm
Load angle: 0 degrees

A similar connection is applied compared to beam S15 resulting in the following failure modes:

\[
F_{rel} = \min \left\{ \begin{array}{l}
F_1 + F_2 \\
F_1 + F_4 \\
F_2 + F_3 \\
F_2 + F_4 \\
F_3 + F_4 \\
F_4 + F_4
\end{array} \right\} = \min \left\{ \begin{array}{l}
50 + 50 = 100 \text{ kN} \\
50 + 65 = 115 \text{ kN} \\
50 + 84 = 134 \text{ kN} \\
50 + 65 = 115 \text{ kN} \\
84 + 65 = 149 \text{ kN} \\
65 + 65 = 130 \text{ kN}
\end{array} \right\}
\]

With a total resistance of 100 kN per bolt as governing factor. Multiplied by two plates makes 201,5 kN (round off difference). Three rows of six bolts each would result in an effective number of 4,23 bolts (equal to beam S15). Resulting in a total connection strength of:

\[
F_{v,ef,Rk} = F_{v,Rk} \cdot n_{ef} \cdot a = 201 \cdot 4,23 \cdot 3 = 2524 \text{ kN}
\]

\[
F_{v,ef,d} = k_{mod} \cdot \frac{F_{v,ef,Rk}}{\gamma_M} = 0,8 \cdot \frac{2524}{1,2} = 1682 \text{ kN}
\]

\[
\frac{F_{Ed}}{F_{v,ef,d}} = \frac{983}{1682} = 0,58 < 1,0
\]

The connection satisfies with ease.

Therefore it is more economical to decrease the amount of bolts. An alternative of three rows with five bolt each results in an affective amount of: 3,50

The check of the alternative leads to:

\[
F_{v,ef,Rk} = F_{v,Rk} \cdot n_{ef} \cdot a = 201 \cdot 3,50 \cdot 3 = 2142 \text{ kN}
\]

\[
F_{v,ef,d} = k_{mod} \cdot \frac{F_{v,ef,Rk}}{\gamma_M} = 0,8 \cdot \frac{2142}{1,2} = 1428 \text{ kN}
\]

\[
\frac{F_{Ed}}{F_{v,ef,d}} = \frac{983}{1428} = 0,69 < 1,0
\]

And satisfies the check. The inner steel plates are not checked since the loads are smaller compared to beam S15 and the steel plates are equal. It is assumed no buckling will appear in the plate.

S14 block shear check

The check of the block shear resistance is similar to the beam S15 however different values are now governing since the bar is loaded in compression instead of tension. In practise this means that the resistance is higher compared to the tensile resistance.

For the inner part and edge part (side lamellas):

\[
F_{p,Rk} = \min \left\{ \begin{array}{l}
A_{h,p} \cdot f_{h,0,k} \\
A_{h,p} \cdot f_{h,0,k}
\end{array} \right\} = \min \left\{ \begin{array}{l}
16000 \cdot 31,48 = 508,8 \text{ kN} \\
508,4 \text{ kN}
\end{array} \right\}
\]

\[
F_{q,Rk} = \min \left\{ \begin{array}{l}
A_{h,q} \cdot f_{h,0,k} \\
A_{h,q} \cdot f_{h,0,k}
\end{array} \right\} = \min \left\{ \begin{array}{l}
32000 \cdot 31,48 = 1007,6 \text{ kN} \\
1186,7 \text{ kN}
\end{array} \right\}
\]

Summarised with the middle lamellas and the safety factors the total block shear resistance of beam S14 becomes:

\[
\frac{F_{Ed}}{F_{tot,0,Rd}} = \frac{983}{2021} = 0,49 < 1,00
\]

As expected the connection satisfies the check with ease.
Check S7 double outer steel plate

The lower bars S7 and S8 have a slightly different connection compared to the diagonals. Since these elements are ‘hidden’ in the floor package the aesthetic argument is non-existing when designing this connection. Therefore a simple (and economic) outer steel plate connection is proposed. Each bar is connected with two plates to maintain the symmetric load distribution. This means that bar S7 is connected to the tube with a total of four plates.

Load acting on bar S7: 1330 kN
Since the bar exists of two elements the total load on a single element becomes: 665 kN

Thickness plate: 6 mm
t₂: 240 mm
h₁: 490 mm

Failure modes according to the Eurocode:

\[
F_{v,Rk} = \min \left\{ 0.5 \cdot f_{h,2,k} \cdot t_2 \cdot d, \frac{1,15 \sqrt{2} M_{v,Rk} \cdot f_{h,2,k} \cdot d + F_{Ax,Rk}}{4} \right\}
\]

\[
F_{v,Rk} = \min \left\{ 0.5 \cdot 3148 \cdot 240 \cdot 20 = 75571 N \right\}
\]

\[
h_1 = 1,15 \sqrt{2} \cdot 0.47 \cdot 10^9 \cdot 3148 \cdot 20 + \frac{44659}{4} = 39141 N
\]

It is assumed that only one shear panel is occurring for each steel plate since it is applied at the border of the wood. However there are two steel plates applied for the connection making the total resistance per bolt connector:

\[
F_{v,Rk} = \min \left\{ F_1 + F_2, F_2 + F_2 \right\} = 78 kN
\]

\[
F_{v,Rk} = \min \left\{ 39 + 39, 75 + 75 \right\} = 150 kN
\]

The effective number is bolts is determined with the following proposal:

Rows: 3
Bolts per row: 7
Total bolts: 21
a₁: 120 mm
a₂: 130 mm
a₃: 150 mm
a₄: 115 mm

Resulting in an effective number of:

\[
n_{ef} = \min \left\{ \frac{n}{0.9} \sqrt{a_1} = 7 \cdot \frac{120}{13 \cdot 20} = 4.75 \right\}
\]

And a total resistance of:

\[
F_{v,ef,Rk} = F_{v,Rk} \cdot n_{ef} \cdot a = 78 \cdot 4.75 \cdot 3 = 1111 kN
\]

\[
F_{v,ef,d} = k_{mod} \cdot \frac{F_{v,ef,Rk}}{\gamma_M} = 0.8 \cdot \frac{1111}{1.2} = 740 kN
\]

\[
F_{Ed} = \frac{665}{740} = 0.90 < 1.0
\]

The connection satisfies. The plate capacity is no problem at all due to the relatively small loads per plate.

Note: since the steel plate has a thickness of 6 mm each it is treated as a ‘thin’ plate.
**(Alternative) Block shear according to Metsäwood**

Metsäwood prescribes a method to calculate the block resistance and the plug shear resistance of Kerto products. The method used here is an alternative of the inner steel plate problems described with bar S14 and S15.

The characteristic block shear capacity of timber member:

$$F_{bt,k} = L_{net,t} \cdot t_1 \cdot k_{bt} \cdot f_{t,0,k} = 200 \cdot 240 \cdot 1,25 \cdot 35 = 2100 \text{ kN}$$

$$k_{bt} = 1,25 \text{ (LVL)}$$

$$L_{net,t} = (n_2 - 1) \cdot (a_2 - D) = (3 - 1) \cdot (120 - 20) = 200 \text{ mm}$$

$$t_1 = 240 \text{ mm}$$

$$f_{t,0,k} = 35 \text{ N/mm}^2$$

The characteristic plug shear capacity of a Kerto member can be calculated with the following equation:

$$F_{ps,k} = L_{net,t} \cdot \left( t_{ef} \cdot f_{t,0,k} + (a_3 + (n_1 - 1) \cdot a_1) \cdot f_{v,0,k} \right)$$

Where the effective section is determined with $R_k$, the characteristic load-carrying capacity per shear plane per fastener.

$$t_{ef} = \frac{R_k}{d \cdot f_{h,0,k}} = \frac{75571}{20 \cdot 31,48} = 41,38 \text{ mm}$$

Resulting in:

$$F_{ps,k} = 200 \cdot (41,38 \cdot 35 + (150 + (5 - 1) \cdot 120) \cdot 2,3)$$

$$= 579,49 \text{ kN}$$

Makes a total design resistance of:

$$F_{v,ref,d} = k_{mod} \cdot F_{v,ref,Rk} \cdot \gamma_M = 0,8 \cdot \frac{579}{1,2} = 386 \text{ kN}$$

$$F_{Ed,per\ plate} = \frac{0,5 \cdot 665}{386} = 0,86 < 1,0$$

So the check satisfies.

**Block shear according to Eurocode**

The Eurocode also prescribes a method to calculate the block and plug shear problem.

$$F_{bs,Rk} = \max \begin{cases} 1,5 \cdot A_{net,t} \cdot f_{t,0,k} \\ 0,7 \cdot A_{net,v} \cdot f_{v,k} \end{cases}$$

The ‘max’ indicates that the largest value is governing, which is rather peculiar.

$$t_{ef} = \frac{1,4 \cdot M_{v,Rk} \cdot f_{h,k}}{d} = 1,4 \cdot \frac{0,47 \cdot 10^6}{31,48 \cdot 20} = 38,25 \text{ mm}$$

Furthermore:

$$L_{net,t} = \sum t_{t,j} = 240 \text{ mm}$$

$$L_{net,v} = \sum t_{v,i} = 630 \text{ mm}$$

$$A_{net,t} = \frac{L_{net,t} \cdot t_1}{2} \cdot \left( L_{net,t} + 2 \cdot t_{ef} \right)$$

$$= \frac{630 \cdot 240}{2} = 151200 \text{ mm}$$

$$A_{net,v} = \frac{630 \cdot 240}{2} \cdot 38 \cdot (240 + 2 \cdot 38) = 99540 \text{ mm}$$

The characteristic strength becomes:

$$F_{bs,Rk} = \max \begin{cases} 1,5 \cdot 57600 \cdot 31,48 = 3024,0 \text{ kN} \\ 0,7 \cdot 99540 \cdot 4,1 = 286,0 \text{ kN} \end{cases}$$

Makes a total design resistance extreme when the max value is chosen. When the minimal value is taken the connection does not satisfy:

$$F_{v,ref,d} = k_{mod} \cdot F_{v,ref,Rk} \cdot \gamma_M = 0,8 \cdot \frac{286}{1,2} = 190 \text{ kN}$$

$$F_{Ed,per\ plate} = \frac{0,5 \cdot 665}{190} = 1,75 > 1,0$$

Since the Metsäwood approach is more extensive and clear this method is chosen.
Check S8 double outer steel plate

The lower bars of S8 are designed in a similar way as bar S7. The loads acting on this member are however much smaller compared to S7. Therefore less bolts are required to make the connection satisfy.

Load acting on bar S8: 40 kN
Since the bar exists of two elements the total load on a single element becomes: 20 kN

The failure modes are equal to S7.

The effective number of bolts is determined with the following proposal:

Rows: 3
Bolts per row: 3
Total bolts: 9
a1: 120 mm
a2: 130 mm
a3: 150 mm
a4: 115 mm

Resulting in an effective number of:

\[ n_{\text{ef}} = \min \left\{ n = 3, \frac{n^{0,94} \sqrt[3]{a}}{13d} = 3^{0,94} \sqrt[13]{rac{120}{20}} = 2,21 \right\} \]

And a total resistance of:

\[ F_{v,\text{ef,Rk}} = F_{v,Rk} \cdot n_{\text{ef}} \cdot a = 104 \cdot 2,21 \cdot 3 = 693 \text{ kN} \]
\[ F_{v,\text{ef,d}} = k_{\text{mod}} \cdot \frac{F_{v,\text{ef,Rk}}}{\gamma_M} = 0,8 \cdot \frac{693}{1,2} = 462 \text{ kN} \]
\[ \frac{F_{\text{Ed}}}{F_{v,\text{ef,d}}} = \frac{20}{462} = 0,04 < 1,0 \]

The connection satisfies with ease, less bolts may be used. However this is not the case, due to fluctuating loads a different load-case may cause much larger tensile or even compressional forces in this bar.

Block shear according to Metsäwood

Since the calculation is similar to bar S7 only the results are stated below, the Metsäwood approach is chosen:

\[ F_{\text{bt,k}} = L_{\text{net,t}} \cdot t_1 \cdot k_{\text{bt}} \cdot f_{\text{t,0,k}} = 200 \cdot 240 \cdot 1,25 \cdot 35 = 2100 \text{ kN} \]

\[ k_{\text{bt}} = 1,25 \text{ (LVL)} \]
\[ L_{\text{net,t}} = (n_2 - 1) \cdot (a_2 - D) = (3 - 1) \cdot (120 - 20) = 200 \text{ mm} \]
\[ t_1 = 240 \text{ mm} \]
\[ f_{\text{t,0,k}} = 35 \text{ N/mm}^2 \]

And:

\[ F_{p,s,k} = L_{\text{net,t}} \cdot (t_1 \cdot f_{\text{t,0,k}} + (a_3 + (n_1 - 1) \cdot a_1) \cdot f_{\text{t,0,k}}) \]
\[ F_{p,s,k} = 200 \cdot (41,38 \cdot 35 + (150 + (3 - 1) \cdot 120) \cdot 2,3) = 496 \text{ kN} \]

Resulting in a design resistance of:

\[ F_{v,\text{ef,d}} = k_{\text{mod}} \cdot \frac{F_{v,\text{ef,Rk}}}{\gamma_M} = 0,8 \cdot \frac{496}{1,2} = 312 \text{ kN} \]
\[ \frac{F_{\text{Ed,per plate}}}{F_{v,\text{ef,d}}} = 0,5 \cdot 40 \frac{312}{312} = 0,06 < 1,0 \]

So the connection satisfies with ease. When the Eurocode approach was chosen for the calculation of the plug shear resistance the check would have been fulfilled with a value of 0,7. Bar S8 is a dangerous bar since the difference of the point load has very large influence on the behaviour of the beam, it can transform from the small value up to 500 kN (which is the case in another load-case). Therefore it is not a safe statement to execute this bar with less bolts.
Bolts check S8

The steel connection can be subdivided in the steel plate, the steel bolts, the welding and the steel tube. The timber element checks are already elaborated. Now the bolts are checked on clench (stuiik failure).

\[ \beta_p = \frac{9d}{8d + 3t_p} = \frac{9 \cdot 20}{8 \cdot 20 + 3 \cdot 6} = 1.01 > 1.0 \]

\[ \beta_p = 1.0 \]

The failure can be checked with the following equation:

\[ F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_{ub} \cdot d \cdot t}{\gamma_{M2}} \]

With:

\[
\begin{align*}
2.8 \frac{e_d}{d_0} & - 1.7 = 2.8 \frac{90}{20 + 2} - 1.7 = 9.75 \\
1.5 \frac{d_0}{d_0} & - 1.7 = 1.5 \frac{90}{20 + 2} - 1.7 = 4.43
\end{align*}
\]

And:

\[
\begin{align*}
\alpha_b &= \min \left\{ \frac{f_{ub}}{f_u}, 1 \right\} = 1.0 \\
\alpha_d &= \min \left\{ \frac{e_d}{3d_0}, \frac{d_0}{3d_0}, \frac{1}{4} \right\} \\
\end{align*}
\]

Resulting in:

\[ F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_{ub} \cdot d \cdot t}{\gamma_{M2}} = \frac{2.5 \cdot 10 \cdot 800 \cdot 20 \cdot 6}{1.25} = 192 \text{ kN} \]

Resistance per bolt.

In order to calculate the resistance of the complete steel plate connection the shear resistance of the bolt is calculated with bolt type 8.8:

\[ F_{v,Rd} = \frac{\alpha_v \cdot f_{ub} \cdot A}{\gamma_{M2}} = \frac{0.6 \cdot 660 \cdot 244.8}{1.25} = 78.00 \text{ kN} \]

When loaded in tension:

Punching shear of the bolt through the steel plate:

\[ B_{p,Rd} = \frac{0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_{ub}}{\gamma_{M2}} = \frac{0.6 \cdot \pi \cdot 32 \cdot 6 \cdot 660}{1.25} = 191 \text{ kN} \]

The smallest resistance value per bolt is the shear force resistance of the bolt. The complete resistance of the connection can be calculated by multiplying the resistance with the amount of bolts.

\[ F_{v,Rd} = 78.7 \cdot 3 = 1638 \text{ kN} \]

\[ \frac{F_{Ed}}{F_{v,Rd}} = \frac{623}{1638} = 0.38 < 1.00 \]

The check satisfies: the bolts will not fail.
Steel plate check S8 & S15

The steel plate is checked on clench (stuik) failure in order to test if the thickness of 6 mm and 10 mm suffices. This check is elaborated for the connection S8 and S15.

\[
\alpha_c = \min \left( \frac{1}{f_{b}} \right) = 1,0 \\
\frac{e_t}{3 \cdot d_g} = 1,0 \\
\frac{s_t}{3 \cdot d_g - 1} = 1 \\
\frac{120}{3 \cdot (20 + 2) - 1} = 1,56
\]

Compressional strength steel plate S7:
\[
A_{net} = A - 3 \cdot d_g \cdot t_w = 6 \cdot 340 - 3 \cdot (20 + 2) \cdot 6 = 1644 \ mm^2 \\
N_{u,Rd} = \frac{0.9 \cdot A_{net} \cdot f_{u}}{\gamma_{M2}} = \frac{0.9 \cdot 1644 \cdot 470}{1,25} = 556 \ kN \\
N_{Ed} = \frac{1330 / 4}{556} = 0,60 < 1,0
\]

Tensile strength steel plate S15:
\[
A_{net} = A - 3 \cdot d_g \cdot t_w = 10 \cdot 340 - 3 \cdot (20 + 2) \cdot 10 = 2740 \ mm^2 \\
N_{u,Rd} = \frac{0.9 \cdot A_{net} \cdot f_{u}}{\gamma_{M2}} = \frac{0.9 \cdot 2740 \cdot 470}{1,25} = 927 \ kN \\
N_{Ed} = \frac{1246 / 2}{927} = 0,67 < 1,0
\]

Both plates satisfy on tensile strength.

Welding check S15

The steel plates are connected to the steel tube with a welded connection. It is assumed that only a normal load is acting on the welding. The connection of steel plate S15 is taken since the largest load are appearing in this steel plate. (623 kN each)

\[
M_{Ed} = 0 \ (assumption) \\
V_{Ed} = 0 \ (assumption) \\
F_{H,Ed} = 623 \ kN
\]

The welding size is determined with the following equation:
\[
\sigma_{\perp} = \tau_{\perp} = \frac{F_{Ed} \sqrt{D}}{4a\ell_{eff}} = \frac{t \cdot \ell_{eff} \cdot \sigma_x \cdot \sqrt{2}}{4a \ell_{eff}} = \frac{t}{2a \sqrt{2}} \sigma_x
\]

With:
\[
\sigma_x = \frac{F}{A} = \frac{623 \cdot 10^3}{340 \cdot 10} = 183 \ N / mm^2
\]

No problems occur.
A welding size is assumed on the basis of half the tube, furthermore half of the load is assumed to be transferred for each welding.

\[ a_{\text{min}} = \frac{F_{\text{Ed}}}{f_{w,u} \cdot \sum \ell_{\text{eff}}} = \frac{623 \cdot 10^3 \cdot 0.5}{360 \cdot 470} = 1.84 \text{ mm} \]

\[ \ell_{\text{eff}} \approx 470 \text{ mm} \quad (2\pi r) \]

\[ f_{w,u} = 360 \quad \text{(S235)} \]

The chosen \( a \) is 8 mm.

Now the check can be elaborated.

\[ \sigma_\perp = \tau_\perp = \frac{0.5 \cdot F_{H,\text{Ed}} \cdot \frac{1}{2} \sqrt{2}}{a \ell_{\text{eff}}} = \]

\[ \frac{0.5 \cdot 623 \cdot 10^3 \cdot \frac{1}{2} \sqrt{2}}{8 \cdot 470} = 58.6 \text{ N/} \text{mm}^2 \]

\[ \tau_\parallel = 0 \]

Resulting in:

\[ \sqrt{\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_\parallel^2)} = \sqrt{58.6^2 + 3(58.6^2 + 0)} = 117 \text{ N/} \text{mm}^2 \]

\[ \frac{f_u}{\beta_w \cdot \gamma_{M2}} = \frac{360}{0.8 \cdot 1.25} = 360 \text{ N/} \text{mm}^2 \]

check : \[ \frac{117.2}{360} = 0.32 < 1.00 \]

And:

\[ \sigma_\perp = 58.9 \text{ N/} \text{mm}^2 \]

\[ 58.9 \text{ N/} \text{mm}^2 < \frac{0.9 \cdot f_u}{\gamma_{M2}} = \frac{0.9 \cdot 360}{1.25} = 259 \text{ N/} \text{mm}^2 \]

Both check satisfy. Therefore the welding will not fail.
Besides the checks of the steel plates, bolts and timber elements the steel tube has to be checked as well. A relative large diameter is chosen to make the bar more stiff to avoid large deformations. It also helps to distribute the forces from plates to tube since more surface is available. First a 2D approach is elaborated.

Loads acting on the tube (Figure 5.31) result in the following results:

\[ M_{\text{max}} = 339 \, \text{kNm} \]
\[ V_{\text{Ed,max}} = 1290 \, \text{kN} \]
\[ N_{\text{max}} = 0 \, \text{kN} \]

**Figure 5.31**
2D load scheme. This simplification is doubtful since the diagonals load the tube under an angle which is not taken into account. However it gives a proper initial position of the calculation.
Chord face failure

Plastic failure of the chord cross-section. According to Eurocode 1993-1-8, 7.4.2 table 7.3.

$k_p$: 1 (for tensile loaded connections)
$f_{y0}$: 355
Beta: 2,125
$Y_{M5}$: 1
$b_1$: 340
d_0: 300 mm
t_0: 20 mm

$k_p$ check for compression loaded connections:

$$\sigma_{Ed} = 194$$
$$n_p = \frac{f_{y0}}{Y_{M5}} = \frac{355}{1,00} = 0,54$$
$$k_p = 1-0,3\cdot n_p\cdot (1+n_p)$$
$$k_p = 1-0,3\cdot 0,54\cdot (1+0,54) = 0,75 < 1,0$$

So:

$$N_{t,Rd} = k_p\cdot f_{y0}\cdot t_0\cdot (4+20\beta^2)$$
$$= 0,75\cdot 355\cdot 20^2\cdot (4+20\cdot 2,15^2)$$
$$= 10879 \text{ kN}$$

$$M_{sp,1,Rd} = 0,5\cdot b_1\cdot N_{t,Rd} = 0,5\cdot 340\cdot 1,1\cdot 10^7 = 1836 \text{ kNm}$$

Checks results in:

$$\frac{N_{Ed}}{N_{t,Rd}} = \frac{1246}{10879} = 0,12 < 1,00$$

$$\frac{M_{Ed}}{M_{t,Rd}} = \frac{0}{1836} = 0 < 1,00$$

The $N_{Ed}$ is the results of the normal loads acting on the tube since multiple steel plates are transferring loads through the tube.

Punching shear failure

Punching shear failure of a hollow section chord wall. Crack initiation leading to rupture of the brace members from the chord member.

$$\sigma_{max} \cdot t_i < \frac{2t_0\left(\frac{f_{y0}}{\sqrt[3]{3}}\right)}{Y_{M5}}$$

$$\sigma_{max} = \frac{700\cdot 10^3}{340 \cdot 10} = 194 \text{ N / mm}^2$$

$$\sigma_{max} \cdot t_i = \frac{700\cdot 10^3\cdot 10}{340 \cdot 10} = 1944 \text{ N / mm}$$

$$\frac{2\cdot 20\left(\frac{355}{\sqrt[3]{3}}\right)}{1,00} = \frac{8198}{1,00} = 8198 \text{ N / mm}$$

$$\frac{1944}{8198} = 0,23 < 1,00$$

Deformation

The stiffness of the element is governing so it seems. A maximal deflection of 3 mm is allowed in the tube.

$$w_{max} = \frac{l}{250} = \frac{840}{250} = 3,3 \text{ mm}$$

The 2D model of the profile fulfils only just with a deflection of 3 mm. The deflection is in horizontal direction since the tube is modelled in that way.
A 2D simplification does not give proper results, therefore a part of the connection is modelled in SCIA in 3D (Figure 5.33). This gives a better view on how the tube distributes the stresses and how it deforms.

**Figure 5.33**
3D SCIA model, showing the 3D deflection of the elements.
Modelling

The model is constructed similar as the designed detail, the timber elements are however left out to give a better overview of the results. Only the steel is modelled and the forces in the timber bars are translated to loads acting directly on the steel plates.

The point loads acting in the bars are translated to line loads with the following equations:

\[
q_{15} = \frac{F}{l} = \frac{1246 \cdot 10^3 / 2}{340} = 1832 \text{ kN/m}
\]

\[
q_{14} = \frac{F}{l} = \frac{983 \cdot 10^3 / 2}{340} = 1445 \text{ kN/m}
\]

\[
q_{7} = \frac{F}{l} = \frac{1330 \cdot 10^3 / 4}{450} = 739 \text{ kN/m}
\]

\[
q_{8} = \frac{F}{l} = \frac{40 \cdot 10^3 / 4}{450} = 22 \text{ kN/m}
\]

These line loads are applied on the SCIA model resulting in very precise stress results.

The steel plates are fixed to the steel tube, welding joints are not taken into account.

Possible errors

Since modelling complex structures always leads to results, not knowing right or wrong, it is very important to distinguish possible errors.

Supports

In reality the connection is not supported. Equilibrium in the connection prevents the connection from moving. However it is to be expected that a small deformation will occur. In the model it was required to design supports, therefore four stiff connections are added on both sides of the tube.

Stiffness plates

In the 3D deformation it is clear that the steel plates deform the most (up to 3 mm in x direction). In reality this will not occur like this since the plates are connected to timber elements preventing it from deforming. By making the steel plates infinite stiff creates a more realistic result for the stresses acting on the steel tube is created.
Validation stresses

The results of the model are evaluated to check if the behaviour of the tube seems understandable.

Plate stresses:
The stresses in plates seem to be correct in the outer planes. (Point 1 and 2 in Figure 5.34)

\[
\sigma_{x,SCIA}^{15,SCIA} = 123.3 \text{ N} / \text{mm}^2
\]

\[
\sigma_{x,SCIA}^{7,expected} = \frac{F}{A} = \frac{1330 \cdot 10^3}{450 \cdot 6} = 123.1 \text{ N} / \text{mm}^2
\]

\[
\sigma_{x,SCIA}^{8,SCIA} = 3.6 \text{ N} / \text{mm}^2
\]

\[
\sigma_{x,SCIA}^{9,expected} = \frac{F}{A} = \frac{40 \cdot 10^3}{450 \cdot 6} = 3.7 \text{ N} / \text{mm}^2
\]

The stress check for the diagonals is harder to elaborate since the axis are rotated by 47 degrees. (Point 3 in Figure 5.34).

\[
\sigma_{x,SCIA}^{315,SCIA} = 110 \text{ N} / \text{mm}^2
\]

\[
\sigma_{x,47,SCIA}^{7,SCIA} = \frac{110}{\cos(47)} = 161 \text{ N} / \text{mm}^2
\]

\[
\sigma_{x,47,expected}^{3,SCIA} = \frac{F}{A} = \frac{1246 \cdot 10^3}{340 \cdot 10} = 183.3 \text{ N} / \text{mm}^2
\]

So a small difference occurs. This can be declared due to the 3D deformation and rotation of the diagonal, with a decreasing of the stresses as result.
Another occurrence is the spreading of the stresses. The stress pattern of plane 1 in Figure 5.35 shows a very realistic spreading with an increase of stresses in the plate near the tube (since the height of the plate decreases) and with extreme stresses at the connection. The stress pattern explains the connection detail of the Eurocode:

In point 1 a small part of the plate is removed, with the stress model it becomes clear why: the stresses are too large in this (small) area.

When taking a closer look at the 3D stresses in Figure 5.35 the stress pattern seems logical. In point 1 the stresses caused by the diagonal plates is transferred to the tube resulting in a (positive) increase of the stresses in x direction. On the other side of the tube where the diagonal stresses are tensile stresses the stresses in x direction are negative. Again this was to be expected.
Maximal stresses

The maximal stresses in the tube are found in between the two plates of diagonal S14. The results are obtained from the 3D stress analysis in SCIA with the extreme values taken from the surface and a maximal total load case as showed in Figure 5.36.

\[ \sigma_x = 150 \, N / mm^2 \]
\[ \sigma_y = 508 \, N / mm^2 \]

The maximal shear stress is found at the supports on the side of the tube, this shear force cannot occur in reality since the supports do not exist. Furthermore it was curious that only very small shear stresses occurred on the tube. The shear stresses at the location between S14 planes:

\[ \tau_{xy} = 10 \, N / mm^2 \]
The stresses at the location are checked with the following equation for an elastic situation:

\[
\left(\frac{\sigma_{x,Ed}}{f_y / \gamma_{M0}}\right)^2 + \left(\frac{\sigma_{y,Ed}}{f_y / \gamma_{M0}}\right)^2 - \left(\frac{\sigma_{x,Ed}}{f_y / \gamma_{M0}}\right)\left(\frac{\sigma_{y,Ed}}{f_y / \gamma_{M0}}\right) + 3\left(\frac{\tau_{Ed}}{f_y / \gamma_{M0}}\right)^2 \leq 1
\]

Results for point 1 in:

\[
\frac{150}{355 / 1,00}^2 + \frac{508}{355 / 1,00}^2 - \left(\frac{150}{355 / 1,00}\right)\left(\frac{508}{355 / 1,00}\right) + 3\left(\frac{10}{355 / 1,00}\right)^2 = 1,62 > 1,00
\]

So the tube does not fulfil. This was to be expected when researching the value of sigma y, which is very large. An alternative is stiffening the tube in the middle, resulting in lower stresses. Another possibility is to choose a higher steel strength. With S460 the connection fulfils:

\[
\frac{150}{460 / 1,00}^2 + \frac{508}{460 / 1,00}^2 - \left(\frac{150}{460 / 1,00}\right)\left(\frac{508}{460 / 1,00}\right) + 3\left(\frac{10}{460 / 1,00}\right)^2 = 0,96 < 1,00
\]

It however to be expected that choosing a stronger steel class leads to a much more expansive connection. If this is the case it would be a better option to locally stiffen the tube, by increasing the thickness of the tube. If this is impossible (large production costs) the complete tube thickness can also be increased. A total thickness of 22 mm:

\[
\frac{125}{355 / 1,00}^2 + \frac{300}{355 / 1,00}^2 - \left(\frac{125}{355 / 1,00}\right)\left(\frac{300}{355 / 1,00}\right) + 3\left(\frac{10}{355 / 1,00}\right)^2 = 0,62 < 1,00
\]

**Deflection**

The deflection of the connection is hard to distinguish since the beam is modelled with supports which are not there in reality. However a conclusion can be given about the deformation of the middle of the tube compared to the edges. The deflection is calculated with the strengthened tube with a thickness of 22 mm, resulting in the following deformations in the middle of the tube:

Horizontal deflection (from left to right caused by deformation of the tube): 1,4 mm

Vertical deflection, caused by deformation of the tube: 0 mm.

Interesting to notice is that the vertical deflection \(u_z\) is 0 mm in the middle of the tube, visible by the green line in Figure 5.38. The bottom of the tube deforms in positive and negative direction, the center remains however undeformed in the vertical direction, this was not to be expected.
Connection detail

The connection is modelled in 3D to get an overview of the loads and practical issues. A 2D representation of the model is projected to show the final result (Figure 5.39).

Figure 5.39
Detail of the truss connection.
The dimensions as drawn are the calculated dimensions.
Scale 1:20
Total deflection truss

It is very complex to calculate the final deflection of truss 2.1. The deflection of the truss is described earlier on with already the creep factor included.

The total deflection of the timber truss (as calculated earlier): 27.8 mm.

The relative low deflection can be explained due to the extra bending in the part in between the two supports, pointing the truss upwards right before the cantilever part (system 1 in Figure 5.40). By adding a load in the cantilever the deflection of both parts will decrease (system 2 in Figure 5.40). However it is important to state that a larger relative load in the cantilever can cause extreme deflections. This creates a very heavy connection at the left connection. In this thesis this connection is not calculated, but it is a similar connection with tube as designed earlier.

![Figure 5.40](image)

**Deflection of the connections**

In reality the truss has an extra deflection, based on slip in the connection. The connections start to deform in the timber, it is very hard to conclude what the total deflection will be since not every connection deforms in the same direction. However it is safe to state that the truss will not fail since there is quite some margin (27.8 mm with an $u_{\text{max}}$ of 55.6 mm) and the deflection of the tube is almost 0 mm in z direction. It is unlikely that all connections deform in the same direction with a total of 28 mm.

In order to check the total deflection a non-linear plastic calculation of the truss and connection need to be calculated.
Floor-truss connection

The floor main T beam is connected to the lower bar of the timber trusses. The connection is based on contact compression resulting in a very straightforward connection. The steel plate is checked and the compression perpendicular to the grain on the timber truss is checked.

The connection is designed with the following reaction force: \( V_{\text{Ed}} = 195 \text{ kN} \)

**Z profile check**

Geometric properties:
- Depth: 640 mm
- Thickness: 20 mm
- Width A: 240 mm
- Height B: 510 mm
- Width c: 170 mm
- Steel: S355

Load applied to the 2D model:
\[
q_v = \frac{195 \times 10^3}{640} = 304 \text{ N/mm}
\]

Results in a bending moment in point 1 of:
\[
M_1 = M_2 = 195 \times 10^3 \times 170 = 33,2 \times 10^6 \text{ Nmm}
\]

Depending on the modelling (with rolling support or without, Figure 5.41) the bending moment in point can be reduced.

This bending moment needs to be transferred by the steel plate. The bending moment capacity (around) the weak axis for a 1 mm strip can be formulated as:
\[
M_{\text{pl}} = \frac{W_{\text{pl}} \cdot f_y}{\gamma_{M0}} = \frac{1}{6} \cdot 640 \times 20^2 \cdot 355}{1,00} = 15,1 \times 10^6 \text{ Nmm}
\]

The bending moment is much larger than the moment resistance of the section. Therefore the profile will fail.

**Reinforced profile**

A new profile is introduced with a locally reinforced upper plate (part A) with a new height of 12 mm (Figure 5.42 and 5.43). This method is used to be more efficient with the amount of steel.

Furthermore four plates are introduced to transfer the load from part C to part A. Resulting in a significantly lower the bending moment in parts B and C. The bending moment in point 1 is now non-existing while the moment in part 2 is still present, therefore plane A needs to be reinforced at certain parts.

Four plates are introduced to prevent the lower plate from deforming, furthermore it spreads the bending moment better over the upper plate. Using the q load of 304 N/mm resulting of a reaction force of 27 kN for the outer plates and 74 kN for the inner plates.

---

Figure 5.41
Two ways of modelling the Z profile in Matrixframe, with the left one as most realistic (and reduced bending moment in point 2).
Figure 5.42
Reinforced connection:
A = upper plate
B = vertical plate
C = lower plate

Figure 5.43
Reinforced connection:
Front view, a total of 4 plates transfer the loads to the upper plate A.
The bending moment is reduced since the arm is decreased. The required support length is determined:

\[ f_{c,90,\text{edge},k} = 6,0 \text{ N / mm}^2 \]
\[ f_{c,90,\text{edge,d}} = 0,8 \cdot \frac{6,0}{1,2} = 4,0 \text{ N / mm}^2 \]
\[ \sigma_{90} = \frac{F}{A} = 195 \cdot 10^3 \text{ N / mm}^2 \]
\[ x = 76 \text{ mm} \]

The large width of the beam is beneficial for the supports, resulting in a length of min. 76 mm. A support length of 160 mm is applied. (Profile of 170 mm). A shear check of the timber member is not elaborated.

The bending moment caused by the reaction force is calculated with MatrixFrame with a reduced bending moment of 4,9 kNm. The required reinforced steel height to transfer the bending moment can be calculated very straightforward:

\[ M_{Rd} = \frac{W_{pl} \cdot f_y}{\gamma_{M0}} = 5,0 \cdot 10^6 \text{ Nmm} \]
\[ M_{Ed} = 4,9 \cdot 10^6 \text{ Nmm} \]
\[ \frac{M_{Ed}}{M_{Rd}} = 0,98 < 1,00 \]

It is assumed that the bending moment is completely transferred by the reinforced piece (libje). Resulting in a height of 84 mm. This height is very problematic. This can be solved by applying more vertical plates to spread the bending moment.

Another way is to calculate more precise. This method is used. A 2D plate is modelled in SCIA resulting in much smaller profiles. Reasons why the SCIA model is much more beneficial is due to the possibility to design a line support (which is sort of the case in reality) and to activate the plate.

With a reinforced height of 30 mm (visible in the Figures right) the following stresses appear in the critical point 1 in Figure 5.44.

\[ \sigma_{x,\text{max}} = 28,3 \text{ N / mm}^2 \]
\[ \sigma_{y,\text{max}} = 108,8 \text{ N / mm}^2 \]
\[ \tau_{\text{max}} = -90 \text{ N / mm}^2 \]

An elastic check is elaborated:

\[ \left( \frac{\sigma_{x,Ed}}{f_y / \gamma_{M0}} \right)^2 + \left( \frac{\sigma_{y,Ed}}{f_y / \gamma_{M0}} \right)^2 - \left( \frac{\sigma_{x,Ed}}{f_y / \gamma_{M0}} \right) \left( \frac{\sigma_{y,Ed}}{f_y / \gamma_{M0}} \right) + 3 \left( \frac{\tau_{Ed}}{f_y / \gamma_{M0}} \right)^2 \leq 1 \]

Results for point 1 in:

\[ \left( \frac{28,3}{355 / 1,00} \right)^2 + \left( \frac{108,8}{355 / 1,00} \right)^2 - \left( \frac{28,3}{355 / 1,00} \right) \left( \frac{108,8}{355 / 1,00} \right) + 3 \left( \frac{-90}{355 / 1,00} \right)^2 = 0,25 < 1,00 \]

It will be even possible to the decrease the thickness of the plates, this seems realistic since a plate thickness of 12 mm is exceptional in this cases.

Plate C is checked on shear force since the plate has a reduced height.

Thickness part C: 12 mm

\[ V_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = 12 \cdot 640 \cdot \frac{355}{\sqrt{3}} = 1574 \text{ kN} \]
\[ \frac{V_{Ed}}{V_{pl,Rd}} = \frac{195}{1574} = 0,12 < 1,00 \]

The check satisfies with ease. No check is elaborated for the tensile strength of part B since it is safe to assume that the plate will not fail to tension.
Compression truss

The connection causes compression perpendicular to the grain on the timber truss. Vertical load acting on the truss is calculated with 195 kN resulting in the following stress, the width of support is 640 mm + 20 mm, taking the side-plates and spacing into account.

The reinforcement of the steel has a height of 30 mm. The upper plate - between reinforcement has a height of 12 mm.

The stresses on the timber bar can be calculated, assuming that the loads are uniformly distributed.

\[
\sigma_{c,90} = \frac{F}{A} = \frac{195 \cdot 10^3}{660 \cdot 240} = 1.23 \text{ N/mm}^2
\]

(Locally at the reinforcements the stresses are higher)

The capacity of the timber:

\[
f_{c,90,\text{edge,k}} = 6.0 \text{ N/mm}^2
\]

\[
f_{c,90,\text{edge,d}} = 0.8 \cdot \frac{6.0}{1.2} = 4.0 \text{ N/mm}^2
\]

\[
\frac{\sigma_{90}}{f_{c,90,\text{edge,d}}} = \frac{1.23}{4.0} = 0.31 < 1.00
\]

So the support width of 640 mm satisfies the check. However problems may occur in point 2 where sigma y is 20 N/mm², when these stresses are distributed over a length of 220 mm no problems occur so the 12 mm plate between the reinforced parts do play an important role in the distribution of the loads.
Detail 1.2 redefined

Truss lower chord (principle detail)
Scale: 1:6,25
Date: 13-01-16

Location in the building:

Reglit Channel glazing
Aluminium L profile 100x200
Reynobond aluminium connector
Insulation 150 mm
Insulation 55 mm
Reynobond aluminium system
Reynobond aluminium system
Foil water retaining
Insulation 55 mm
Insulation 150 mm
Foil vapour barrier
Convecto unit
Diagonal truss girder
Truss-truss connection

The truss-truss connection can be interpreted as a stacking of several connection details (Figure 5.45). The connection is not calculated since it is a similar connection as the tube connection calculated before. First there is the upper truss with the diagonal connected to the tube similar as seen before (connection 1.2). Furthermore the lower bar of the upper truss (1.3) is connected in a similar way to the tube. The vertical member of the truss has a different connection, it is designed to transfer compression loads by a steel ‘platform’. This plate is connected with two other plates to the tube. The plate is also connected to the timber member with three bolts for security, the load distribution is 100% taken by the horizontal plate.

The tube is connected to a steel column (1.4), this steel column is visible from the inside of the hofjes (Figure 5.1). No buckling check is made since it is safe to assume that it will not buckle due to the small buckling length. If buckling would be a problem the thickness of the tubular column can be increased without any problems.

The steel column is supported by a welded plate (2.1), which is connected to the lower truss. This truss has an equal connection as the upper truss, this connection is however rotated by 90 degrees (2.2-2.4). By rotating it the section of the connection becomes visible.

Note: no diagonal member is drawn in the lower bar since it would create a messy drawing. the upper truss however proves that the vertical and the diagonal member cannot intersect each other since they are placed in the same plane, therefore the vertical member stops relative far from the tube, with an increased plate length, and increased buckling problem of the plates as result. This is important to check in further developments. It can be solved with thicker plates or by complicated sawing measures.

A cut-trough needs to be made in the diagonal since the steel plate does intersect the diagonal member.
Figure 5.45
Sketch truss-truss connection, which can be seen as a stacking of connection details.
3D Detail
Figure 5.46
Modified 3D detail. Similar to the detail in the architectural thesis, now with the bolts in the diagonal truss member.
6
Street elements
South façade

The steel columns in the south façade (Figure 6.1) are loaded under bending due to the wind. Occasionally compression loads also act on the columns. This compression loads will be caused by the own weight of the façade and (locally) due to a floor. It is to be expected that the wind load has greater influence on the column compared to the compressional loads, since the compressional loads on the floors are mainly carried by the timber trusses. Resulting in columns acting as vertical beams. A check on strength and stability is done, but more important is the horizontal deflection of the façade caused by the wind load.
The column is next to grid line intersection E-11 (Figure 6.2) However the column is not located on this crossing. The concrete wall of the basement is located on grid line E and the building has a small cantilever (to visually disconnect it from the ground) to the south. So all columns (cores) are located 500 mm south of the grid.

The loads of the façade elements and the wind load are modelled as point loads in the connections between column and façade. The own weight of the façade results in an extra bending moment in the column due to the eccentricity of 200 mm.

**Permanent loads**

- Own weight column: 0,3 kN/m²
- Façade elements: Glass façade + mullions: 2,5 kN/m²
  A: façade per connection: 3,25 m²
- Load façade: 8,13 kN
- Lighting fixtures interior: One point load of: 0,5 kN
Variable loads

Wind load (extreme south wind):
\[ F_{w1} = c_s c_d c_r q_p(b) A_{ref} = 0,85 \cdot 0,8 \cdot 0,884 \cdot 3,25 = 1,95 \text{ kN} \]
\[ c_s c_d = 0,85 \]
\[ c_r(D) = 0,8 \]
\[ q_p(b) = 0,884 \text{ kN} / \text{m}^2 \]
\[ A_{ref} = 3,25 \text{ m}^2 \]

Total loads ULS
\[ 1,2 \cdot 0,5 = 0,6 \text{ kN} \]
\[ 1,2 \cdot 8,13 = 9,75 \text{ kN} \]
\[ 1,2 \cdot 0,3 = 0,36 \text{ kN} / \text{m}^2 \]
\[ 1,5 \cdot 1,95 = 2,93 \text{ kN} \]

Total loads SLS
\[ 1,0 \cdot 0,5 = 0,5 \text{ kN} \]
\[ 1,0 \cdot 8,13 = 8,13 \text{ kN} \]
\[ 1,0 \cdot 0,3 = 0,3 \text{ kN} / \text{m}^2 \]
\[ 1,0 \cdot 1,95 = 1,95 \text{ kN} \]

2D loads results

The column is modelled in 2D in matrixFrame as shown in Figure 6.3. Resulting in compression load \( N_{Ed} \), bending moment in one direction \( M_y \) and shear load \( V_{Ed} \). Since the column is not checked in a 3D situation the \( M_z \) acting on the column is unknown. This should be taken into account when checking the element.

\[ N_{Ed} = 32 \text{ kN} \]
\[ V_{Ed} = 5,5 \text{ kN} \]
\[ M_y = 16 \text{ kNm} \]

Chosen profile

Since the compression load is relative small and the deflection is expected to be governing a IPE 200 profile is chosen.

\[ W = \frac{M_{Ed}}{f_{yd}} = \frac{16 \cdot 10^6}{275} = 58 \cdot 10^3 \text{ mm}^3 \]

\[ W_{IPE200,pl} = 220,6 \cdot 10^3 \text{ mm}^3 \]

Section classification:

\[ b \frac{t_1}{5,6} = 17,8 < 33 \]

The section is a class 1 section so a plastic calculation is allowed.

Figure 6.3
Simplified mechanical scheme of column E11.
Chosen Profile IPE 200

\[
\begin{align*}
&h = 200 \text{ mm} & &\gamma_{m0} = 1,00 \\
&b = 100 \text{ mm} & &\gamma_{m1} = 1,00 \\
&l_y = 1943,0 \cdot 10^4 \text{ mm}^4 & &\gamma_{m2} = 1,25 \\
&l_z = 142,3 \cdot 10^4 \text{ mm}^4 \\
&i_y = 80,7 \text{ mm} \\
&i_z = 22,36 \text{ mm} \\
&A = 2848 \text{ mm}^2 \\
&W_y = 220 \cdot 10^3 \text{ mm}^3 \\
&W_z = 44,61 \cdot 10^3 \text{ mm}^3
\end{align*}
\]

Compression strength

\[
\frac{N_{Ed}}{N_{c,Rd}} \leq 1,0
\]

\[
N_{c,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{2848 \cdot 275}{100} = 783 \cdot 10^3 \text{ N}
\]

\[
\frac{N_{Ed}}{N_{c,Rd}} = \frac{32 \cdot 10^3}{783 \cdot 10^3} = 0,04 \leq 1,0
\]

Bending strength

\[
\frac{M_{Ed}}{M_{c,Rd}} \leq 1,0
\]

\[
M_{c,Rd} = \frac{W_y \cdot f_y}{\gamma_{M0}} = \frac{220 \cdot 10^3 \cdot 275}{100} = 60,5 \cdot 10^6 \text{ N}
\]

\[
\frac{M_{Ed}}{M_{c,Rd}} = \frac{16 \cdot 10^6}{60,5^6} = 0,26 \leq 1,0
\]

Shear strength

\[
\frac{V_{Ed}}{V_{c,Rd}} \leq 1,0
\]

Since a IPE200 is considered as a class 1 section:

\[
V_{c,Rd} = V_{pl,Rd} = \frac{A \cdot (f_y \cdot f_r)}{\gamma_{M0}}
\]

\[
A = \frac{A \cdot h}{b + h} = \frac{2048 \cdot 200}{200 + 100} = 1899 \text{ mm}^2
\]

\[
\frac{275}{\sqrt{3}} = 158,8
\]

\[
V_{c,Rd} = \frac{1899 \cdot 158,8}{100} = 301,56 \text{ kN}
\]

\[
\frac{V_{Ed}}{V_{c,Rd}} = \frac{5,5}{302} = 0,02 < 1,0
\]

 Compression, shear force and bending force combined

Since all forces act on the section at the same time the load combination is checked in a linear way. However it should be clear that it will not give any problems mentioning the low values calculated earlier.

\[
\frac{N_{Ed}}{N_{c,Rd}} + \frac{V_{Ed}}{V_{c,Rd}} + \frac{M_{x,Ed}}{M_{x,c,Rd}} + \frac{M_{z,Ed}}{M_{z,c,Rd}} < 1,0
\]

\[
0,04 + 0,02 + 0,26 + 0 = 0,32 < 1,0
\]

Buckling due to compression (strong axis)

Buckling can occur since the column is loaded in compression and bending. First the strong axis (perpendicular to the façade) is checked.

\[
N_{cr} = \frac{\pi^2 \cdot EI}{l_{buc}^2} = \frac{2 \cdot 1.05 \cdot 1943,0 \cdot 10^4}{\gamma_{M0}} = 402,7 \text{ kN}
\]

\[
\overline{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} \cdot \frac{l_{buc}}{i + \lambda_i} = \sqrt{\frac{2848 \cdot 275}{402,7 \cdot 10^3}} = 1,39
\]

\[
\Phi = 0,5 \cdot \left[ 1 + \alpha \left( \overline{\lambda} - 0,2 \right) + \overline{\lambda}^2 \right]
\]

With alpha = 0.21 for hot formed profiles in strong axis.

\[
\Phi = 0,5 \cdot \left[ 1 + 0,21 \cdot (1,39 - 0,2) + 1,39^2 \right] = 1,60
\]
\[ \chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = \frac{1}{1.60 + \sqrt{1.60^2 - 1.39^2}} = 0.42 \]

\[ N_{b, Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0.42 \cdot 2848.275}{100} = 329,38 \text{ kN} \]

\[ N_{Ed} = \frac{32}{329,38} \Rightarrow 0.097 < 1.0 \]

As expected no buckling will occur at the strong axis.

**Buckling due to compression (weak axis)**

Buckling in the weak axis is more likely.

\[ N_{cr} = \frac{\pi^2 \cdot EI}{L_{buc}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 142,3 \cdot 10^4}{1000^2} = 29.49 \text{ kN} \]

\[ \lambda = \frac{\sqrt{A \cdot f_y}}{N_{cr} \cdot \iota + \lambda_i} = \frac{2848.275}{\sqrt{29.49 \cdot 10^3}} = 5.15 \]

\[ \lambda = \sqrt{\frac{N_{Ed}}{N_{cr}}} = \sqrt{\frac{783}{29.49}} = 5.15 \]

With \( \alpha = 0.34 \) for hot formed profiles in weak axis.

\[ \Phi = 0.5 \cdot [1 + \alpha (\lambda - 0.2) + \lambda^2] \]

\[ \Phi = 0.5 \cdot [1 + 0.34 (5.15 - 0.2) + 5.15^2] = 14.62 \]

\[ \chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = \frac{1}{14.62 + \sqrt{14.62^2 - 5.15^2}} = 0.035 \]

\[ N_{b, Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0.035 \cdot 2848.275}{100} = 27.67 \text{ kN} \]

\[ N_{Ed} = \frac{32}{27.67} = 1.16 > 1.0 \]

The column does not meet the buckling criteria. Therefore the buckling length in the weak axis is decreased. By connecting the columns with horizontal mullions (carriers of the glazing) the buckling length decreases to 5 meters. (More connections are possible, but unnecessary).

Buckling in the weak axis with 5 meter buckling length:

\[ N_{cr} = \frac{\pi^2 \cdot EI}{L_{buc}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 142,3 \cdot 10^4}{2500^2} = 117.97 \text{ kN} \]

\[ \lambda = \frac{\sqrt{A \cdot f_y}}{N_{cr} \cdot \iota + \lambda_i} = \frac{2848.275}{\sqrt{117.97 \cdot 10^3}} = 2.58 \]

\[ \lambda = \sqrt{\frac{N_{Ed}}{N_{cr}}} = \sqrt{\frac{783}{117.97}} = 2.58 \]

With \( \alpha = 0.34 \) for hot formed profiles in weak axis.

\[ \Phi = 0.5 \cdot [1 + \alpha (\lambda - 0.2) + \lambda^2] \]

\[ \Phi = 0.5 \cdot [1 + 0.34 (2.58 - 0.2) + 2.58^2] = 4.22 \]

\[ \chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = \frac{1}{4.22 + \sqrt{4.22^2 - 2.58^2}} = 0.13 \]

\[ N_{b, Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0.13 \cdot 2848.275}{100} = 103.46 \text{ kN} \]

\[ N_{Ed} = \frac{32}{103.46} = 0.31 > 1.0 \]

The column suffices with the decreased buckling length.

**Torsion**

It may be assumed that a IPE profile is not sensitive for torsion. Therefore no check has to be elaborated.

**Lateral torsional buckling**

Since the column has the load scheme of a slender beam lateral torsional buckling can occur. However the column is connected to the façade in the compression zone of the column section, therefore lateral torsional cannot occur. A different loading case, with a wind suction instead of pressure make a different story since the façade will be connected in the tensile zone in that case. This load case will be elaborated further on.
Buckling: compression & bending

Since the horizontal load increases the instability of the column a combination check should be equated.

\[
\frac{N_{Ed}}{\chi_y \cdot N_{Rk}} + k_{yy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{M_{y,Rk}} + k_{yz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{z,Rk}} \leq 1
\]

\[
\frac{N_{Ed}}{\chi_z \cdot N_{Rk}} + k_{zy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{M_{y,Rk}} + k_{zz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{z,Rk}} \leq 1
\]

\(\chi_{LT} = 0\) since no lateral torsional buckling can occur with the chosen wind load and with \(M_{z,Ed}\) is unknown the total buckling check is equal to the check done before. Resulting in a column which fulfils the tests.

Horizontal deflection

According to the MatrixFrame model the horizontal deflection of the column in SLS is equal to 22 mm (Figure 6.4). The horizontal deflection is the governing factor, since a smaller profile: IPE 180 would result in a deflection of 32 mm. The deflection found is 22 mm and therefore fulfils the requirement:

\[
w_{max} = \frac{l}{300} = \frac{10000}{300} = 30 \text{ mm}
\]

Figure 6.4
Horizontal deflection
Case 1, SLS
A second load case is elaborated with the chosen profile IPE 200 since lateral torsional buckling may occur in this load case it has to be checked. The wind load is acting from the north instead of from the south in this case.

**Permanen loads**

Similar compared to case 1.

**Variable loads**

Wind load (extreme north wind resulting in wind suction at the south façade):

\[
F_{w1} = c_s c_d \cdot c_f \cdot q_p(b) \cdot A_{ref}
\]

\[
F_{w1} = 0,85 \cdot -0,5 \cdot 0,884 \cdot 3,25 = -1,22 \text{ kN}
\]

\[
c_s c_d = 0,85
\]

\[
c_f(D) = -0,5
\]

\[
q_p(b) = 0,884 \text{ kN/m²}
\]

\[
A_{ref} = 3,25 \text{ m²}
\]

**Total loads ULS**

\[
1,2 \cdot 0,5 = 0,6 \text{ kN}
\]

\[
1,2 \cdot 8,13 = 9,75 \text{ kN}
\]

\[
1,2 \cdot 0,3 = 0,36 \text{ kN/m}
\]

\[
1,5 \cdot -1,22 = -1,83 \text{ kN}
\]

**2D loads results**

The column is modelled in 2D in matrixFrame, resulting in compression load \( N_{Ed} \), bending moment in one direction \( M \), and shear load \( V_{Ed} \). Since the column is not checked in a 3D situation the \( M \), acting on the column is unknown. This should be taken into account when checking the element.

\[
N_{Ed} = 32 \text{ kN}
\]

\[
V_{Ed} = 4 \text{ kN}
\]

\[
M_y = 12 \text{ kNm}
\]

**Chosen Profile IPE 200**

\[
h = 200 \text{ mm}
\]

\[
b = 100 \text{ mm}
\]

\[
l_y = 1943,0 \cdot 10^4 \text{ mm}^4
\]

\[
l_z = 142,3 \cdot 10^4 \text{ mm}^4
\]

\[
i_y = 80,7 \text{ mm}
\]

\[
i_z = 22,36 \text{ mm}
\]

\[
A = 2848 \text{ mm}^2
\]

\[
W_y = 220 \cdot 10^3 \text{ mm}^3
\]

\[
W_z = 44,61 \cdot 10^3 \text{ mm}^3
\]

**Compression strength**

\[
\frac{N_{Ed}}{N_{c,Rd}} \leq 1,0
\]

\[
N_{c,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{2848 \cdot 275}{100} = 783 \cdot 10^3 \text{ N}
\]

\[
\frac{N_{Ed}}{N_{c,Rd}} = \frac{32 \cdot 10^3}{783 \cdot 10^3} = 0,04 \leq 1,0
\]

**Bending strength**

\[
\frac{M_{Ed}}{M_{c,Rd}} \leq 1,0
\]

\[
M_{c,Rd} = \frac{W_y \cdot f_y}{\gamma_{M0}} = \frac{220 \cdot 10^3 \cdot 275}{100} = 60,5 \cdot 10^6 \text{ N}
\]

\[
\frac{M_{Ed}}{M_{c,Rd}} = \frac{12 \cdot 10^6}{60,5 \cdot 10^6} = 0,20 \leq 1,0
\]

**Shear strength**

\[
\frac{V_{Ed}}{V_{c,Rd}} \leq 1,0
\]

Since a IPE200 is considered as a class 1 section:

\[
V_{c,Rd} = V_{pl,Rd} = \frac{A_y \cdot (f_y \cdot \sqrt{3})}{\gamma_{M0}}
\]
Compression, shear force and bending force combined

Since all forces act on the section at the same time the load combination is checked in a linear way. However it should be clear that it will not give any problems mentioning the low values calculated earlier.

\[
\frac{N_{Ed}}{N_{c,Rd}} + \frac{V_{Ed}}{V_{c,Rd}} + \frac{M_{y,Ed}}{M_{y,c,Rd}} + \frac{M_{z,Ed}}{M_{z,c,Rd}} < 1,0
\]

0,04 + 0,01 + 0,20 + 0 = 0,25 < 1,0

Buckling due to compression (strong axis)

Buckling can occur since the column is loaded in compression and bending. First the strong axis (perpendicular to the façade) is checked.

\[
N_{cr} = \frac{\pi^2 \cdot EI}{l_{puc}^2} = \frac{\pi^2 \cdot 2,1 \cdot 10^5 \cdot 1943,0 \cdot 10^4}{10000^2} = 402,7 \text{ kN}
\]

\[
\lambda = \frac{A \cdot f_y}{\sqrt{N_{cr}}} = \frac{l_{puc}}{i + \lambda_i} = \sqrt{2848 \cdot 275} \cdot 402,7 \cdot 10^3 = 1,39
\]

\[
\lambda = \sqrt{\frac{N_{pl,i}}{N_{cr}}} = \frac{783}{402,7} = 1,93
\]

With alpha = 0,21 for hot formed profiles in strong axis.

\[
\Phi = 0.5 \cdot \left[ 1 + \alpha (\lambda - 0.2) + \lambda^2 \right]
\]

\[
\Phi = 0.5 \cdot \left[ 1 + 0.21(1,93 - 0.2) + 1,93^2 \right] = 1,60
\]

\[
\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = \frac{1}{1,60 + \sqrt{1,60^2 - 1,39^2}} = 0,42
\]

\[
N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0,42 \cdot 2848 \cdot 275}{100} = 329,38 \text{ kN}
\]

\[
N_{Ed} = \frac{32}{329,38} \Rightarrow 0,097 < 1,0
\]

As expect no buckling will occur at the strong axis.

Buckling due to compression (weak axis)

Buckling in the weak axis with 5 meter buckling length:

\[
N_{cr} = \frac{\pi^2 \cdot EI}{I_{puc}^2} = \frac{\pi^2 \cdot 2,1 \cdot 10^5 \cdot 142,3 \cdot 10^4}{5000^2} = 117,97 \text{ kN}
\]

\[
\lambda = \frac{A \cdot f_y}{\sqrt{N_{cr}}} = \frac{l_{puc}}{i + \lambda_i} = \sqrt{2848 \cdot 275} \cdot 117,97 \cdot 10^3 = 2,58
\]

\[
\lambda = \sqrt{\frac{N_{pl,i}}{N_{cr}}} = \frac{783}{117,97} = 2,58
\]

With alpha = 0,34 for hot formed profiles in weak axis.

\[
\Phi = 0.5 \cdot \left[ 1 + \alpha (\lambda - 0.2) + \lambda^2 \right]
\]

\[
\Phi = 0.5 \cdot \left[ 1 + 0,34(2,58 - 0,2) + 2,58^2 \right] = 4,22
\]

\[
\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = \frac{1}{4,22 + \sqrt{4,22^2 - 2,58^2}} = 0,13
\]

\[
N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0,13 \cdot 2848 \cdot 275}{100} = 103,46 \text{ kN}
\]

\[
N_{Ed} = \frac{32}{103,46} \Rightarrow 0,31 > 1,0
\]

The column suffices with the decreased buckling length.

Torsion

It may be assumed that a IPE profile is not sensitive for torsion. Therefore no check has to be made.
Lateral torsional buckling

The wind load causes suction on the façade, as a result of this the façade on the tensile zone of the column and lateral torsional buckling can occur. This has influence on the total buckling case. A check is done to research if lateral torsional protection needs to be applied.

\[ M_{b,rd} = \chi_{LT} \cdot W_y \cdot f_y / \gamma_{M1} \]

\[ W_y = W_{pl,y} = 200 \cdot 10^3 \text{ mm}^3 \]

\[ f_y = 275 \]

\[ \gamma_{M1} = 1,00 \]

The lateral torsional reduction factor can be calculated with the help of the following equations:

\[ \chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \lambda_{LT}^2}} \]

\[ \Phi_{LT} = 0,5 \left[ 1 + \alpha_{LT} \left( \lambda_{LT} - 0,2 \right) + 0,5 \lambda_{LT}^2 \right] \]

\[ \frac{h}{b} = \frac{200}{100} = 2 \rightarrow \alpha_{LT} = 0,21 \]

\[ \lambda_{LT} = \sqrt{\frac{W_y \cdot f_y}{M_{LT}}} \]

\[ M_{cr} \] can be elaborated according to the Dutch Annex.

\[ M_{cr} = k_{red} \cdot \frac{C}{L_g} \cdot \sqrt{E \cdot I_x \cdot G \cdot I_i} \]

\[ C = \frac{\pi \cdot C_1 \cdot L_g}{L_{kip}} \left( \sqrt{1 + \left( \frac{\pi^2 \cdot S_i^2}{L_{kip}^2} \cdot (C_2 + 1) \right)} + \frac{\pi \cdot C_2 \cdot S_i}{L_{kip}} \right) \]

\[ C = \frac{\pi \cdot 113 \cdot 10000}{5000} \]

\[ \left( \sqrt{1 + \left( \frac{\pi^2 \cdot 734^2}{5000^2} \cdot (0,45 + 1) \right)} + \frac{\pi \cdot 0,45 \cdot 734}{5000} \right) = 9,43 \]

\[ C_1 = 1,13 \]

\[ C_2 = 0,45 \]

\[ L_g = 10000 \]

\[ L_{kip} = 5000 \]

\[ S = 734 \]

Now C is known it is possible to equate \( M_{cr} \):

\[ M_{cr} = k_{red} \cdot C \cdot \frac{\sqrt{E \cdot I_x \cdot G \cdot I_i}}{L_g} \]

\[ M_{cr} = 1 \cdot \frac{9,43}{10000} \cdot \sqrt{2,1 \cdot 10^5 \cdot 142,3 \cdot 10^4 \cdot 8,1 \cdot 10^4 \cdot 6,848 \cdot 10^4} \]

\[ M_{cr} = 38,39 \text{ kNm} \]

With:

\[ k_{red} = 1 \]

\[ C = 9,43 \]

\[ L_g = 10000 \]

\[ E = 210000 \]

\[ G = 81000 \]

\[ I_x = 142,3 \cdot 10^4 \text{ mm}^4 \]

\[ I_i = 6,848 \cdot 10^4 \text{ mm}^4 \]

This results into a relative lateral torsional buckling slenderness of:

\[ \lambda_{LT} = \sqrt{\frac{W_y \cdot f_y}{M_{cr}}} = \sqrt{\frac{200 \cdot 10^3 \cdot 275}{38,39 \cdot 10^6}} = 1,25 \]

\[ 181 \]
Now all factors are known to equate the lateral torsional buckling check.

\[
\frac{M_{b,\text{Ed}}}{M_{b,\text{Rd}}} < 10
\]

\[
\frac{12}{30.01} = 0.40 < 1.0
\]

So the column fulfils the lateral torsional buckling test.

**Buckling: compression & bending**

Since the horizontal load increases the instability of the column a combination check should be equated.

\[
\frac{N_{\text{Ed}}}{\chi_y N_{Rk}} + k_{yy} \frac{M_{y,\text{Ed}} + \Delta M_{y,\text{Ed}}}{\chi_y M_{y,Rk} \gamma_{M1}} + k_{yz} \frac{M_{z,\text{Ed}} + \Delta M_{z,\text{Ed}}}{\chi_y M_{z,Rk} \gamma_{M1}} \leq 1
\]

\[
\frac{N_{\text{Ed}}}{\chi_x N_{Rk}} + k_{yy} \frac{M_{y,\text{Ed}} + \Delta M_{y,\text{Ed}}}{\chi_x M_{y,Rk} \gamma_{M1}} + k_{yz} \frac{M_{z,\text{Ed}} + \Delta M_{z,\text{Ed}}}{\chi_x M_{z,Rk} \gamma_{M1}} \leq 1
\]

\[
\chi_{LT} \neq 0 \quad \text{so the lateral torsional buckling has influence on the total buckling problem. Therefore this combination check has to be elaborated.}
\]

\[
C_{my} = 0.95 + 0.05 \cdot \alpha_n = 0.95 + 0.05 \cdot \frac{1}{12} = 0.96
\]

\[
\alpha_n = \frac{M_n}{M_s} = \frac{1}{12}
\]

\[
k_{yy} = C_{my} \left(1 + \frac{\Delta y - 0.2}{\chi_y N_{Rk}} \frac{N_{\text{Ed}}}{\gamma_{M1}} \right)
\]

\[
k_{yy} = 0.96 \left(1 + \frac{1.39 - 0.2}{\chi_y N_{Rk}} \frac{32 \cdot 10^3}{0.42 \cdot 783.3 \cdot 10^3} + 0.42 \frac{32 \cdot 10^3}{0.42 \cdot 783.3 \cdot 10^3} \right) = 1.07
\]

Since \(M_y\) is unknown and taken zero the first out of two equations is governing.

With:

\[
N_{\text{Ed}} = 32 \cdot 10^3 \quad N
\]

\[
N_{Rk} = 783.3 \cdot 10^3 \quad N
\]

\[
\chi_y = 0.42
\]

\[
k_{yy} = 1.07
\]

\[
M_{y,\text{Ed}} = 12 \cdot 10^6 \quad Nmm
\]

\[
\Delta M_{y,\text{Ed}} = 0
\]

\[
\chi_{LT} = 0.49
\]

\[
M_{y,Rk} = 60.5 \cdot 10^6 \quad Nmm
\]

So the column easily fulfils the buckling requirement.
In the second equation the $k_{zy}$ may be assumed to be zero for IPE beams according to the Dutch Annex, resulting in the following check:

\[
\frac{N_{Ed}}{\chi_{L,T} \cdot N_{Rk}} + k_{zy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{L,T} \cdot M_{y,Rk}} + k_{zz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{z,Rk}} \leq 1
\]

$k_{zy} = 0$

\[
\frac{N_{Ed}}{\chi_{L,T} \cdot N_{Rk}} = 0 + 0 = \frac{32 \cdot 10^3}{0,13 \cdot 783,2 \cdot 10^3} = 0,31 \leq 1
\]

Is equal to the buckling around the weak axis. With

$k_{zy} = 0.6 \cdot k_{yy} = 0.6 \cdot 1.07 = 0.64$

the total check results in 0.64 which is still smaller than 1.0. Therefore the beam fulfils the check.

**Horizontal deflection**

Again the horizontal deflection of this slender column is the determining factor. The total horizontal deflection in the SLS will be 18 mm (Figure 6.5). Therefore the profile succeeds in every calculation.

**Alternative profile ?**

Since the horizontal deflection is by far the most determining factor an alternative profiles can be opted. A cable structure or a vertical truss can decrease the material usage for this façade columns. However this is not done in this design since some columns do have some compression load which has great influence on the buckling behaviour of the column, and less on the deflection of the column (calculated hereafter). Since both types are aligned in the same line one single column type is chosen for the façade elements.

![Figure 6.5](image-url)

*Horizontal deflection Case 2, SLS*
Steel façade column 2 (case 1)

This column, located at grid line intersection of E and 8 (Figure 6.6) is checked in a similar way as the column of E-11. This column however has a larger compressive load due to the staircase system (Figure 6.7 and 6.8), which are partly carried by the column. The column is therefore checked on strength and stability. Since the calculations are similar to E-11 the elaboration is stated only briefly in this case.

Total length column: 15.8 m  
C.t.c distance columns: 1.3 m  
Connections with façade: every 2.5 m  
Steel strength: S275 (standard)  
Connections column: hinge and rolling bar (fictional)
Permanent loads column

Own weight column: 0,3 kN/m
Load façade: 8,13 kN

Variable loads column

Wind load (extreme south wind):
\[ F_{w1} = c_sc_d \cdot c_r \cdot q_p(b) \cdot A_{ref} = \]
\[ 0,85 \cdot 0,8 \cdot 0,884 \cdot 3,25 = 1,95 \text{ kN} \]
\[ F_{w2} = c_sc_d \cdot c_r \cdot q_p(b) \cdot A_{ref} = \]
\[ 0,85 \cdot 0,8 \cdot 1,045 \cdot 3,25 = 2,31 \text{ kN} \]

Total loads ULS column
1,2 \cdot 8,13 = 9,75 \text{ kN}
1,2 \cdot 0,3 = 0,36 \text{ kN / m}
1,5 \cdot 1,95 = 2,93 \text{ kN}

Total loads SLS
1,0 \cdot 8,13 = 8,13 \text{ kN}
1,0 \cdot 0,3 = 0,3 \text{ kN / m}
1,0 \cdot 1,95 = 1,95 \text{ kN}
1,0 \cdot 2,93 = 2,93 \text{ kN}

Permanent loads floors
Width carried by column: 1,3 m
CLT floor 1,5 kN/m²
Total 1,95 kN/m

Variable loads floors
Balcony/staircase 4,0 kN/m²
Momentaneous 1
Total 5,2 kN/M

Figure 6.8
Mechanical scheme as drawn in MatrixFrame.
Total loads ULS floors
\[ 1.5 \cdot 5.2 + 1.2 \cdot 1.95 = 23.4 + 7.8 = 10.14 \text{ kN/m}^3 \]

Total loads SLS floors
\[ 1.0 \cdot 5.2 + 1.0 \cdot 1.95 = 7.15 \text{ kN/m}^3 \]

2D loads results

The column is modelled in 2D in matrixFrame, resulting in compression load \( N_{Ed} \), bending moment in one direction \( M_y \) and shear load \( V_{Ed} \). Since the column is not checked in a 3D situation the \( M_z \) acting on the column is unknown. For the ULS:

- \( N_{Ed} = 86 \text{ kN} \)
- \( V_{Ed} = 5.5 \text{ kN} \)
- \( M_y = 6 \text{ kNm} \)

Chosen profile IPE 200
- \( h = 200 \text{ mm} \)
- \( b = 100 \text{ mm} \)
- \( I_y = 1943.0 \cdot 10^4 \text{ mm}^4 \)
- \( I_z = 142.3 \cdot 10^4 \text{ mm}^4 \)
- \( i_y = 80.7 \text{ mm} \)
- \( i_z = 22.36 \text{ mm} \)
- \( A = 2848 \text{ mm}^2 \)
- \( W_y = 220 \cdot 10^3 \text{ mm}^3 \)
- \( W_z = 44.61 \cdot 10^3 \text{ mm}^3 \)

Compression strength
\[ \frac{N_{Ed}}{N_{c,Rd}} \leq 1.0 \]
\[ N_{c,Rd} = \frac{A \cdot f_y}{\gamma_m} = \frac{2848 \cdot 275}{100} = 783 \cdot 10^3 \text{ N} \]
\[ N_{Ed} = \frac{85 \cdot 10^3}{783 \cdot 10^3} = 0.15 \leq 1.0 \]

Bending strength
\[ \frac{M_{Ed}}{M_{c,Rd}} \leq 1.0 \]
\[ M_{c,Rd} = \frac{W_y \cdot f_y}{\gamma_m} = \frac{220 \cdot 10^3 \cdot 275}{100} = 60.5 \cdot 10^6 \text{ N} \]
\[ M_{Ed} = 6 \cdot 10^6 \text{ Nm} \]
\[ \frac{M_{Ed}}{M_{c,Rd}} = 0.10 \leq 1.0 \]

Shear strength
\[ \frac{V_{Ed}}{V_{c,Rd}} \leq 1.0 \]

Since a IPE200 is considered as a class 1 section:
\[ V_{c,Rd} = V_{pl,Rd} = \frac{A_f (f_{yd})}{\gamma_{m0}} \]
\[ A_f = A \cdot h = \frac{2048 \cdot 200}{200 + 100} = 1899 \text{ mm}^2 \]
\[ \frac{275}{\sqrt{3}} = 158.8 \]
\[ V_{c,Rd} = \frac{1899 \cdot 158.8}{100} = 301.56 \text{ kN} \]
\[ V_{Ed} = \frac{4}{302} = 0.013 \leq 1.0 \]

Compression, shear force and bending force combined
\[ \frac{N_{Ed}}{N_{c,Rd}} + \frac{V_{Ed}}{V_{c,Rd}} + \frac{M_{y,Ed}}{M_{c,Rd}} + \frac{M_{z,Ed}}{M_{z,c,Rd}} \leq 1.0 \]
\[ 0.15 + 0.013 + 0.10 + 0 = 0.26 < 1.0 \]

Buckling due to compression (strong axis)

Buckling can occur since the column is loaded in compression and bending. First the strong axis (perpendicular to the façade) is checked. The largest possible buckling length is 8 meters.
\[ N_{cr} = \frac{\pi^2 \cdot E I}{l_{buc}^2} = \frac{\pi^2 \cdot 2.1 \times 10^5 \cdot 1943.0 \times 10^4}{8000^2} = 629 \text{ kN} \]

\[ \bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \sqrt{\frac{2848 \cdot 275}{629 \times 10^3}} = 1.12 \]

\[ \bar{N} = \sqrt{\frac{N_{cr}}{N_{cr}}} = \sqrt{\frac{783}{629}} = 1.12 \]

With alpha = 0.21 for hot formed profiles in strong axis.

\[ \Phi = 0.5 \cdot \left[ 1 + \alpha (\bar{\lambda} - 0.2) + \bar{\lambda}^2 \right] \]

\[ \Phi = 0.5 \cdot \left[ 1 + 0.21(1.12 - 0.2) + 1.12^2 \right] = 1.22 \]

\[ \chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{1.22 + \sqrt{1.22^2 - 1.12^2}} = 0.58 \]

\[ N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = 0.58 \cdot 2848 \cdot 275 \times 100 = 460 \text{ kN} \]

\[ \frac{N_{Ed}}{N_{b,Rd}} = \frac{85}{460} = 0.18 < 1.0 \]

As expected no buckling will occur at the strong axis.

**Buckling due to compression (weak axis)**

Buckling in the weak axis is more likely, again the buckling length is decreased to 5 meters. The mullions of the façade work as buckling stabilizers.

\[ N_{cr} = \frac{\pi^2 \cdot E I}{l_{buc}^2} = \frac{\pi^2 \cdot 2.1 \times 10^5 \cdot 1423 \times 10^4}{5000^2} = 117.97 \text{ kN} \]

\[ \bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \sqrt{\frac{2848 \cdot 275}{117.97 \times 10^3}} = 2.58 \]

\[ \bar{N} = \sqrt{\frac{N_{cr}}{N_{cr}}} = \sqrt{\frac{783}{117.97}} = 2.58 \]

With alpha = 0.34 for hot formed profiles in weak axis.

\[ \Phi = 0.5 \cdot \left[ 1 + \alpha (\bar{\lambda} - 0.2) + \bar{\lambda}^2 \right] \]

\[ \Phi = 0.5 \cdot \left[ 1 + 0.34(2.58 - 0.2) + 2.58^2 \right] = 4.22 \]

\[ \chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{4.22 + \sqrt{4.22^2 - 2.58^2}} = 0.13 \]

\[ N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0.13 \cdot 2848 \cdot 275}{100} = 103.46 \text{ kN} \]

\[ \frac{N_{Ed}}{N_{b,Rd}} = \frac{85}{103.46} = 0.82 < 1.0 \]

The column suffices with the decreased buckling length.

**Horizontal deflection**

The maximal deflection requirements can be described as:

\[ w_{max} = \frac{l}{300} \cdot \frac{15800}{300} = 52 \text{ mm} \]

The deflection calculated with matrixFrame is 4 mm at maximum. This very small deflection can be explained since the larger compressive forces make the system stiffer. Furthermore, this system is stiffer compared with the stand-alone column E11. Therefore the deflection is not governing but the buckling instead.
Steel façade column 2 (case 2)

Column 2 at E-8 is also checked with the different wind direction (extreme wind load from the north). In this case lateral torsional buckling is appearing and therefore needs to be checked.

Changing variable loads

Wind load (extreme north wind resulting wind suction):
\[ F_{w1} = c_c c_d c_r q_p(b) A_{ef} \]
\[ F_{w1} = 0,85 \cdot 0,5 \cdot 0,884 \cdot 3,25 = -1,22 \text{ kN} \]
\[ c_c c_d = 0,85 \]
\[ c_r(D) = -0,5 \]
\[ q_p(b) = 0,884 \text{ kN/m}^2 \]
\[ A_{ef} = 3,25 \text{ m}^2 \]

2D loads results

\[ N_{Ed} = 85 \text{ kN} \]
\[ V_{Ed} = 4 \text{ kN} \]
\[ M_y = 4 \text{ kNm} \]

Buckling due to compression (strong axis)

Buckling can occur since the column is loaded in compression and bending. First the strong axis (perpendicular to the façade) is checked.

\[ N_{cr} = \frac{\pi^2 \cdot E I}{l_{buc}^2} = \frac{\pi^2 \cdot 2,1 \cdot 10^5 \cdot 1943,0 \cdot 10^4}{8000^2} = 629 \text{ kN} \]
\[ \lambda = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \frac{l_{buc}}{i + \lambda_i} = \sqrt{\frac{2848 \cdot 275}{629 \cdot 10^3}} = 1,12 \]
\[ \chi = \frac{N_{pl}}{N_{cr}} = \sqrt{\frac{783}{629}} = 1,12 \]

With alpha = 0,21 for hot formed profiles in strong axis.

\[ \Phi = 0,5 \cdot \left[ 1 + \alpha(\lambda - 0,2) + \lambda^2 \right] \]
\[ \Phi = 0,5 \cdot \left[ 1 + 0,21(1,12 - 0,2) + 1,12^2 \right] = 1,22 \]

\[
\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = \frac{1}{1,22 + \sqrt{1,22^2 - 112^2}} = 0,58
\]

\[
N_{b,Rd} = \frac{N_{Ed}}{\gamma_{M1}} = \frac{85}{1,00} = 85 \text{ kN}
\]

As expect no buckling will occur at the strong axis.

Buckling due to compression (weak axis)

Buckling in the weak axis with 5 meter buckling length:

\[ N_{cr} = \frac{\pi^2 \cdot E I}{l_{buc}^2} = \frac{\pi^2 \cdot 2,1 \cdot 10^5 \cdot 142,3 \cdot 10^4}{5000^2} = 117,97 \text{ kN} \]
\[ \lambda = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \frac{l_{buc}}{i + \lambda_i} = \sqrt{\frac{2848 \cdot 275}{117,97 \cdot 10^3}} = 2,58 \]
\[ \chi = \frac{N_{pl}}{N_{cr}} = \sqrt{\frac{783}{117,97}} = 2,58 \]

With alpha = 0,34 for hot formed profiles in weak axis.

\[ \Phi = 0,5 \cdot \left[ 1 + \alpha(\lambda - 0,2) + \lambda^2 \right] \]
\[ \Phi = 0,5 \cdot \left[ 1 + 0,34(2,58 - 0,2) + 2,58^2 \right] = 4,22 \]

\[
\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = \frac{1}{4,22 + \sqrt{4,22^2 - 2,58^2}} = 0,13
\]

\[
N_{b,Rd} = \frac{N_{Ed}}{\gamma_{M1}} = \frac{0,13 \cdot 2848 \cdot 275}{1,00} = 103,46 \text{ kN}
\]

\[
N_{b,Rd} = \frac{85}{103,46} = 0,82 < 1,0
\]

The column suffices with the decreased buckling length.
Lateral torsional buckling

The wind load causes suction on the façade, as a result of this the façade on the tensile zone of the column and lateral torsional buckling can occur. This het influence on the total buckling case. A check is made to research if lateral torsional protection needs to be applied.

\[ M_{b,Rd} = \chi_{LT} \cdot W_y \cdot \frac{f_y}{\gamma_{M1}} \]

\[ W_y = W_{p,y} = 200 \cdot 10^3 \text{ mm}^3 \]

\[ f_y = 275 \]

\[ \gamma_{M1} = 1,00 \]

The lateral torsional reduction factor can be calculated with the help of the following equations:

\[ \chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \]

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

\[ h = \frac{200}{100} = 2 \rightarrow \alpha_{LT} = 0,21 \]

\[ \bar{\lambda}_{LT} = \sqrt{\frac{W_y \cdot f_y}{M_{Cr}}} \]

Now C is known it is possible to equate \( M_{crit} \):

\[ M_{Cr} = k_{red} \cdot \frac{C}{L_g} \cdot \sqrt{E \cdot I_z \cdot G \cdot I_I} \]

\[ M_{Cr} = 1 \cdot \frac{14.89}{15800} \cdot \sqrt{210000 \cdot 1423 \cdot 10^4 \cdot 81000 \cdot 6848 \cdot 10^4} = 38,39 \text{ kNm} \]

This results into a relative lateral torsional buckling slenderness of:

\[ \bar{\lambda}_{LT} = \frac{\sqrt{W_y \cdot f_y}}{M_{Cr}} = \sqrt{38,39 \cdot 10^6} = 1,25 \]

Followed by:

\[ \chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \]

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

\[ h = \frac{200}{100} = 2 \rightarrow \alpha_{LT} = 0,21 \]

\[ \bar{\lambda}_{LT} = \frac{\sqrt{W_y \cdot f_y}}{M_{Cr}} \]
\[ \bar{\lambda}_{LT} = \sqrt{\frac{W_y \cdot f_y}{M_{cr}}} = 1.25 \]
\[ \alpha_{LT} = 0.21 \]
\[ \Phi_{LT} = 0.5 \left[ 1 + \alpha_{LT} \left( \lambda_{LT}^2 - 0.2 \right) + \lambda_{LT}^2 \right] \]
\[ \Phi_{LT} = 0.5 \left[ 1 + 0.21 \left( 1.25 - 0.2 \right) + 1.25^2 \right] = 1.40 \]
\[ \chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \lambda_{LT}^2}} = \frac{1}{1.40 + \sqrt{1.40^2 - 1.25^2}} = 0.49 \]

Now all factors are known to equate the lateral torsional buckling check.
\[ M_{b,Ed} = \chi_{LT} \cdot W_y \cdot f_y = 0.49 \cdot 200 \cdot 10^3 \cdot 275 = 30,01 \text{ kNm} \]
\[ \frac{M_{Ed}}{M_{b,Ed}} < 10 \]
\[ \frac{4}{30.01} = 0.13 < 1.0 \]
So the column fulfills the lateral torsional buckling test.

**Buckling: compression & bending**

Since the horizontal load increases the instability of the column a combination check should be equated.
\[ \frac{N_{Ed}}{\chi_y \cdot N_{Rk}} + k_{yy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \cdot \gamma_{M1}} + k_{yz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\gamma_{M1}} \leq 1 \]
\[ \frac{N_{Ed}}{\chi_z \cdot N_{Rk}} + k_{zy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \cdot \gamma_{M1}} + k_{zz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\gamma_{M1}} \leq 1 \]
\[ \chi_{LT} \neq 0 \text{ so the lateral torsional buckling has influence on the total buckling problem. Therefore this combination check has to be elaborated.} \]

\[ C_{my} = 0.95 + 0.05 \cdot \alpha_h = 0.95 + 0.05 \cdot \frac{1}{12} = 0.96 \]
\[ \alpha_h = \frac{M_h}{M_s} = \frac{1}{12} \]
\[ k_{yy} = C_{my} \left( 1 + \frac{\lambda_y - 0.2}{\chi_y} \right) \frac{N_{Ed}}{N_{Rk}} \]
\[ k_{yy} = 0.96 \left( 1 + (1.25 - 0.2) \cdot \frac{85 \cdot 10^3}{0.49 \cdot 783.2 \cdot 10^3} \right) = 1.18 \]

Since \( M_z \) is unknown and taken zero the first out of two equations can be solved.
\[ \frac{N_{Ed}}{\chi_y \cdot N_{Rk}} + k_{yy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \cdot \gamma_{M1}} + k_{yz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\gamma_{M1}} \leq 1 \]
\[ \frac{N_{Ed}}{\chi_z \cdot N_{Rk}} + k_{zy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \cdot \gamma_{M1}} + k_{zz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\gamma_{M1}} \leq 1 \]
\[ \frac{85 \cdot 10^3}{0.49 \cdot 783.3 \cdot 10^3} + 1.18 \cdot 4 \cdot 10^6 + 0 \]
\[ 0.49 \cdot 60.5 \cdot 10^6 \]
\[ k_{yz} \cdot 0 = 0.37 < 1 \]

Furthermore:
\[ \frac{N_{Ed}}{\chi_z \cdot N_{Rk}} + k_{zy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \cdot \gamma_{M1}} + k_{zz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\gamma_{M1}} \leq 1 \]
\[ \frac{85 \cdot 10^3}{0.13 \cdot 783.3 \cdot 10^3} + 0.7 \cdot 4 \cdot 10^6 + 0 \]
\[ 0.49 \cdot 60.5 \cdot 10^6 \]
\[ k_{zz} \cdot 0 = 0.92 < 1 \]

However, the second equation the \( k_{zy} \) may be assumed to be zero for IPE beams according to the Dutch Annex, resulting in the following check:
\[ \frac{N_{Ed}}{\chi_{z} \cdot N_{Rk}} + 0 = \frac{85 \cdot 10^3}{0.13 \cdot 783.2 \cdot 10^3} = 0.82 < 1 \]

The beam fulfills the check.
Steel column connection

The difference between column E8 and E11 is the compressional load transferred by the element. Column E8 is an element loaded with a relative large compression force due to the floor at level 12 meter, while column E11 only takes the wind load and the own weight of the façade. The steel columns are connected to the concrete retaining walls (basement walls) in a very regular way, with a steel plate (voetplaat). This plate is connected to the concrete with anchoring bolts. These anchors cause extra point loads on the retaining wall and need to be checked in a further development.

First the dimensions of the plate need to be determined, this will be minimal required dimensions due to the small loads, furthermore the thickness of the plate should be calculated, again it is expected that the minimal dimensions suffices. Afterwards the amount (probably four) and the size of the anchors can be calculated. Together with the welding connection of the column an the plate. Finally the concrete needs to be checked on a local punching shear failure.

For this thesis it was not relevant to calculate such a standard connection, therefore it is only mentioned.
Overview
Core A wall A
Cora A wall B
Retaining wall
Basement floor
Pile foundation plan
Parking garage
Concrete core A

The cores are calculated as four, independent stability walls (Figure 7.1) with a governing south-west wind load case. This conservative approach is chosen since the gap in the core-tube is relatively large. The stiffness of the core will be larger in reality, however no problems are expected since it is a mid-rise building.
Figure 7.1

Floor level +13 m with the stability walls for the north-south direction located. For the west-east direction the other two walls of each core provide stability.

1. Core A, wall A
2. Core A, wall B
3. Core B, wall A
4. Core B, wall B
Wind loads cores

The wind loads as determined in the chapter ‘Loads’ are used to calculate the wind loads acting on each part of the cores. This is done by an approximation calculation in which the floor plan of the building is simplified to a beam, supported by four spring supports. Each spring illustrates one wall of a core. For the south-west wind direction there are three scenarios calculated.

1. From level 0 to level +7 m
2. From level +7 to level +17 (cantilever)
3. From level +17 to level +22

The stiffness of each spring determines the amount of load it takes, since the stiffer the element the more load is attracted to that element. In this phase it is difficult to determine the exact spring stiffness of a core. However it is easy to make a statement on the relative stiffness of each spring. Therefore the width of the wall is taken and slightly reduced since the openings of the elevator doors reduce the stiffness of the wall.

Spring stiffness for each wall (south-west wind)
- Core A left wall (grid line 3) = 4,4·10³ kN/m
- Core A right wall (4) = 3,8·10³ kN/m
- Core B left wall (9) = 5,4·10³ kN/m
- Core B right wall (9-10) = 4,5·10³ kN/m

Note: the stiffness of Core A right wall (4) is determined with a simple SCIA model. In the model a point load is applied to a solid wall and the deflection at the top of the wall is measured (Figure 7.2). The deflection of the model is compared to a solid wall with gaps (elevator entrances) and the relative stiffness is calculated. The stiffness of the wall with gaps is 85% compared to the solid wall without gaps.

The q load is applied for situation 1 and 2 (Figure 7.3). The suction and pressure coefficient work both in equal direction on the core. Therefore the suction factor is taken positive and summarized with the pressure.

\[
q_{\text{pressure}} = 0,60 \text{ kN} / \text{m}^1
\]
\[
q_{\text{suction}} = -0,38 \text{ kN} / \text{m}^1
\]
\[
Q_{uls} = 1,5 \cdot (0,6 + 0,38) = 0,90 + 0,56
\]
\[
Q_{uls} = 1,0 \cdot (0,6 + 0,38) = 0,60 + 0,38
\]

The q load for situation 3:

\[
q_{\text{pressure}} = 0,71 \text{ kN} / \text{m}^1
\]
\[
q_{\text{suction}} = -0,44 \text{ kN} / \text{m}^1
\]
\[
Q_{uls} = 1,5 \cdot (0,71 + 0,44) = 1,06 + 0,67
\]
\[
Q_{uls} = 1,0 \cdot (0,71 + 0,44) = 0,71 + 0,44
\]

The results are shown in Figure 7.4. The wind load acting on the steel trusses in the roof is not taken into account with this overview. For this upper 2 meters of the building a reduced wind load is assumed and applied to the cores.

![Figure 7.2](image)
Stiffness of the walls calculated with the deflections found in SCIA.
Three scenarios used to calculate the reaction forces in the springs.

Figure 7.4
Reaction loads of the south-west wind load on the cores.

-3 - 0
Basement (water pressure)

1. Low
0 - 7
Core A left wall (3) 11,4 kN 7,6 kN
right wall (4) 11,1 kN 7,4 kN
Core B left wall (9) 24,6 kN 16,4 kN
right wall (9-10) 21,4 kN 14,3 kN

2. Middle
7 - 17
Core A left wall (3) 20,5 kN 13,3 kN
right wall (4) 17,1 kN 11 kN
Core B left wall (9) 23,8 kN 15,4 kN
right wall (9-10) 19,7 kN 12,7 kN

3. Top
17 - 21
Core A left wall (3) 23,8 kN 15,5 kN
right wall (4) 20,4 kN 13,3 kN
Core B left wall (9) 27,8 kN 18,2 kN
right wall (9-10) 23,1 kN 15,1 kN
Core A

For calculating the concrete cores the cores are simplified to two stocky beams each with a stiff connection at the foundations. This simplification is summarized with an endless stiff core on a rotating foundation, since the pile foundation acts as a spring (Figure 7.5). The equations used to calculate the required amount of reinforcement are design calculations.

**Loads acting on the concrete cores:**

1. Determination of the acting wind loads on the core. (As shown previously)
2. Determination of the normal loads acting directly on the core.
3. Determination of the normal loads acting indirectly on the core, which is equal to the building weight calculation. (aanpendelende belasting).

These loads are applied to a MatrixFrame model to calculate the bending moment at the foundation. The design calculation is similar as done in the book ‘Ontwerpen in gewapend beton’ CB47.1.
Loads acting directly on core A

An assumption is made of the loads which are directly transferred by the core. Since there is a cantilever applied at the west part of the building the core will mostly transfer 50% of the loads of the west part of the building. The other 50% is transferred by the timber truss system. The core is existing of four walls, it is assumed that the load acting on the core is equally divided by the four walls. This is a simplification.

<table>
<thead>
<tr>
<th>Load</th>
<th>Reduction factor</th>
<th>Total load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Own weight core</td>
<td>3456 kN</td>
<td>3456 kN</td>
</tr>
<tr>
<td>Elevator and building services</td>
<td>100 kN</td>
<td>100 kN</td>
</tr>
<tr>
<td>Emergency staircase</td>
<td>50 kN</td>
<td>50 kN</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Load</th>
<th>Reduction factor</th>
<th>Total load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>760 0,5</td>
<td>380 kN</td>
</tr>
<tr>
<td>Truss B</td>
<td>1135 1</td>
<td>1135 kN</td>
</tr>
<tr>
<td>Truss CC</td>
<td>812 0,5</td>
<td>406 kN</td>
</tr>
<tr>
<td>Truss DD</td>
<td>1225 0,5</td>
<td>612 kN</td>
</tr>
<tr>
<td>Level 16</td>
<td>2655 0,5</td>
<td>1327 kN</td>
</tr>
<tr>
<td>Level 12</td>
<td>1181 0,5</td>
<td>590 kN</td>
</tr>
<tr>
<td>Level 9</td>
<td>1022 0,5</td>
<td>511 kN</td>
</tr>
<tr>
<td>Level 6</td>
<td>1619 0,5</td>
<td>809 kN</td>
</tr>
<tr>
<td>Level 1,5</td>
<td>2282 0,25</td>
<td>570 kN</td>
</tr>
<tr>
<td>Level -3</td>
<td>5550 0,1</td>
<td>555 kN</td>
</tr>
<tr>
<td>Total</td>
<td>6898 kN</td>
<td></td>
</tr>
<tr>
<td>Total including safety 1,35</td>
<td>9312 kN</td>
<td></td>
</tr>
</tbody>
</table>

**Total load per wall (max)** 2328 kN

**Total load per wall (55% min)** 1280 kN

q (max) 89 kN/m

Wind loads acting on core A - left wall

<table>
<thead>
<tr>
<th>Load</th>
<th>ULS</th>
<th>SLS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 0 - 7</td>
<td>11,4 kN</td>
<td>7,6 kN</td>
</tr>
<tr>
<td>Level 7 - 17</td>
<td>20,5 kN</td>
<td>13,3 kN</td>
</tr>
<tr>
<td>Level 17 - 21</td>
<td>23,8 kN</td>
<td>15,5 kN</td>
</tr>
<tr>
<td>Level 21 - 23</td>
<td>8 kN</td>
<td>6 kN</td>
</tr>
</tbody>
</table>

Indirect Normal load
(Aanpendelende belasting)

<table>
<thead>
<tr>
<th>Load</th>
<th>Total</th>
<th>25%</th>
<th>12380 kN (per wall)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total</td>
<td>49518 kN</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Wall A - design

Structural model

The loads calculated earlier are applied on a simplified core with a fictional inclination at the foundation. With this model the maximal bending moment in the foundation becomes clear (Figure 7.6).

\[ M_{\text{foundation}} = 6068 \text{ kNm} \]

Pile foundation assumptions

Little is known about the soil properties of the soil of the location. Therefore it is hard to make a statement about a proper foundation. A fictional pile foundation is assumed in order to make a more reliable core check (Figure 7.7). The fictional pile foundation of the left wall of core A exists out of 8 concrete prefab (pre-stressed) piles.

- Dimensions piles: 350 x 350 mm²
- Pile capacity: 1200 kN
- Pile length: 12 m
- Concrete type: C50/60

The spring stiffness of the piles is determined with the following equations:

\[ k_{\text{pile}} = \frac{E \cdot A}{(1 + \alpha) \cdot l} \]

With alpha as factor for flexibility in the soil. When the soil is weak the core can rotate easier, since no information of the soil is known the value is assumed to be 1. So the rotation stiffness of the foundation becomes:

\[ C = \sum k_{\text{pile},j} \cdot \alpha_j^2 \]

\[ C = 189 \cdot 10^3 \cdot (4 \cdot 1.7^2 + 4 \cdot 2.9^2) = 8.5 \cdot 10^6 \text{ kNm / rad} \]

Concrete core A wall left geometric information

- Wall height: 26000 mm
- Width: 300 mm
- Length: 4600 mm
- Concrete: C20/25

First the concrete wall is designed with reinforcement steel B500, round 12-150 mm.
1. Imperfections

First the imperfections of the concrete core are calculated. The bending moment caused by the wind load:

\[ M_{\text{foundation}} = 6068 \text{ kNm} \]

The extra bending moment caused by imperfections can be calculated with the following equation:

\[ \theta_i = \theta_0 \cdot \alpha_n \cdot \alpha_m \]

With:

\[ \theta_0 = \frac{1}{300} \]
\[ \alpha_n = \frac{2}{\sqrt{l}} = \frac{2}{\sqrt{26}} = 0,39 < 0,67 \]
\[ \alpha_n = 0,67 \]
\[ \alpha_m = \sqrt{0,5 \cdot \left(1 + \frac{1}{m}\right)} = \sqrt{0,5 \cdot \left(1 + \frac{1}{20}\right)} = 0,72 \]

Results in:

\[ \theta_i = \frac{1}{300} \cdot 0,67 \cdot 0,72 = 0,0016 \]

The imperfection causes an extra horizontal load of:

\[ q_{H,i} = \theta_i \cdot \frac{N_{V,Ed}}{l} \]
\[ q_{H,i} = 0,0016 \cdot \frac{12,38 \cdot 10^6}{26 \cdot 10^3} = 0,76 \text{ kN / m} \]

Resulting in the final bending moment at the foundation:

\[ M_{\text{tot,Ed}} = M_{\text{wind,Ed}} + M_{\text{imperfections,Ed}} = M_{\text{wind,Ed}} + \frac{1}{2} q_{H,i} l^2 \]
\[ M_{\text{tot,Ed}} = 6068 + \frac{1}{2} \cdot 0,76 \cdot 26^2 = 6325 \text{ kNm} \]

So the bending moment at the foundation increased slightly.

\[ \text{Figure 7.7} \]
Fictional foundation under left wall of core A
2. Check of 2nd-order calculation is required

A 2nd-order calculation is not necessary if the following equation is satisfied:

\[
\frac{n}{n-1} \leq 1,1 \quad \text{so} \quad n \geq 11
\]

\[
n = \frac{N_{cr}}{N_{V,Ed}}
\]

The buckling resistance of the wall is calculated with an approximation of the stiffness of the core. The relative amount of reinforcement bars is calculated.

\[
\rho_s = \frac{A_s}{A_c} = \frac{3 \cdot \pi \cdot (0.5 \cdot 12)^2 \cdot (1000 / 150)}{300 \cdot 1000}
\]

\[
\rho_s = \frac{1507}{300000} = 0,005
\]

Resulting in:

\[
\alpha_n = \frac{N_{Ed,mini}}{A_c \cdot f_{cd} + A_s \cdot f_{yd}}
\]

\[
\alpha_n = \frac{1280 \cdot 10^3}{300 \cdot 4600 \cdot \frac{25}{15} + 0,005 \cdot 300 \cdot 4600 \cdot 435} = 0,048
\]

Now the fictional elasticity modulus can be determined:

\[
E_f = \left[1,6 + 420 \cdot 0,004 + (14,0 - 160 \cdot 0,004) \cdot 0,048\right] \cdot 10^3
\]

\[
E_f = 4060 \quad N / \text{mm}^2
\]

With the fictional elasticity modulus the stiffness of the concrete core becomes:

\[
EI = 4060 \cdot \frac{1}{12} \cdot 300 \cdot 4600^3 = 9,88 \cdot 10^{15} \quad \text{Nmm}^2
\]

The buckling length of the wall can now be determined with the following equation (since the building has over five levels):

\[
l_0 = l \sqrt{1,12^2 + \frac{\pi^2}{2 \rho}}
\]

With:

\[
\rho = \frac{Cl}{(EI)_d} = \frac{8,5 \cdot 10^{12} \cdot 26 \cdot 10^3}{9,88 \cdot 10^{15}} = 22,37
\]

Results in the following buckling length:

\[
l_0 = 26 \sqrt{1,12^2 + \frac{\pi^2}{2 \cdot 22,37}} = 31,57 \quad \text{m}
\]

And a buckling resistance of:

\[
N_{cr} = \pi^2 \left(\frac{E I}{l_0^2}\right)_2 = \pi^2 \frac{9,88 \cdot 10^{15}}{(31,57 \cdot 10^3)^2} = 97,83 \quad \text{kN}
\]

\[
n = \frac{N_{cr}}{N_{V,Ed}} = \frac{97,8 \cdot 10^6}{12,80 \cdot 10^6} = 7,64 < 11
\]

So the 2nd-order effect may not be neglected.

3. Calculation 2nd-order effect

The bending moment at the foundation is (again) increased by the 2nd-order effect. The enlargement can be described with the calculated in.

\[
M_{Ed} = \frac{n}{n-1} M_{0,Ed} = \frac{7,64}{7,64 - 1} \cdot 6325 = 7277 \quad \text{kNm}
\]

The eccentricity is checked:

\[
e_0 \geq \frac{h}{30} = \frac{4600}{30} = 153 \quad \text{mm}
\]

The eccentricity with a maximal Normal load acting on the concrete wall becomes:

\[
e_0 = \frac{M_{Ed}}{N_{Ed}} = \frac{7277 \cdot 10^6}{2328 \cdot 10^3} = 3126 \quad \text{mm}
\]

Therefore the eccentricity check satisfies.
4. Calculation of the reinforcement

Now all loads are known it is possible to check the applied reinforcement bars and decide if extra reinforcement is required. An overview of the stresses acting on the section of the concrete wall:

\[
\begin{align*}
N_{\text{max}} & \quad 2328 \quad \text{kN} \\
N_{\text{min}} & \quad 1280 \quad \text{kN}
\end{align*}
\]

\[
\begin{align*}
N_{\text{Ed, max}} & = \frac{N_{\text{Ed}}}{A_c} = \frac{-2328 \cdot 10^3}{300 \cdot 4600} = -1.68 \quad \text{N/mm}^2 \\
N_{\text{Ed, min}} & = \frac{N_{\text{Ed}}}{A_c} = \frac{-1280 \cdot 10^3}{300 \cdot 4600} = -0.92 \quad \text{N/mm}^2 \\
M_{\text{Ed}} & = \frac{M_{\text{Ed}}}{W_c} = \frac{7277 \cdot 10^6}{1 \cdot 300 \cdot 4600^2} = 6.87 \quad \text{N/mm}^2
\end{align*}
\]

Adding the stresses result in the following maximal normal stresses:

\[
\sigma_{c,d,\text{max}} = -1.68 - 6.87 = -8.55 \quad \text{N/mm}^2
\]
\[
\sigma_{c,d,\text{min}} = -1.68 + 6.87 = 5.19 \quad \text{N/mm}^2
\]

Minimal normal stresses (Figure 7.8):

\[
\sigma_{c,d,\text{max}} = -0.92 - 6.87 = -7.79 \quad \text{N/mm}^2
\]
\[
\sigma_{c,d,\text{min}} = -0.92 + 6.87 = 5.95 \quad \text{N/mm}^2
\]

The stresses calculated show that there is a certain area where tensile stresses are appearing in the section. These tensile stresses are distributed by the reinforcement, and the tensile capacity should therefore be sufficient.

The scenario with the minimal normal loads cause the largest tensile stresses, therefore this situation is governing.

Resulting in a distance \( x \) for tensile stresses, with a length of:

\[
x = \frac{5.95}{5.95 + 7.79} \cdot 4.6 \cdot 10^3 = 1.92 \quad \text{m}
\]

In this length the reinforcement bars transfer the tensile loads. The standard reinforcement used: Round 12-150 in front, middle and back of the wall.

\[
A_s = 2261 \quad \text{mm}^2 / m
\]

\[
\sigma_{\text{capacity}} = \frac{F_{rdl}}{A_c} = \frac{339 \cdot 435}{150 \cdot 300} = 3.28 \quad \text{N/mm}^2
\]

The chosen reinforcement does not satisfy. A double net does satisfy.

\[
\sigma_{\text{double}} = 3.28 \cdot 2 = 7.64 \quad \text{N/mm}^2
\]

\[
\sigma > 3.28 \quad \text{N/mm}^2 \quad \text{for 894 mm}
\]

So the additional reinforcement satisfies. On the next pages a section drawing with the applied reinforcement is shown (Figure 7.9). However, partial instability is checked first.

Also the crack-width caused by the tensional stresses may be problematic, this is checked after the design calculation is completed.

\[\text{Figure 7.8} \quad \text{Stress figure of Core A left wall.}\]
5. Partial instability check

Partial instability means the collapse of one part of the core. In this check the wall between the basement floor and level +1.5 m is checked. 1/8 of the wall is taken and checked on bending and compression. The maximal normal forces are used.

Length wall: 575 mm
Height wall: 4500 mm
Thickness: 300 mm

Maximal normal force at edge of wall: 8.55 N/mm²
Tensile force with this Nmax: 5.19 N/mm²

After 894 mm from the edge the compression stresses are decreased to:

\[ \sigma_{c,d} = \frac{894}{4600} \cdot (8.55 + 5.19) = 2.67 \text{ N/mm}^2 \]

Normal force in this point:

\[ N_{Ed} = \frac{8.55 + (8.55 - 2.67)}{2} \cdot 300 \cdot 894 = 1935 \text{ kN} \]
\[ M_{Ed} = 12 \text{ kNm} \]

To check if a 2nd-order calculation is necessary the slenderness of the section is checked. The maximum slenderness can be calculation with:

\[ \lambda_{lim} = \frac{20 \cdot ABC}{\sqrt{n}} \]

With

A: 0.7
B: 1.1
C: 2.7

\[ n = \frac{N_{Ed}}{A_c \cdot f_{cd}} = \frac{1935 \cdot 10^3}{300 \cdot 894 \cdot 1.5} = 0.54 \]

So:

\[ \lambda_{lim} = \frac{20 \cdot ABC}{\sqrt{n}} = \frac{20 \cdot 0.7 \cdot 1.1 \cdot 2.7}{\sqrt{0.54}} = 56.58 \]

Present slenderness:

\[ \lambda = \sqrt{\frac{12 \cdot l}{h}} = \sqrt{12 \cdot 0.7 \cdot \frac{4500}{300}} = 36.37 \]
\[ 36.37 < 56.58 \]

So no 2nd-order check is required.

The first order check can be done with an assumed imperfection of 1/600 over the buckling length of the wall. A summation of the (assumed) bending moment in the floor and the extra eccentricity caused by the imperfections is elaborated.

\[ e_0 = \frac{12 \cdot 10^6}{1935 \cdot 10^3} + 0.7 \cdot 4500 \div 600 = 11.45 \text{ mm} \]
\[ 11 < 20 \text{ mm} \]

\[ e_0 = 20 \text{ mm is governing} \]

Furthermore:

\[ a = c + \frac{1}{2} \varphi_z = 30 + 6 = 36 \text{ mm} \]
\[ \frac{a}{h} = \frac{36}{300} = 0.12 \]
\[ \frac{N_{Ed}}{A_c \cdot f_{cd}} - \frac{20}{1.5} \cdot \frac{300 \cdot 894}{1.5} = 0.54 \]
\[ \frac{N_{Ed}}{A_c \cdot f_{cd}} \cdot \frac{e_0}{h} = 0.54 \cdot \frac{20}{300} = 0.04 \]

With use of the M-N-interaction diagrams it becomes clear that no additional reinforcement is required.

The chosen design reinforcement (Figure 7.9):

\[ \varnothing 12 -150 \text{ mm } \cdot 3 \]
extra \[ 3 \cdot 6 \times \varnothing 12 \text{ mm } = 18 \]
Figure 7.9
Reinforcement drawing of the left wall of Core A
Crack control (SLS)

For durability reasons cracking of the concrete wall needs to be checked. The cracks cause corrosion of the reinforcement, which needs to be prevented. First a minimum reinforcement check is elaborated. Afterwards the crack control is checked with charts and with a (more precise) direct calculation.

A small strip of the wall is used for calculating the cracking (Figure 7.10) This is the ultimate outer strip, containing one layer of reinforcement. It is assumed that the cross-bars do not transfer any tensile stress.

Concrete class:  C20/25
Reinforcement:  B500
h:  81,5 mm
(Height = coverage c + diameter cross bar + half the web distance of 75 mm)
b:  300 mm
c:  30 mm

Structural class:  S3
Durability class:  XC1

The concrete is designed as a sandwich panel with the structural part inside the building. When the concrete core would have been designed as one massive block the durability class would have been XS1 (building near the coast) which is very unfavourable. The basement retaining wall - calculated later - is designed in class XS1.

Minimal required reinforcement

The minimal required amount of reinforcement is calculated with the following equation:

\[ A_{s,min} = \frac{k_c \cdot k \cdot f_{ct,eff} \cdot A_{ct}}{\sigma_s} \]

With:

\[ k = 1,0 \quad (h < 300 \text{ mm}) \]
\[ f_{ct,eff} = 2,2 \text{ N/mm}^2 \]
\[ A_{ct} = 300 \cdot 81,5 = 24450 \text{ mm} \]
\[ \sigma_s = f_yk = 500 \text{ N/mm}^2 \]

And for compression + bending \( k_c \) becomes:

\[ \sigma_c = \frac{N_{ct,SLS}}{A_c} = \frac{1724 \cdot 10^3}{4600 \cdot 300} = 1,24 \text{ N/mm}^2 \]
\[ k_1 = 1,5 \]
\[ h = 81,5 \text{ mm} \]
\[ h^* = 81,5 \text{ mm} \]

\[ k_c = 0,4 \cdot \left[ 1 - \frac{\sigma_c}{k_1 \cdot (h/h^*) \cdot f_{ct,eff}} \right] \]

\[ 0,4 \cdot \left[ 1 - \frac{1,24}{1,5 \cdot (81,5/81,5) \cdot 2,2} \right] = 0,24 \]

Figure 7.10
Strip used when calculating the crack control.
Now the required amount of reinforcement can be checked.

\[ A_{s,\text{min}} = \frac{0,24 \cdot 1,0 \cdot 2,2 \cdot 24450}{500} = 26,74 \text{ mm}^2/m \]

The provided amount of reinforcement is much larger:

\[ A_{s,\text{bar}} = \frac{1}{4} \pi \cdot 12^2 = 113 \text{ mm}^2 \]
\[ A_{s,\text{prov}} = 3 \cdot 113 = 339 \text{ mm}^2 \]

So no problems occur in this check. Therefore the crack control checks can be elaborated according to NEN-EN 1992-1-1 7.3.3.1.

1. Check with charts

The stresses in the reinforcement bars directly after the initial cracks are calculated with the crack force of the section. This crack force is transferred by the reinforcement resulting in a steel stress of:

\[ \sigma_s = \frac{f_{\text{ct,eff}} \cdot b \cdot s (1 + \alpha_c \cdot \rho)}{n \cdot \frac{1}{4} \pi \cdot \varnothing^2} \]

With:

\[ f_{\text{ct,eff}} = 2,2 \text{ N / mm}^2 \]
\[ b = 300 \text{ mm} \]
\[ s = 106 \text{ mm} \]
\[ A_s = n \cdot \frac{1}{4} \pi \cdot \varnothing^2 = 339 \text{ mm}^2 \]
\[ \alpha_c = \frac{E_s}{E_c} = \frac{210000}{30000} = 7 \]
\[ \rho = \frac{A_s}{A_c} = \frac{339}{300 \cdot 81,5} = 0,0139 \]

Results in the following steel stresses:

\[ \sigma_s = \frac{2,2 \cdot 300 \cdot 106 (1 + 7 \cdot 0,0139)}{339} = 226 \text{ N / mm}^2 \]

Since the durability class XC1 is used the maximal crack width is \( w_{\text{max}} = 0,30 \text{ mm} \)

Interpolation of the chart 7.2N in NEN-EN 1992-1-1 7.3.3.1 results in a maximal reinforcement diameter of 20 mm. This value has to be corrected with the following equation:

\[ \varnothing_s = \varnothing_s \cdot \left( \frac{f_{\text{ct,eff}}}{2,9} \right) \cdot \frac{k_s h_{\text{cr}}}{2(h-d)} \]

With \( d \) as:

\[ d = h - c - \varnothing_{\text{crosbar}} - \frac{1}{2} \varnothing_{\text{bar}} = \]
\[ d = 81,5 - 30 - 8 - \frac{1}{2} \cdot 12 = 37,5 \text{ mm} \]

Results in a maximal diameter of:

\[ \varnothing_s = 20 \cdot \left( \frac{2,2}{2,9} \right) \cdot \frac{0,24 \cdot 81,5}{2(81,5 - 37,5)} = 3,5 \text{ mm} \]

So the criteria is not met.

Furthermore the maximal reinforcement bar distance needs to be checked. The maximal reinforcement bar distance given by chart 7.3N gives a value of \( s_{\text{max}} < 220 \text{ mm} \). Since \( s_{\text{applied}} = 106 \) and the web distance is 75 mm this value is met. Therefore the check satisfies.

Since only one out of two requirements should satisfy the check can be called a success. However a more precise crack calculation is elaborated to give more insight in the width of the crack.
2. Direct calculation

A more precise crack control is elaborated using the maximal crack width, which can be calculated using the following equation:

\[ s_{r, \text{max}} = k_3 c + \frac{k_4 k_5 \varnothing}{\rho_{p, \text{eff}}} \]

With:

- \( k_3 = 3,4 \)
- \( c = 30 \text{ mm} \)
- \( k_1 = 0,8 \)
- \( k_2 = 0,5 \)
- \( k_4 = 0,425 \)
- \( \varnothing = 12 \)

And:

- \( \rho_{p, \text{eff}} = \rho_{s, \text{eff}} = \frac{A_s}{A_{c, \text{eff}}} \)

The effective surface can be calculated with the following equations:

\[ A_{c, \text{eff}} = b \cdot h_{\text{eff}} \]

\[ h_{\text{eff}} = \min \left\{ \frac{2,5(h - d)}{(h - x)}, \frac{h}{3}, \frac{h}{2} \right\} \]

With use of distance \( x \):

\[ x = \left( -\alpha_e \cdot \rho_l + \sqrt{(\alpha_e \cdot \rho_l)^2 + 2 \cdot \alpha_e \cdot \rho_l} \right) d \]

\[ \rho_l = \frac{A_s}{b \cdot d} = \frac{339}{300 \cdot 37,5} = 0,03 \]

\[ \alpha_e = \frac{E_s}{E_c} \]

Since the beam is loaded under a long duration the effect of creep needs to be taken into account. This results in a reduced elasticity modulus for the concrete in the SLS condition.

With help of graph 3.1 in NEN-EN 1992-1-1 this reduced \( E \) can be read with:

\[ t_o = 28 \text{ d} \]

\[ \text{class} = N \]

\[ h_{b} = \frac{2A_c}{u} = \frac{2 \cdot (300 \cdot 81,5)}{2 \cdot 81,5 + 300} = 105,6 \text{ mm} \]

Results in:

- \( \varphi(\infty, t_o) = 1,0 \)
- \( \psi(\infty, t_o) = 1,5 \)

So:

\[ \alpha_s = \frac{E_s}{E_c} = \frac{2,1 \cdot 10^5}{3,0 \cdot 10^4} = 17,5 \]

\[ x = \left( -17,5 \cdot 0,03 + \sqrt{(17,5 \cdot 0,03)^2 + 2 \cdot 17,5 \cdot 0,03} \right) \]

\[ -37,5 = 23,5 \text{ mm} \]

Resulting in the following effective height:

\[ h_{\text{eff}} = \min \left\{ \frac{2,5 \cdot (h - d)}{(h - x)}, \frac{h}{3}, \frac{h}{2} \right\} \]

\[ h_{\text{eff}} = \left\{ \frac{2,5 \cdot (81,5 - 37,5)}{(81,5 - 37,5)} \right\} = 19,32 \text{ mm} \]

\[ h_{\text{eff}} = \left\{ \frac{81,5}{2} \right\} = 40,75 \text{ mm} \]

However, since this fictional beam is completely loaded under tension the effective height of this beam is assumed to be equal to the height. Therefore \( h_{\text{eff}} = 81,5 \text{ mm} \).

\[ A_{c, \text{eff}} = b \cdot h_{\text{eff}} = 300 \cdot 81,5 = 24450 \text{ mm}^2 \]

\[ \rho_{p, \text{eff}} = \frac{A_s}{A_{c, \text{eff}}} = \frac{339}{24450} = 0,0134 \]

Resulting in a maximal crack width of:

\[ s_{r, \text{max}} = 3,4 \cdot 30 + \frac{0,8 \cdot 0,5 \cdot 0,425 \cdot 12}{0,0134} = 249 \text{ mm} \]
The difference in strain of the reinforcement minus the average concrete strain between the cracks can be calculated with the following equation:

\[ \varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t f_{ct,eff} \left(1 + \alpha_s \cdot \rho_{p,eff}\right)}{E_s} \geq 0,6 \cdot \frac{\sigma_s}{E_s} \]

With:
- \( \sigma_s = 226 \, N / mm^2 \)
- \( k_t = 0,4 \)
- \( f_{ct,eff} = 2,2 \, N / mm^2 \)
- \( \rho_{p,eff} = 0,0134 \)
- \( \alpha_s = 17,5 \)
- \( E_s = 2,1 \cdot 10^5 \)

Resulting in:

\[ \varepsilon_{sm} - \varepsilon_{cm} = \frac{226 - 0,4 \cdot 2,2 (1 + 17,5 \cdot 0,0134)}{2,1 \cdot 10^5} \]

\[ \varepsilon_{sm} - \varepsilon_{cm} = 70 \cdot 10^{-5} \]

and

\[ 0,6 \cdot \frac{\sigma_s}{E_s} = 0,6 \cdot \frac{226}{2,1 \cdot 10^5} = 64 \cdot 10^{-5} \]

\[ 70 \cdot 10^{-5} > 64 \cdot 10^{-5} \]

The crack-width becomes:

\[ w_k = s_{r,\text{max}} \left( \varepsilon_{sm} - \varepsilon_{cm} \right) \]

\[ w_k = 249 \cdot 70 \cdot 10^{-5} = 0,17 \, mm \]

\[ w_{\text{max}} = 0,30 \, mm \]

\[ w_k < w_{\text{max}} \]

So the reinforcement satisfies the crack control. No different reinforcement is chosen.
Deflection wall A

The exact deflection of wall A is hard to determine. The stiffness of the wall decreases during the cracking, with an increasing deflection as result. Furthermore the loads acting on the core are not separated into semi permanently loads and characteristic loads, making it hard to distinguish the loads causing the creep factor.

A simplified (design) approach is used to calculate the total deflection at height +26 meter. The deflection is build up by:
1. The deflection of the wall due to the wind load.
2. The rotation of the foundation causing an extra deflection at the top.

It is to be expected that the deflection will be very small since the wind load acting on the wall is relatively small for the amount of concrete used. The maximal allowed (horizontal) deflection is:

\[ u_{\text{max}} = \frac{1}{150} \cdot l = \frac{26000}{150} = 173 \text{ mm} \]

1. Deflection of the wall

The deflection of the wall is approximated with the following equation:

\[ u = \frac{1}{4} \frac{M \cdot l^2}{(EI)_{\text{ref}}} \]

The difficulty of calculating the deflection is the changing stiffness of the wall due to the presence of cracks. This is simplified by taking a reduced stiffness into account. Furthermore the bending moment used, is the final bending moment caused by all acting loads in the SLS. No difference between long and short-term loads is made despite the wind load (causing the most bending moment) is a short-term loading. In reality the long-term and semi-long-term loads can be calculated using an eccentricity in combination with the normal load, causing a bending moment.

The 2nd order effect is assumed to increase the bending moment with 25% resulting in the following bending moment:

\[ M_{\text{SLS}} = 4100 \text{ kNm} \]
\[ M_{\text{SLS,extra}} = 4100 \cdot 1.25 = 5125 \text{ kNm} \]

The reduced stiffness is calculated assuming as a short-term load in a cracked section.

\[ (EI)_{x,0} = \delta_{x,0} (EI)_0 \]
\[ (EI)_{x,0} = 6 \cdot \left( \frac{d}{h} \right)^3 \cdot \left( \frac{x}{d} \right)^2 \cdot \left( 1 - \frac{1}{3} \frac{x}{d} \right) (EI)_0 \]

With:

\[ \frac{x}{d} = -\alpha_e \rho_i + \sqrt{\left( \alpha_e \rho_i \right)^2 + 2 \cdot \alpha_e \rho_i} \]
\[ \rho_i = \frac{A_s}{A_c} \]
\[ \frac{1}{4} \pi \cdot 12^2 \cdot 3 \cdot \left[ \left( \frac{2300}{150} \right) + 6 \right] \]
\[ \frac{300 \cdot 4600}{(210000 + 0.05 \cdot 1)} = 0.005 = 0.51\% \]
\[ \alpha_e = \frac{E_s}{E_{\text{cm}}} = \frac{E_s}{E_{\text{c,eff}}} = \frac{2.1 \cdot 10^5}{\left( \frac{300000}{1 + 1.5} \right)} = 17.5 \]
\[ \alpha_e \rho_i = 0.089 \]

Resulting in:

\[ \frac{x}{d} = -0.089 + \sqrt{(0.089)^2 + 2 \cdot 0.089} = 0.34 \]

Inserting this in the stiffness equation results in the following reduction of the stiffness:

\[ d = h - c - \frac{1}{2} \varnothing - \varnothing_{\text{bracket}} = 4600 - 3 - 6 - 8 = 4556 \text{ mm} \]

\[ (EI)_{x,0} = 6 \cdot \left( \frac{4556}{4600} \right)^3 \cdot (0.34)^2 \cdot \left( 1 - \frac{1}{3} \cdot 0.34 \right) (EI)_0 \]
\[ (EI)_{x,0} = 0.59 (EI)_0 \]
The stiffness of the wall is now approximated:

\[
(EI)_0 = \frac{E_{cm}}{1 + \phi(t_0)} \cdot \frac{1}{12} \cdot bh^3
\]

\[
= \frac{30 \cdot 10^3}{1 + 1,5} \cdot \frac{1}{12} \cdot 300 \cdot 4600^3 = 2,92 \cdot 10^{16} \text{Nmm}^4
\]

\[
(EI)_0 = 0,59 \cdot (EI)_0 = 0,59 \cdot 2,92 \cdot 10^{16} = 1,72 \cdot 10^{16} \text{Nmm}^4
\]

Now the deflection can be calculated:

\[
\phi_1 = \frac{1}{4} \cdot \frac{5125 \cdot 10^6 \cdot 26000^2}{1,72 \cdot 10^{16}} = 50 \text{ mm}^2
\]

2. Deflection caused by rotation in the foundation

The deflection caused by rotation in the foundation is relatively simple to approximate. In reality this factor is very hard to calculate since it is depending on the piles, the soil and the loading conditions. For the simple approach the rotation in the foundation and the horizontal deflection at the top of the core becomes:

\[
\phi_p = \frac{M \sum q^2}{k_p} = \frac{M}{C} = \frac{5125}{8,5 \cdot 10^6} = 0,0006 \text{ rad}
\]

\[
\phi_2 = \phi_1 \cdot l = 0,0006 \cdot 26000 = 15 \text{ mm}
\]

Total deflection

The total deflection can be calculated by summarizing the deflection by the foundation and the deflection of the wall:

\[
u_{tot} = u_1 + u_2 = 50 + 15 = 65 \text{ mm} < 173 \text{ mm}
\]

The horizontal deflection satisfies, the deflection is rather small. A small deflection was expected, still not this small.

Review

To review the results some checks are elaborated. The relative deflection caused by the foundation is:

\[
\frac{15}{50} \cdot 100\% = 30\%
\]

The relative deflection of the foundation shows that the stiffness possibly is correct. Furthermore the wall is modelled as an 1D element in MatrixFrame with a rotating spring. The reduced stiffness value and the spring stiffness are applied. Furthermore the normal loads and wind loads are applied. The total deflection is shown in Figure 7.11 below. A deflection of 62 mm is found, which is very similar to the calculated 65 mm.

Note: the check does not have influence on a (possibly) wrong stiffness since their equal in both the calculated as the MatrixFrame model.
Thermal expansion

A check regarding thermal expansion is elaborated. Since thermal expansion leads to extra tensile loads enough reinforcement should be applied in the section. Dilatations are difficult to apply in a (vertical) slab with risks of leaking and moisture problems. Therefore all tensile loads are distributed by the reinforcement.

1. Thermal expansion

The expansion coefficient of steel is almost equal to the expansion coefficient of concrete with a value of 0.012 mm/mK. Therefore a wall increases 0.012 mm per meter wall per degrees K.

The maximal temperature in Oslo: +30 degrees.
Minimal temperature in Oslo: -20 degrees.
Temperature of construction: +15 degrees

So the total increase of length becomes:

\[ \Delta L_{15-30} = 26 \cdot 15 \cdot 0,012 = 4,68 \text{ mm} \]
\[ \Delta L_{15-15} = 26 \cdot 30 \cdot 0,012 = 9,36 \text{ mm} \]

The compression and tensional load now can be elaborated:

\[ N = \frac{FL}{EA} \]
\[ F = \frac{N \cdot EA}{L} \]
\[ F_{\text{comp}} = \frac{9,36 \cdot 10200 \cdot 4600 \cdot 300}{26000} = 5067 \text{ kN} \]
\[ F_{\text{tensile}} = \frac{4,68 \cdot 10200 \cdot 4600 \cdot 300}{26000} = 2533 \text{ kN} \]

The compression load (due to shrinkage) should be distributed by the concrete while the tensional load is transferred by the reinforcement.

The compressional capacity of the concrete is calculated:

\[ f_{cd} = 13,3 \text{ N/mm}^2 \]
\[ N_{Rd} = f_{cd} \cdot A = 13,3 \cdot 4600 \cdot 300 = 18354 \text{ kN} \]

Results in:

\[ \frac{N_{Ed}}{N_{Rd}} = \frac{5067}{18354} = 0,27 < 1,00 \]

So the compression loads caused due shrinkage of the materials can be transferred by the concrete.

The tensional capacity of the concrete is calculated:

\[ f_{ct,eff} = 2,2 \text{ N/mm}^2 \]
\[ N_{Rd} = f_{ct,eff} \cdot A = 2,2 \cdot 4600 \cdot 300 = 3036 \text{ kN} \]

Results in:

\[ \frac{N_{Ed}}{N_{Rd}} = \frac{2533}{3039} = 0,83 < 1,00 \]

So the tensional loads caused due to expansion of the materials can be transferred by the concrete. Therefore no reinforcement check is needed. When a larger value is taken for the maximal temperature the reinforcement will be activated.

Example with a maximal temperature of 35 degrees, extra reinforcement would have been required:

\[ \Delta L_{15-35} = 26 \cdot 20 \cdot 0,012 = 6,24 \text{ mm/mK} \]
\[ F = \frac{6,24 \cdot 10200 \cdot 4600 \cdot 300}{26000} = 3378 \text{ kN} \]
\[ N_{ed} - N_{Rd} = 3378 - 3036 = 342 \text{ kN} \]
\[ A_s = \frac{N_{Ed}}{f_y} = \frac{342 \cdot 10^3}{435} = 786 \text{ mm}^2 / m \]
\[ \varnothing 12 - 125, A_s = 905 \text{ mm}^2 / m \]
2. Dehydration shrinkage

The dehydration shrinkage of concrete can be calculated with the following equation (according to Annex B of NEN-EN 1992-1-1)

\[
\varepsilon_{cd,0} = \left( 220 + 110 \cdot \alpha_{ds1} \cdot \exp \left( - \alpha_{ds2} \cdot \frac{f_{cm}}{f_{cm0}} \right) \right) \\
\cdot 0.85 \cdot 10^{-6} \cdot \beta_{RH}
\]

With:

\[
f_{cm} = 28 \, N / mm^2
\]
\[
f_{cm0} = 10 \, N / mm^2
\]
\[
\alpha_{ds1} = 4
\]
\[
\alpha_{ds2} = 0.12
\]
\[
RH = 72 - 89\%
\]
\[
RH_0 = 100\%
\]
\[
\beta_{RH} = 1.55 \left[ 1 - \left( \frac{RH}{RH_0} \right)^3 \right] = 1.55 \left[ 1 - \left( \frac{72}{100} \right)^3 \right] = 0.97
\]

Resulting in:

\[
\varepsilon_{cd,0} = \left( 220 + 110 \cdot 4 \cdot \exp \left( -0.12 \cdot \frac{28}{10} \right) \right) \\
\cdot 0.85 \cdot 10^{-6} \cdot 0.97 = 0.86 \, mm
\]

This is very small and therefore has not influence on the concrete section.
Wall B - Lintels

A design calculation of the right wall is elaborated (Figure 7.12). This wall contains several openings due to the elevator entrances. This has influences on the stiffness of the wall and the local stresses. A check is done to prove that the wall will function as one plate, instead of two separated walls. In other words: the capacity of the lintels (the vertical distance in between the openings) is checked.

Opening dimensions:
Width: 1840 mm
Height: 2300 mm

Lintel dimensions:
Width: 1840 mm
Height: 2200 mm
Thickness: 300 mm

Load acting on core A - right wall

<table>
<thead>
<tr>
<th>Level</th>
<th>ULS</th>
<th>SLS</th>
</tr>
</thead>
<tbody>
<tr>
<td>-3 - 0</td>
<td>-1,5 kN</td>
<td>-1,5 kN</td>
</tr>
<tr>
<td>0 - 7</td>
<td>11,1 kN</td>
<td>7,4 kN</td>
</tr>
<tr>
<td>7 - 17</td>
<td>17,1 kN</td>
<td>11 kN</td>
</tr>
<tr>
<td>17 - 21</td>
<td>23,8 kN</td>
<td>13,3 kN</td>
</tr>
<tr>
<td>21 - 23</td>
<td>8 kN</td>
<td>6 kN</td>
</tr>
</tbody>
</table>

Total load per wall (max): 2328 kN
Total load per wall (55% min): 1280 kN
Indirect Normal load: 12380 kN

Bending moments

The bending moment found in the foundations:
(Note: The bending moment is increased with 20%, this is a reasonable increasing for the imperfections and 2nd order effect): 6312 kNm
The bending moment at level +1500: 4464 kNm
Difference in bending moment: 1848 kNm

Modelling steps with the lintel dimensions and the vertical load acting on the fictional cantilever.
The stresses caused by the bending moment difference can be determined with:

\[ \sigma = \frac{M \cdot e}{l} \]

\[ \sigma_1 = \frac{1848 \cdot 10^6 \cdot 2190}{2.27 \cdot 10^{12}} = 1.78 \text{ N/mm}^2 \]

\[ \sigma_2 = \frac{1848 \cdot 10^6 \cdot 950}{2.27 \cdot 10^{12}} = 0.77 \text{ N/mm}^2 \]

The volume of the stress diagram then becomes:

\[ L_k = 300 \cdot 1240 \cdot \left( \frac{1.78 + 0.77}{2} \right) \cdot 10^{-3} = 474.3 \text{ kN} \]

The lintel is modelled as a cantilever beam, resulting in a bending moment of zero in the middle of the lintel. A point load is applied to the cantilever beam with the following bending moment and shear forces:

\[ M_k = 0.95 \cdot 474.3 = 450 \cdot 10^6 \text{ kNm} \]

\[ V_{Ed} = 474.3 \text{ kN} \]

The required bending moment reinforcement bars are calculated with the following equation:

\[ A_b = \frac{M_{Ed}}{f_{yd} \cdot 0.9d} \]

Used reinforcement in the lintel:

\[ \varnothing 16 \text{ mm} \]

\[ c_{nom} = 25 \text{ mm} \]

\[ \varnothing_{brackets} = 8 \text{ mm} \]

\[ d = 2200 - 25 - 8 - (16 / 2) = 2159 \]

\[ A_b = \frac{450 \cdot 10^6}{435 \cdot 0.9 \cdot 2159} = 532 \text{ mm}^2 \]

Chosen reinforcement:

3 \( \varnothing 16 \text{ mm} \)

\[ A_b = 603 \text{ mm}^2 \]

**Shear check**

The maximal allowed shear force without using the capacity of the brackets.

\[ k = 1 + \frac{200}{d} = 1 + \frac{200}{2159} = 1.30 \]

\[ v_{min} = 0.035k^{3/2} \cdot f_{ck}^{1/2} \]

\[ v_{min} = 0.035 \cdot 1.30^{3/2} \cdot 20^{1/2} = 0.23 \text{ N/mm}^2 \]

\[ v_1 = 0.6 \left( \frac{f_{ck}}{250} \right) = 0.6 \left( \frac{20}{250} \right) = 0.552 \]

The concrete compression diagonals are assumed under an angle of 21.8 degrees.

\[ v_{Rd,max} = \frac{Z}{d} \cdot v_1 \cdot f_{cd} \cdot \sin \theta \cdot \cos \theta \]

\[ v_{Rd,max} = 0.9 \cdot 0.552 \cdot 20 \cdot \sin(21.8) \cdot \cos(21.8) = 3.43 \text{ N/mm}^2 \]

\[ v_{Ed} = \frac{V_{Ed}}{b \cdot d} = \frac{474 \cdot 10^2}{300 \cdot 2159} = 0.73 \text{ N/mm}^2 \]

\[ v_{Ed} = \frac{0.73}{3.43} = 0.21 < 1.00 \]

So the dimensions of the lintels satisfy in the design calculation. This was to be expected since the lintels are relatively large. However:

\[ \frac{v_{Ed}}{v_{min}} = \frac{0.73}{0.23} = 3.17 > 1.00 \]

\[ \frac{A_{sw}}{s_w} = \frac{V_{Ed}}{f_{yed} \cdot z \cdot \cot(\theta)} \]

\[ = \frac{474.3 \cdot 10^2}{435 \cdot 0.9 \cdot 2159 \cdot \cot(21.8)} = 0.224 \text{ mm}^2 / m \]

Brackets are necessary to transfer the shear forces. The following brackets are applied:

\[ \varnothing 8 - 300(355 \text{ mm}^2 / m) \]

Now the lintel satisfies.
The Northern wall (grid line A) of the basement is checked on lateral loads caused by the soil (Figure 7.13 and 7.14). Furthermore the reinforcement is estimated. This wall is the largest wall under soil pressure, therefore this wall is taken.

Not the complete wall is checked, only the parts between the two stability walls, this is a conservative schematization since the walls will (locally) increase the stiffness of the wall since they will act as supports. The wall part which is checked has a length of 20.8 meters and a width of 400 mm. Since the depth of the basement is only just 3 meters it is assumed no extensive measures need to be taken into account in relation to the soil pressure. (Grout anchors or depth walls etc.)

In combination with the retaining wall the reinforcement of the floor is estimated.
Figure 7.14
Global detail of the calculated retaining wall. The dimensions are indicative, since the calculations may have some influence on the dimensions. Furthermore no reinforcement is drawn in the concept detail, since their was no reinforcement determined yet.
Figure 7.15
The schematization of the retaining wall and floor is described. The soil pressure results in a bending moment $M_{ad}$ in the wall. This bending moment is transferred to the floor and added to the bending moment $M_{bd}$ caused due to the floor loads. Resulting in a larger bending moment $M_{ad} + M_{bd}$. However the shear force capacity of the floor increases since extra normal loads are pressing on the floor slab. This is very beneficial when designing in concrete.

Length (calculated part) wall: 20.8 m
Wall height: 4.3 m (from bottom of basement floor to top)
Length per support: 5.2 m (grid-line)
Thickness: 400 mm
Pile foundation: one pile of 350 x 350 each 5.2 meters
Stiffness prefabricated pile: 100000 kN/m

Thickness floor: 400 mm
Floor span: 7800 mm

Concrete 30/37
Steel B 500
Durability class XS1
Concrete coverage: Building near the coast which is vulnerable for chlorides from the air and soil, 35 mm (min. req.) + 10 mm (practical) makes 45 mm.
The load action in the plane of the retaining wall

1. Overview loads

The loads working in the plane of the wall are determined (Figure 7.15). The main loads are caused by the own weight of the wall and the floor. The loads are simplified in order to make a clear calculation. For exact loads a extensive 3D model is required.

Own weight wall

\[ q_{\text{wall}} = 0.4 \cdot 4.3 \cdot 25 = 43 \text{ kN/m} \]

Own weight floor:

\[ q_{\text{floor}} = 0.5 \cdot 0.4 \cdot 7.8 \cdot 25 = 39 \text{ kN/m} \]

Weight façade (only first floor, other loads transferred by the timber trusses):

\[ q_{\text{façade}} = 3.5 \text{ kN/m} \]

Floor hofje +1500 (floor spans from truss to truss so a small part of the floor is taken by the retaining wall).

\[ q_{\text{floor,hofje}} = 0.5 \cdot 3.9 \cdot 1.5 = 3 \text{ kN/m} \]

Active soil pressure: Assuming a ground water level on -1 meter and clay soil. The soil pressure increases along depth with a dry soil stress of 16 kN/m² and saturated soil stress: 20 kN/m². The soil pressure works in all directions on a max. height of 3 meter.

\[ q_{\text{soil}} = (16 + 20 + 20) \cdot 1 = 56 \text{ kN/m} \]

Makes a total of:

\[ q_{k_{\text{wall}}} = 1.0 \cdot (43 + 39 + 3.5 + 3) + 1.0 \cdot 56 = 124.5 \text{ kN/m} \]
\[ q_{d_{\text{wall}}} = 1.5 \cdot (43 + 39 + 3.5 + 3) + 1.2 \cdot 56 = 176 \text{ kN/m} \]

Point loads caused by two timber trusses are ignored. In reality this means a local increase of reinforcement, or an increase of the concrete width since the bending moment in the plane of the wall increases. This can also be solved by introducing a stiffer foundation, this however is hard to calculate with since no soil properties are available.

---

**Figure 7.16**
Bending moment results.

**Figure 7.17**
Vertical loads results

**Figure 7.18**
Reaction forces in the spring supports
Results MatrixFrame

Max bending moment: 1136 kNm
Max vertical loads: 560 kN
Max reaction forces: 908 kN

Check if the wall can be treated as a stocky beam. If treated as stocky beam, the following rule should satisfy:

\[ l_{w} \leq 3 \cdot h \]

\[ 3 \cdot 3.9 = 11.7 \text{ m} \]

\[ l_{w} = 20.8 \text{ m} > 11.7 \text{ m} \]

So the wall can not be treated as a stocky beam.

2. Determination of the bending reinforcement

The reinforcement is based on the following equation:

\[ A_s = \frac{M_{ed}}{f_{yd} \cdot 0.9 \cdot d} \]

Two layers of reinforcement are designed, one in the upper part of the floor and one in the lower part. The center of gravity is in the middle of the floor section.

\[ d = 4300 - 400 / 2 = 4100 \text{ mm} \]

\[ A_s = \frac{1136 \cdot 10^6}{435 \cdot 0.9 \cdot 4100} = 707 \text{ mm}^2 \]

Resulting in:

\[ \frac{707}{4100 \cdot 400} = 4.3 \cdot 10^{-4} \]

The required amount of reinforcement is related to the minimal amount required. To make a safe approximation the minimal reinforcement is multiplied by 1.25 resulting in:

\[ A_s,_{\text{min}} = 707 \cdot 1.25 = 883 \text{ mm}^2 \]

The chosen tensitional reinforcement

10 Ø12 \( A_s = 1130 \text{ mm}^2 \)

The tensional reinforcement needs to be spread. Therefore the effective width of the floor is calculated.

\[ b_{eff} = b_{eff,1} + b_{eff,2} + b_w \]

The spreading width in the middle of the floor span follows from:

\[ b_{eff,2} = 0.2 \cdot b \cdot 0.1 \cdot l_0 \leq 0.2 \cdot l_0 \]

\[ l_0 = 20.8 \text{ m} \]

\[ b = 3.9 \text{ m} \]

\[ b_{eff,2} = 0.2 \cdot 3.9 + 0.1 \cdot 20.8 = 2.86 \text{ m} \]

\[ 0.2 \cdot l_0 = 0.2 \cdot 20.8 = 4.16 \text{ m} \]

\[ b_{eff,2} = 2.86 \text{ m} \]

Furthermore the effective width is build up by the thickness of the wall and a small extra slab outside of the basement box (for practical reasons). Resulting in a complete effective width of:

\[ b_{eff} \approx 0.4 + 2.86 + 0.3 = 3.5 \text{ m} \]

So the reinforcement is spread under the width of 3,5 meter. Resulting in a core-to-core distance of 350 mm.

3. Anchoring reinforcement

The loads from the floor are ‘hanged’ till above the compressional zone in the wall. Resulting in an extra reinforcement:

\[ A_{s,\text{mounting}} = \frac{F_{d}}{f_{yd}} = \frac{176 \cdot 10^3}{435} = 404 \text{ mm}^2 / \text{mm} \]

Resulting in 202 mm² in the front and in the back of the wall. This reinforcement is superimposed on the reinforcement required to transfer the shear forces in the wall.
4. Shear stress reinforcement

The shear stress acting on the section can be calculated with:
\[ \nu_{\text{Ed}} = \frac{V_{\text{Ed}}}{b \cdot d} = \frac{560 \cdot 10^3}{400 \cdot 4100} = 0.34 \text{ N} / \text{mm}^2 \]

Shear resistance of the section can due to the low amount of reinforcement be calculated with the equation:
\[ \nu_{\text{min}} = 0.035 k^{3/2} \cdot f_{ck}^{1/2} \]

With:
\[ k = 1 + \sqrt{\frac{200}{4100}} = 1.22 \]
\[ \nu_{\text{min}} = 0.035 \cdot 1.22^{3/2} \cdot 30^{1/2} = 0.25 \text{ N} / \text{mm}^2 \]
Since \( \nu_{\text{Ed}} > \nu_{\text{min}} \) shear force reinforcement should be applied. The amount of required shear force reinforcement is calculated:
\[ A_{s,w} = \frac{V_{\text{Ed}}}{z \cdot f_{ywd} \cdot \cot \theta} = \frac{560 \cdot 10^3}{0.9 \cdot 4100 \cdot 435 \cdot 2.5} = 140 \text{ mm}^2 / m \]

Resulting in 70 mm² in both sides of the wall. Adding this to the mounting reinforcement results in a total required reinforcement of:
\[ 202 + 70 = 272 \text{ mm}^2 / m \]

Chosen reinforcement in both sides of the wall becomes:
\[ \varnothing 10 - 200; \ A_{s,\text{prov}} = 392 \text{ mm}^2 / m \]
\[ A_{s,\text{prov},\text{total}} = 785 \text{ mm}^2 / m \]

 Loads out of the concrete wall plane

1. Overview of the loads.

The wall is loaded out of plane by the soil stresses (working in the vertical and horizontal direction) Resulting in a load of:
\[ q_{k,\text{soil}} = (16 + 20 + 20) \cdot 1 = 36 \text{ kN} / m \]
\[ q_{d,\text{wall}} = 1.2 \cdot 36 = 43.2 \text{ kN} / m \]

The loads acting on the floor exist out of the own weight of the floor and the loads acting on the floor. A relatively large load is chosen since the basement is used for heavy installations. For a slab of 1 meters in width:
\[ q_{\text{floor}} = 0.4 \cdot 25 \cdot 1 = 10 \text{ kN} / m \]
\[ q_{\text{installations}} = 5 \cdot 1 = 5 \text{ kN} / m \]
\[ q_{k,\text{floor}} = 10 + 5 = 15 \text{ kN} / m \]
\[ q_{d,\text{floor}} = 1.2 \cdot 10 + 1.5 \cdot 5 = 19.5 \text{ kN} / m \]

Maximal bending moment in the wall and floor:
\[ M_{\text{Ed,wall}} = \frac{1}{6} \cdot 43.2 \cdot 3.4^2 = 83.23 \text{ kNm} \]
\[ M_{\text{Ed,\text{floor, tot}}} = M_{\text{Ed,\text{wall}}} + M_{\text{Ed,\text{floor}}} \]
\[ M_{\text{Ed,\text{floor, tot}}} = \frac{1}{8} q l^2 + 83 = \frac{1}{8} \cdot 19.5 \cdot 7.8^2 + 83 = 231 \text{ kNm} \]

The maximal vertical loads acting on the wall and floor:
\[ V_{\text{Ed,wall}} = \frac{1}{2} \cdot 43.2 \cdot 3.4 \cdot \frac{2}{3} = 59 \text{ kNm} \]
\[ V_{\text{Ed,\text{floor}}} = \frac{1}{2} \cdot 19.5 \cdot 7.8 = 76 \text{ kNm} \]

The shear force of the wall acts as a compressional force on the floor. This is beneficial for the shear force capacity of the floor section.

An overview of the resulting loads is given of the following page in Figure 7.19.
2. Bending reinforcement of the wall

On the wall the following forces are working (Figure 7.19):

\[ V_{Ed} = 59 \, kN \]
\[ M_{Ed} = 83 \, kNm \]

Furthermore:

\[ d = 400 - 45 - \frac{12}{2} = 349 \, mm \]

An approximation of the required bending reinforcement is elaborated:

\[ A_s = \frac{M_{Ed}}{f_{yd} \cdot 0.9 \cdot d} = \frac{83 \cdot 10^6}{435 \cdot 0.9 \cdot 349} = 607 \, mm^2 \]

The required amount of bending steel is summarized with the required amount of mounting- and shear forces reinforcement of the outer plane of the wall. Resulting in a total required amount of:

\[ A_{s, tot} = 607 + 272 = 879 \, mm^2 / m \]

The chosen reinforcement:

\[ \varnothing 12 - 100, \ A_{s, prov} = 1130 \, mm^2 / m \]

The approximation is checked with an exact calculation. Since:

\[ N_s = N_c \]
\[ N_s = A_s \cdot f_{yd} = (1130 - 272) \cdot 435 = 373 \cdot 10^3 N \]
\[ N_c = \frac{3}{4} \chi_u \cdot f_{cd} \cdot b = \frac{3}{4} \chi_u \cdot \frac{30}{1.5} \cdot 1000 = 15000 \chi_u \]
\[ \chi_u = \frac{373 \cdot 10^3}{15000} = 25 \, mm \]

Furthermore:

\[ z = d - 0.39 \chi_u \]
\[ z = 349 - 0.39 \cdot 25 = 339 \, mm \]
\[ M_{Rd} = A_s \cdot f_{yd} \cdot z = (1130 - 272) \cdot 435 \cdot 339 = 126 \cdot 10^6 Nmm \]
\[ 126 \, kNm > 83 \, kNm \]

So the bending reinforcement satisfies.
3. Crack control wall

A simple crack control is elaborated with the charts. The steel stresses in the SLS can be calculated with the following equation:

\[
\sigma_s = \frac{q_s \cdot A_{\text{req}}}{A_{\text{prov}}} \cdot f_{yd}
\]

\[
\sigma_s = \frac{36}{43,2} \times 879 \cdot 435 = 282 \text{ N/mm}^2
\]

For environmentally class XS1, which is rather extreme, the maximal allowed crack is 0,1 mm. The maximal amount may be increased due to the large coverage which is applied.

\[
k_c = \frac{c_{\text{applied}}}{c_{\text{nom}}} \leq 2
\]

\[
k_c = \frac{45}{35} = 1,28 \leq 2
\]

\[
w_{\text{max}} = 1,28 \cdot 0,1 = 0,13 \text{ mm}
\]

Chart 7.2N and 7.3N in NEN-EN-1992-1-1 gives for the applied environmentally class:

\[
\text{for } \sigma_s = 282 \text{ N/mm}^2
\]

\[
\varnothing_{\text{max}} = 8 \text{ mm}
\]

\[
s_{\text{max}} = 50 \text{ mm}
\]

Both criteria are not met. Therefore a different reinforcement should be chosen. The distance between the bars is decreased resulting in more reinforcement and lower steel stresses.

New reinforcement:

\[
\varnothing_{12-75}, \ A_{\text{prov}} = 1508 \text{ mm}^2 / m
\]

Capacity is checked:

\[
N_s = N_c
\]

\[
N_s = A_s \cdot f_{yd} = (1508 - 272) \cdot 435 = 538 \cdot 10^3 N
\]

\[
N_c = \frac{3}{4} \chi_u \cdot f_{cd} \cdot b = \frac{3}{4} \chi_u \cdot \frac{30}{1,5} \cdot 1000 = 15000 \chi_u
\]

\[
\chi_u = \frac{538 \cdot 10^3}{15000} = 35 \text{ mm}
\]

Furthermore:

\[
z = d - 0,39 \chi_u
\]

\[
z = 349 - 0,39 \cdot 35 = 335 \text{ mm}
\]

\[
M_{\text{pl}} = A_s \cdot f_{yd} \cdot z = (1508 - 272) \cdot 435 \cdot 335 = 180 \cdot 10^6 Nm
\]

\[
180 \text{ kNm} > 83 \text{ kNm}
\]

So the bending reinforcement satisfies with ease.

Crack control of the new reinforcement with the new steel stresses:

\[
\sigma_s = \frac{36}{43,2} \times 879 \cdot 435 = 211 \text{ N/mm}^2
\]

Chart 7.2N and 7.3N in NEN-EN-1992-1-1 gives for the applied environmentally class:

\[
\text{for } \sigma_s = 211 \text{ N/mm}^2
\]

\[
\varnothing_{\text{max}} = 13 \text{ mm}
\]

\[
s_{\text{max}} = 120 \text{ mm}
\]

The new reinforcement satisfies both rules. The bending reinforcement satisfies.

4 Anchoring length (steeklengte)

The reinforcement length to connect the floor and wall to each other is calculated. The length can be calculated with the following equation:

\[
l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 \cdot f_{cd}
\]

\[
l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 \cdot \frac{\varnothing}{4} \cdot \frac{\sigma_{sw}}{f_{td}}
\]

With:

\[
\alpha_1 = 1,0
\]

\[
\alpha_2 = 1 - 0,15 \frac{(c_d - \varnothing)}{\varnothing} = 1 - 0,15 \frac{(45 - 12)}{12} = 0,59
\]

\[
c_d = 45 \text{ mm}
\]

\[
\varnothing = 12 \text{ mm}
\]

\[
\alpha_3 = 1,0
\]

\[
\alpha_5 = 1,0
\]
And:
\[ \alpha_6 = 1,5 \]
\[ f_{bd} = 2,25 \cdot \eta_1 \cdot \eta_2 \cdot f_{cd} \]
\[ \eta_1 = 1,0 \]
\[ \eta_2 = 1,0 \]
\[ f_{cd} = \frac{2,0}{1,5} = 1,3 \, N / mm^2 \]
\[ f_{bd} = 2,25 \cdot 1,3 = 2,9 \, N / mm^2 \]
\[ \sigma_{sd} = \frac{A_{s,req}}{A_{s,prov}} \cdot f_{yd} = \frac{879}{1508} \cdot 435 = 254 \, N / mm^2 \]

Resulting in the following length:
\[ l_0 = \alpha_2 \alpha_3 \alpha_5 \alpha_6 \cdot \frac{\sigma_{sd}}{f_{bd}} \]
\[ l_0 = 1,0 \cdot 0,59 \cdot 1,0 \cdot 1,5 \cdot \frac{12}{2,9} \cdot \frac{254}{1,3} = 519 \, mm \]

The anchoring length should fulfill the following requirements:
\[ \begin{cases} 200 \, mm \\ 15\varnothing = 15 \cdot 12 = 180 \, mm \\ 0,3 \cdot \alpha_6 \cdot l_{bd,req} = 0,3 \cdot 1,5 \cdot l_{bd,req} = 118 \, mm \end{cases} \]
\[ l_{bd,req} = \frac{\varnothing \cdot \sigma_{sd}}{4 \cdot f_{bd}} = \frac{12}{4} \cdot \frac{254}{2,9} = 263 \, mm \]

The anchoring length satisfies. A practical length of 530 mm is applied.

5. Shear force in the bottom of the retaining wall

The shear force acting at the bottom of the wall can be calculated in a similar way as done before:
\[ v_{min} = 0,035k^{3/2} \cdot f_{ck}^{1/2} \]

With:
\[ k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{349}} = 1,75 \]
\[ v_{min} = 0,035 \cdot 1,75^{3/2} \cdot 30^{1/2} = 0,44 \, N / mm^2 \]

Furthermore:
\[ V_{Ed} = 59 \, kN \]
\[ v_{Ed} = \frac{V_{Ed}}{b \cdot d} = \frac{59 \cdot 10^3}{1000 \cdot 349} = 0,16 \, N / mm^2 \]

Since \( v_{Ed} < v_{min} \) no additional shear force reinforcement has to be applied.

Reinforcement of the basement floor

1. Overview of the loads

Per m\(^1\) floor width the following forces apply:
\[ M_{Ed,max} = 231 \, kNm \]
\[ M_{Ed,corner} = 83 \, kNm \]
\[ V_{Ed} = 76 \, kN \]
\[ N_{Ed} = 59 \, kN \]

For the floor an equal coverage of 45 mm is applied. The possibility of the structure being in the XS1 class is taken into account with this coverage. The coverage of the upper reinforcement may be decreased, but for now this is not applied.

\[ c_{nom} \geq c_{min,dur} + 10 \, mm \]
\[ c_{min,dur} = 35 + 10 = 45 \, mm \]
\[ d = 400 - 45 - \varnothing 12 \cdot 2 = 349 \, mm \]

1.1 Bending reinforcement in the floor (max)

The maximal bending moment is used to calculate the required bending reinforcement in the floor.
\[ A_s = \frac{M_{Ed}}{f_{yd} \cdot 0,9 \cdot d} = \frac{231 \cdot 10^6}{435 \cdot 0,9 \cdot 349} = 1690 \, mm^2 \]

Chosen reinforcement:
\[ \varnothing 16 - 100, \, A_{s,prov} = 2010 \, mm^2 / m \]
Reinforcement check:

\[
N_s = N_c \\
N_s = A_x \cdot f_{yd} = 2010 \cdot 435 = 875 \cdot 10^3 N \\
N_c = \frac{3}{4} \chi_u \cdot f_{cd} \cdot b = \frac{3}{4} \chi_u \cdot \frac{30}{1,5} \cdot 1000 = 15000 \chi_u \\
\chi_u = 875 \cdot 10^3 = 58 \text{ mm} \\
\]

Furthermore:

\[
z = d - 0,39 \chi_u \\
z = 349 - 0,39 \cdot 58 = 334 \text{ mm} \\
M_{rd} = A_x \cdot f_{yd} \cdot z = 2010 \cdot 435 \cdot 334 = 292 \cdot 10^6 Nmm \\
292 \text{ kNm} > 231 \text{ kNm} \\
\]

So the bending reinforcement satisfies.

1.2 Bending reinforcement in the floor (corner)

\[
A_x = \frac{M_{rd}}{f_{yd} \cdot 0,9 \cdot d} = \frac{83 \cdot 10^6}{435 \cdot 0,9 \cdot 349} = 608 \text{ mm}^2 \\
\]

Chosen reinforcement:

\( \varnothing 10 - 100, A_{s,prov} = 785 \text{ mm}^2 / m \)

Reinforcement check:

\[
N_s = N_c \\
N_s = A_x \cdot f_{yd} = 785 \cdot 435 = 342 \cdot 10^3 N \\
N_c = \frac{3}{4} \chi_u \cdot f_{cd} \cdot b = \frac{3}{4} \chi_u \cdot \frac{30}{1,5} \cdot 1000 = 15000 \chi_u \\
\chi_u = \frac{342 \cdot 10^3}{15000} = 23 \text{ mm} \\
\]

Furthermore:

\[
z = d - 0,39 \chi_u \\
z = 349 - 0,39 \cdot 23 = 340 \text{ mm} \\
M_{rd} = A_x \cdot f_{yd} \cdot z = 785 \cdot 435 \cdot 340 = 116 \cdot 10^6 Nmm \\
116 \text{ kNm} > 83 \text{ kNm} \\
\]

So the bending reinforcement satisfies.

2. Crack control floor

The steel stresses in middle of the floor span (maximal bending moment).

\[
\sigma_s = \frac{15}{19,5} \cdot \frac{1690}{2010} \cdot 435 = 281 \text{ N / mm}^2 \\
\]

For environmentally class XS1, which is rather extreme, the maximal allowed crack is 0,1 mm. The maximal amount may be increased due to the large coverage which is applied.

\[
k_x = \frac{c_{applied}}{c_{nom}} \leq 2 \\
k_x = \frac{45}{35} = 1,28 \leq 2 \\
w_{max} = 1,28 \cdot 0,1 = 0,13 \text{ mm} \\
\]

Chart 7.2N and 7.3N in NEN-EN-1992-1-1 gives for the applied environmentally class:

\[
\text{for } \sigma_s = 281 \text{ N / mm}^2 \\
\varnothing_{max} = 8 \text{ mm} \\
s_{max} = 50 \text{ mm} \\
\]

Both criteria are not met. Therefore a different reinforcement should be chosen. The distance between the bars is decreased resulting in more reinforcement and therefore lower steel stresses.

New reinforcement:

\( \varnothing 16 - 75, A_{s,prov} = 2680 \text{ mm}^2 / m \)

Resulting in the following steel stress in the SLS:

\[
\sigma_s = \frac{15}{19,5} \cdot \frac{1690}{2680} \cdot 435 = 211 \text{ N / mm}^2 \\
\]

Chart 7.2N and 7.3N in NEN-EN-1992-1-1 gives for the applied environmentally class:

\[
\text{for } \sigma_s = 211 \text{ N / mm}^2 \\
\varnothing_{max} = 15 \text{ mm} \\
s_{max} = 90 \text{ mm} \\
\]
The maximal re-bar diameter requirement is not met, however the maximal bar distance requirement satisfies. Therefore the reinforcement satisfies.

The crack control in the corner near the wall is checked since less steel is applied in this area.

\[ \sigma_s = \frac{15}{19.5} \cdot \frac{608}{785} \cdot 435 = 260 \text{ N/mm}^2 \]

Chart 7.2N and 7.3N in NEN-EN-1992-1-1 gives for the applied environmentally class:

- for \( \sigma_s = 260 \text{ N/mm}^2 \)
- \( \varnothing_{\text{max}} = 10 \text{ mm} \)
- \( s_{\text{max}} = 75 \text{ mm} \)

So the maximal allowed re-bar diameter rule is met. The reinforcement satisfies.

### 3. Shear stress reinforcement

A one meter wide slab is taken to calculate the shear stresses.

\[ V_{Ed} = 76 \text{ kN} \]
\[ N_{Ed} = 59 \text{ kN} \]
\[ d = 400 - 45 - \frac{\varnothing_{16}}{2} = 347 \text{ mm} \]

The concrete shear resistance can be calculated with:

\[ \nu_{\text{min}} = 0.035k^{3/2} \cdot f_{ck}^{1/2} \]

With:

\[ k = 1 + \frac{200}{d} = 1 + \frac{200}{347} = 1.76 \]
\[ \nu_{\text{min}} = 0.035 \cdot 1.76^{3/2} \cdot 30^{1/2} = 0.45 \text{ N/mm}^2 \]

However, due to compressional forces acting on the floor (caused by the soil pressure on the wall) the concrete shear resistance is increased by the following expression:

\[ \nu_{\text{rd,c,increased}} = \nu_{\text{rd,c}} + k_1 \cdot \sigma_{cp} = \nu_{\text{rd,c}} + 0.15 \cdot \frac{N_{Ed}}{A_c} \]

Furthermore:

\[ V_{Ed} = \frac{V_{Ed}}{b \cdot d} = \frac{76 \cdot 10^3}{1000 \cdot 349} = 0.21 \text{ N/mm}^2 \]

Since \( V_{Ed} < \nu_{\text{min}} \) no additional shear force reinforcement has to be applied. This seems peculiar since a relatively large load is applied to the basement floor. Another option is to decrease the thickness of the basement floor, resulting in larger stresses but a decrease of concrete use.

**Chosen reinforcement**

All required reinforcements are known (Figure 7.20). The reinforcement in the floor is reduced at the top since the steel (tension) stresses in that area are relatively low. In practice the reinforcement of the floor may be reduced even more since the ground water is creating an uplifting force which is not taken into account for the current calculation.

**Deformations**

The basement floor is not checked on deformations, since large deformations are not to be expected. The floor is relatively thick for the span (400 mm for a 7.8 meter span). The reason for the thickness of the floor has to do with the durability class of the concrete near the sea and not with the loads.
Figure 7.20
Applied reinforcement in the basement floor and retaining wall. The pile foundation with attachment reinforcement is not calculated and therefore drawn with dashed lines.
A proposal is made for the foundation (Figure 7.21). Although the foundation is not checked (due to missing parameters) a concept design is given. A very clear pile system is designed with footings (poeren) under the cores and concrete strips under the retaining walls. The footings are designed according to CB2\textsuperscript{7.2} and CB4\textsuperscript{7.1} and used to calculate the deflection of the core. The strips connecting all elements are cast in-situ with an arbitrary dimension of 500x400 mm and supported by 350x350 mm prefabricated piles, one each 5.2 meters at least. Calculations may conclude that more piles are necessary at the supports for the trusses. The foundation under the cores are much stiffer than the strips, combining with the higher loads should result in an (almost) equal vertical deflection.

**Figure 7.21**
Floor plan Foundation
Scale 1:200
Parking garage

The elaboration of the structural design part of the project focuses on the embassy alone. However some remarks are given about the parking garage (Figure 7.22), which is designed to be very efficient and act as a solid foundation of the buildings. The parking garage, hosting 180 parking places, forms the basis for the business concept. The embassy should be profitable, visible and giving something back to Oslo. A public parking place at the Dutch embassy has all mentioned elements.

The parking garage is built up by retaining walls, supported by a pile foundation. Since the maximal depth of the parking garage is 3 meters below ground level it is easy to excavate the basement, afterwards a pile foundation is installed and topped with special concrete (Betoniet), resulting in a basement work floor. Afterwards the water of the basement is pumped out and a concrete cover floor is applied, the parking garage floor is then (globally) finished.

The concrete column structure (400x400 mm) of the parking garage correspond with the building volumes on top of the garage. Resulting in a clear load distribution from roof to foundation. Concrete T beams form the roof of the parking garage, these beams span a maximum length of 15.6 meters and are able to carry large loads with a reduced height. Furthermore it creates the possibility to use the public square for celebrations and large gatherings with large loads as a results. No calculations are made, it is however a reasonable approach referring to Cement

Structural grid

The structural grid of the whole plot is developed after the desired dimensions for the parking garage. Every parking place has a width of 2,6 meter and a depth of 5 meters. In combination with a driving lane of 5,6, divided by 2 results in 2,8 meters wide it results in a structural grid of 7,8 meters by 5,2 meters wide. This perfectly fits the 'portrait' orientation of the plot. In the embassy a refinement is added to correspond better, the structural grid is divided in the north-south direction resulting in a structural grid of 3,9 by 5,2 meters.

Figure 7.22
Parking garage with 180 parking lots. The structural grid is developed after beneficial parking lot dimensions.
Conclusion
Isometric rendering

Conclusion
Figure 8.1
Rendered isometric drawing showing the elegance of the timber structural system.
Figure 8.2
Embassy at night
A conclusion is hard to make in a project like this since the design itself is the conclusion. However it is important to mention some elements considering the integral approach of the project.

The elaboration of the structure did not lead to major design changes, this proves the integral process in which the correct structures and dimension where always included in the architecture. An exception is the roof package, which was approximated with Dutch snow loads instead of Norwegian snow loads. However de design did not (visually) change much.

The project may be called a successful integral design since structure and architecture cannot be separated without losing the identity of the building. The most iconic part of the design is without any doubt the timber truss system (Figure 8.1). The structure makes an architec tonic statement, translating a concept to a visual strong element, giving the building its identity. From a structural point of view the trusses make it possible to keep the structural system readable. After calculating the trusses and connections it became clear that the dimensions would not change and that the connections are rather clean, which adds to the design.

Although the entire structure makes a complex whole, the structure is very much suitable for splitting in smaller, easier to calculate, parts. In reality the building may be stronger and stiffer than calculated since the system starts to work together as a whole. The building is relative sustainable due to the material usage, this is very visible, though subtle in some way. When designing the connections sustainability is reached since one connection is suitable for all trusses, creating a more efficient structure.

After checking some characteristic elements of the design it is not clear if the building is ‘buildable’, it is however much likely that it is possible. The real answer can only be given by a total 3D analyses, taking at least some months with an experienced structural designers team. As stated earlier this was not the goal of this thesis.
References
Literature & images

Literature used for the architectural part can be retrieved from ‘Stacked Dutch’ Architectural thesis.

1

3

4


5


7


7.3: Cement. (2000). *Cement nr 6, Parkeergarages*


All images in this thesis are own work unless mentioned here.

3
Figure 3.4: NEN-EN 1991-1: Belastingen op constructies (2007). *Noorwegen: sneeuwbelasting op de grond, Figure C.10*

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Figure 5.32: NEN-EN 1993-1: Staalconstructies (2007).
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