Saimaa University of Applied Sciences
Technology, Lappeenranta
Double Degree Programme in Civil and Construction Engineering
Civil Engineering

Filipp Popov

Design of prefabricated steel structures equipped with a jib crane for auxiliary purposes

Bachelor’s Thesis 2019

Abstract

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Instructors: Petri Himmi, senior lecturer, Saimaa University of Applied Sciences, Mika Myllys, PrePipe Oy

The aim of this work was to test the possibility of developing a quick-assembled steel structure that uses shipping containers as a base and it equipped with two diagonally located cranes for production needs. The demand for such designs is high due to the ability to mobilize production and quickly deploy it in any location. However, the market does not have all the available options of such construction designs, therefore, in this work, a variant of such a design will be developed, which aims to cover all the strength characteristics to be used in a large list of locations in Finland.

The construction of the building is designed in accordance with the Eurocode.

Keywords: steel construction, fast-assembling, jib crane, limit state design method, light-weight steel construction, structural analysis.

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1 Introduction

The main aim of this thesis work is to develop a design that would satisfy production tasks as well as Eurocode norms and standards. This thesis contains full stages of the steel structures design from the preliminary concept of the project to the detailed drawings that are necessary for the preparation of materials in the factories, and then for the erection of the construction on site. One of the main requirements for such kind of construction is quick assembling. That means, this project excludes welded joints carried out at the installation stage, as well as work associated with concrete elements. All connections are provided with bolts. All shipping marks correspond to the overall dimensions of transportation.

Load calculations and structural analysis methods were provided in accordance with the Eurocode.In addition, this thesis considered modern methods of building design related to the use of special software equipment, which can significantly simplify the designing process.

2 Theoretical part

The main goal of the theoretical part is to briefly describe types of structural solutions that are possible to use during the design process. For the project we could use three principal schemes that are described below:

1) Spatial framework using a hinged truss as a roof support structure.
2) Spatial framework using a composite beam as a roof support structure, (This beam is divided into two shipping marks, which are connected by means of a flange connection form a duo-pitch roof frame).
3) Spatial framework using a rigidly connected beam that formed a support structure for the mono-pitch roof envelope.

Constructions described above are the most common when implementing similar projects. Based on the features of our terms of the designed task, from the list of solutions presented, we would choose the most suitable for our case.

Since trusses are more suitable for long-span structures using such kind of a constructive solution would lead to the unreasonable increasing in the project's steel consumption and the weight of the structure would be significantly exaggerated too. Thus, using a truss as a roof supporting structure is not the best solution. Also, using a composite beam may cause a few problems with on-site flange connection implementation because such type of connection requires an accurate fit of the surfaces to be joined. This way, a scheme with a mono-pitch roof construction solution would be the most suitable to meet the requirements of the project (fast-assemblying, cost-effective, easy to implement).

Previously, the supporting elements will be made of prefabricated structures (twist-locks) while the base of the columns will be connected by a traverse to simplify installation and transportation (also vertical ties which unfastening the columns from the plane and preventing a general loss of stability will be welded to the structure at the factory too). Thus, column, traverse and vertical ties would form a truss. This way it is necessary to avoid the appearance of a load outside their transmission points to avoid the occurrence of additional bending moments.

For this, the fastening of the roof beams will be carried out through the nodes located above the columns.

The connection will be carried out using bolts for correct separation at the shipping marks. Also, it is necessary to remember the accuracy of assembly of this structure and provide oversized holes for bolts.

3 Preliminary design stage

On this stage, the overall dimensions of the structure are determined, structural solutions are considered, and design schemes are drawn up for future structural analysis. In the future process of designing constructional solutions may change. The main task of this stage is to determine the purpose and main dimensions of the structure, draw up a preliminary model intended for further calculation.

In our case we will use the 3D AutoCAD software to draw the calculation scheme and export the DXF file that will be used in structural analysis software to calculate the load combinations.

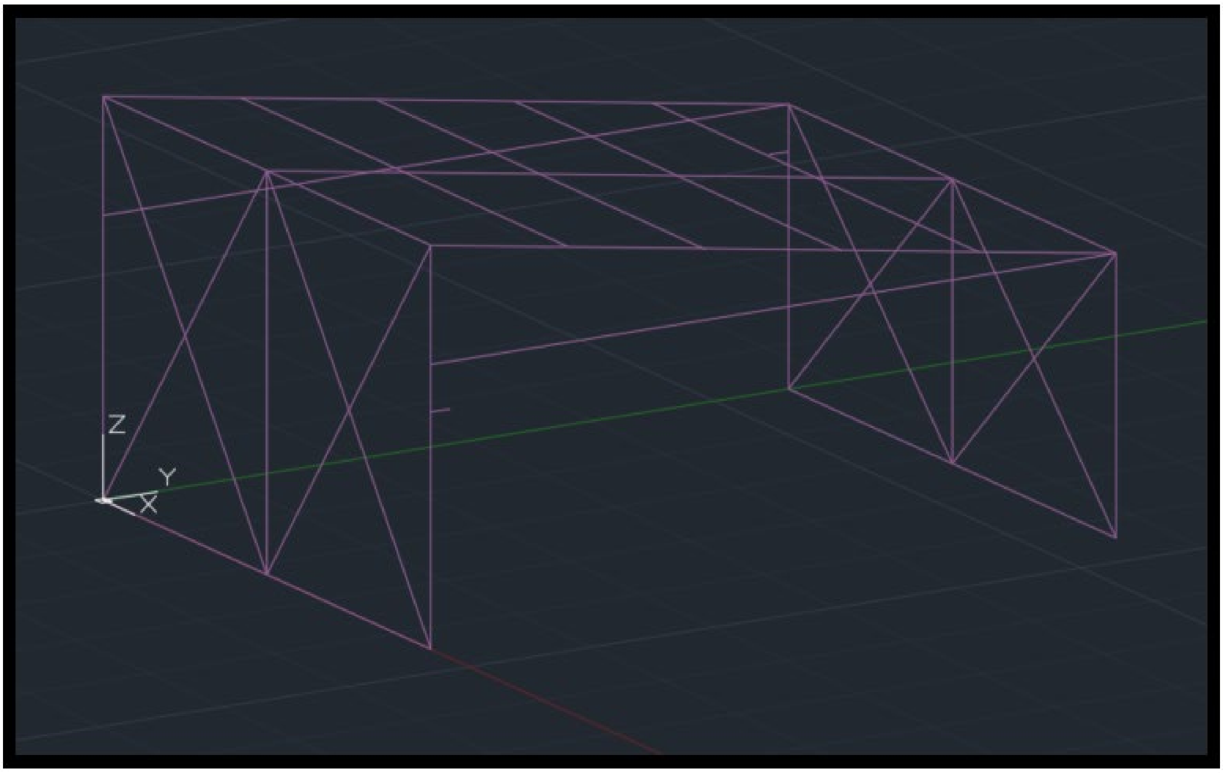


Figure 3.1 Design scheme (Autodesk AutoCAD student's version)

Thus, the main parameters of the future construction:

1. Length in lay-out: 6058 mm (including the 20ft shipping container that used as basement), 5600 mm (frame of the construction).
2. Width in lay-out: 11868 mm (including the 20ft shipping container that used as basement), 7000 mm (frame of the construction).
3. Height: 5990 (including the 20ft shipping container that used as basement), 3400 mm (frame of the construction).

4. Roof slope angle: 8°.
5. Column pitch: 2800 mm.
6. Bulk pitch: 1415 mm.

These dimensions are given as reference. In the process of calculation, they can vary slightly, if the technical specifications allow and there is no restriction on dimensions. In our case, the height of the structure is not limited (however, excessive overestimation or understatement will lead to irrational consumption of material and complications during transportation). Also, it should be remembered that the structure is equipped with cranes that must work independently and not complicate the procedure of unloading or loading materials onto vehicles. The length and width of the structure is limited by the size of the shipping marks and the size of the shipping containers that are used as the base.

Thus, the design scheme represents the dimensions of the designed structure with slight deviations that are considered in the calculation.

Since the enclosing structure will be a profiled flooring that requires fastening to the structure through self-tapping screws, secondary coating beams should be located with the frequency with which it will be easy to mount the profiled flooring. Secondary beams also play the role of unfastening elements, preventing the loss of stability of the main roof beams and twisting of the structure at all.

4 Loads evaluation

4.1 Self-weight of the construction

Dead load of the columns and truss elements is calculated using Robot Structural Analysis software, in accordance with the designated sections of the elements of the frame, columns. The cross-sectional dimensions are determined by their geometric characteristics, as well as based on design experience.

Calculation of the dead load of the roof covering structure is given below:

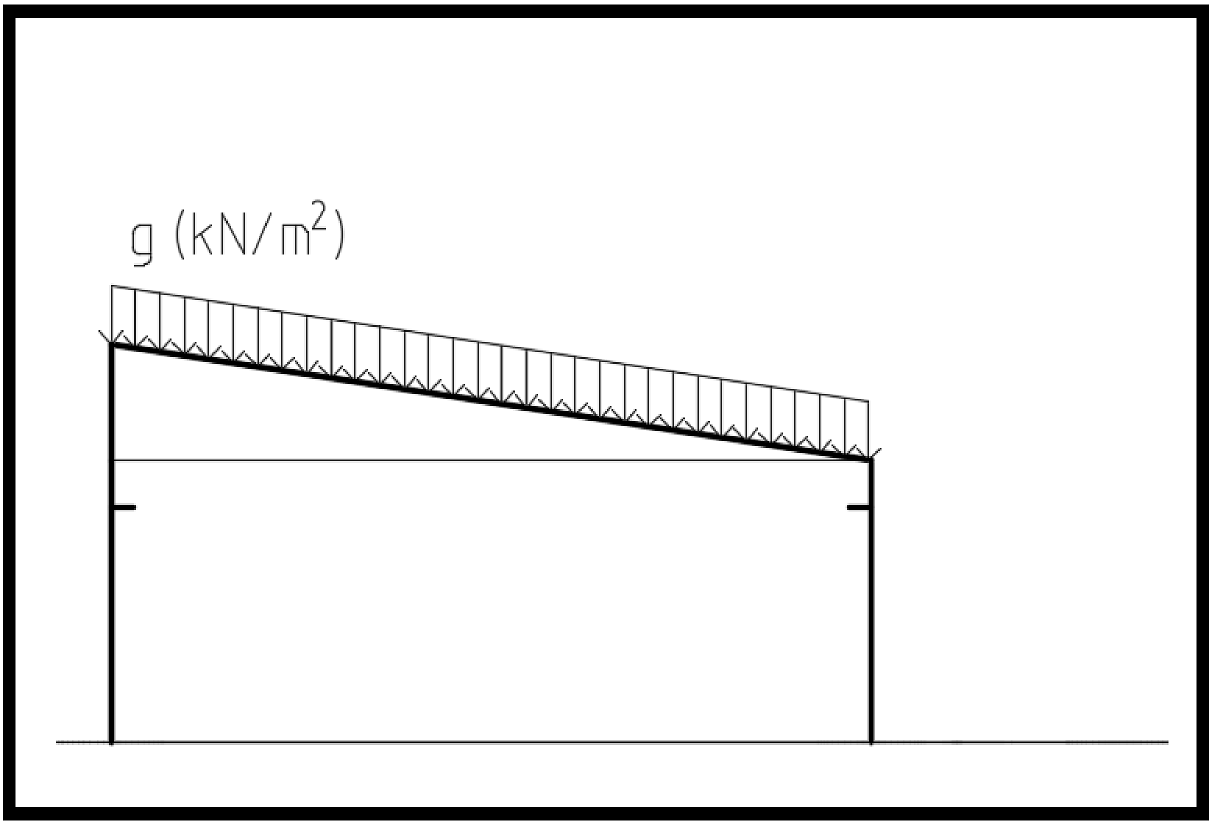


Figure 4.1.1 Load distribution by dead weight (Autodesk AutoCAD student's version drawing)

Distributed over area load occurred by dead load of the roof envelope and snow could be represented as distributed over the element's (horizontal supporting beam in our case) length.

The roof envelope structure is implemented with Ruukki load bearing sheet T45-30L-905 with 0,7 mm thickness and nominal weight of the m^2 — 7,59 kg (0,076 kN/ m^2). This

is not a final decision because the final version of the coverage will be selected from the condition of checking for limit states when a load (calculated further) is applied to it.
Thus, load distributed over the length of the beam :

$$G_k = g_k \cdot a = 0{,}076\ \frac{kN}{m^2} \cdot 1{,}414\ m = 0{,}107\ kN/m \qquad (4.1.1)$$

Where:

g – distributed dead load, kN/m^2;

a – influence area dimention, m.

As for the edge elements, for them distributed over length load would be two time less because of the smaller influence area.

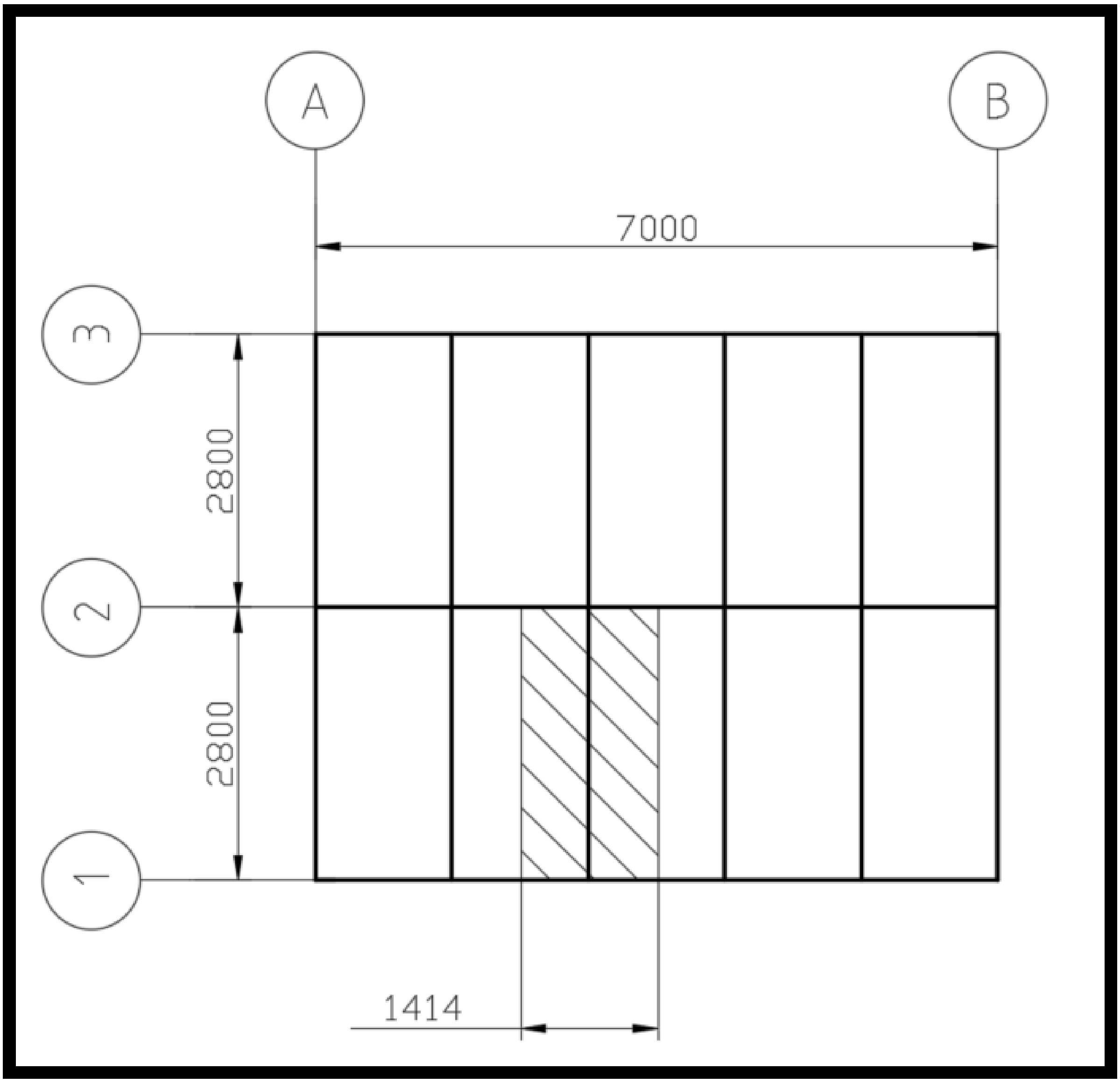


Figure 4.1.2 Influence area of one beam (Autodesk AutoCAD student's version drawing)

4.2 Snow load

Snow load calculation is provided in accordance with EN 1991-1-3 [1] and Finnish National Annex 4 [6].

In this calculation one snow distribution scheme is determined because of the construction of the mono-pitch roof.

Snow load calculated by using formula:

$$S = \mu_i C_e C_t S_k \tag{4.2.1}$$

Where μ_i – is the snow load shape coefficient;

C_e – is the exposure coefficient;

C_t – is the thermal coefficient;

S_k – is the characteristic value of snow load on tile ground.

In our case, the value S_k would be taken as the biggest value between several districts around Lappenranta city location because of the construction's mobility. Thus, the S_k value would be 2.75 kN/m^2 in accordance with Finnish National Annex 4 [6].

This value of the snow load is optimal, since it will not lead to excessive metal consumption in view of the irrational choice of the material section when using excessive snow load. However, in view of the small size of the projected object, it is recommended to avoid the formation of significant snow formations on the roof structure.

The exposure coefficient and thermal factor could be taken: $C_e = 1$, $C_t = 1$.

In our case, it does not involve the use of special heating equipment that can significantly affect the melting of snow in the roof area, which will lead to a decrease in the snow load. As for the snow transfer coefficient, its value is taken as the maximum in view of the formation of the reserve of bearing capacity during the operation of the structure

$\mu_1 = 0{,}8$ – for the slope angle 8° due to the table 4.2.1

This angle of inclination is selected based on considerations of economic efficiency of material consumption as well as the need to form the proper slope to remove precipitation from the roof structure

Table 4.2.1: Snow load shape coefficient (SFS-EN 1991-1-3. Table 5.2)

Angle of pitch of roof α	$0° \leq \alpha \leq 30°$	$30° < \alpha < 60°$	$\alpha \geq 60°$
μ_1	0,8	$0,8(60 - \alpha)/30$	0,0
μ_2	$0,8 + 0,8\ \alpha/30$	1,6	--

Snow load evaluation:

$S = 0{,}8 \cdot 1 \cdot 1 \cdot 2{,}75 = 2{,}2\ kN/m^2$

This way, the length distributed value of the snow load would be:

$S = 2{,}2\ kN/m^2 \cdot 1{,}414\ m \cdot \cos{(8°)} = 3{,}08\ kN/m$

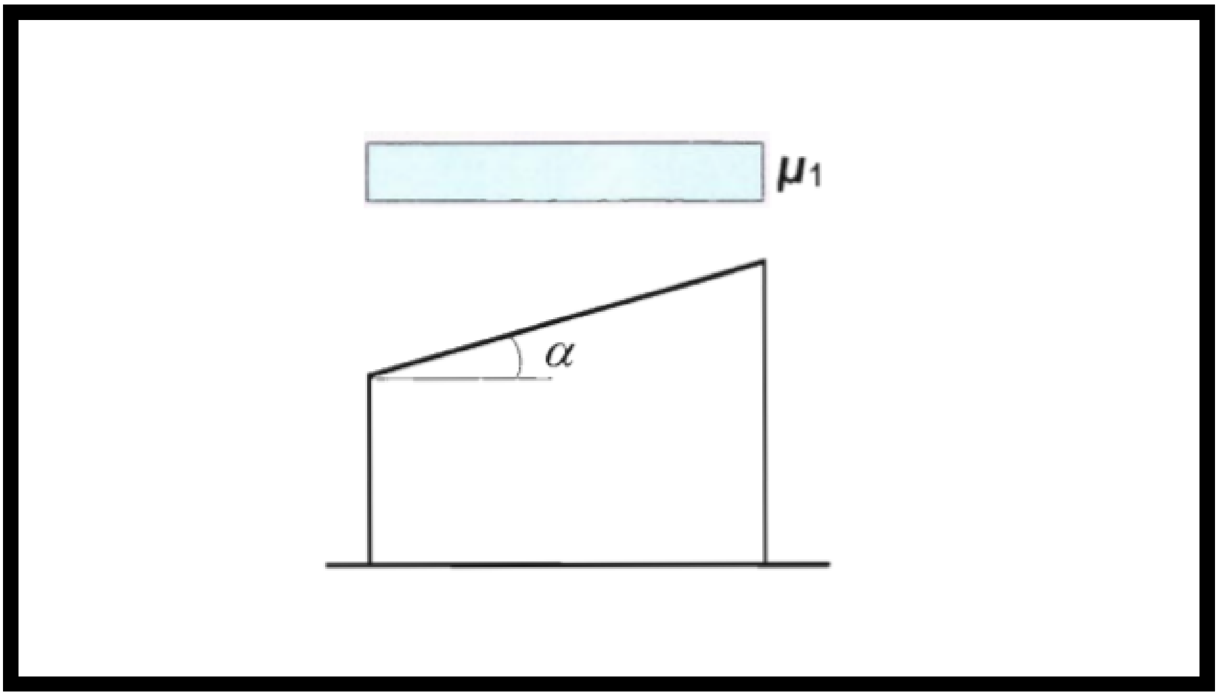


Figure 4.2.1 Snow load shape coefficients (SFS-EN 1991-1-3. Figure 5.2)

4.3 Wind load

Determination of the basic wind velocity:

$$v_b = C_{dir} \cdot C_{season} \cdot v_{b,0} \quad (4.3.1)$$

Where:

v_b – basic wind velocity;

C_{dir} – directional factor;

C_{season} – seasonal factor;

$v_{b,0}$ – fundamental falue of the wind velocity.

In our case:

$v_{b,0} = 0{,}21\ m/s$ in accordance with Finnish National Annex 5 [7].

Terrain category: III

In our case, this type of terrain was selected on the basis of the assumption that the designed structure will be used mainly for production purposes (meaning the prevailing location of such a building is the enclosed territory of a construction or industrial site). Consequently, the proposed type of terrain will exclude direct wind exposure typical of a completely open or coastal terrain, since buildings of this purpose involve the use of fencing required by safety considerations for built-up areas and areas of industrial buildings.

z_0 = 3;

z_{min} = 5 in accordance with EN 1991-1-4 [2];

C_{dir} = 1 as recommended in EN 1991-1-4 [2];

C_{season} = 1 as defined in 4.2 (2) EN 1991-1-4 [2].

$v_b = 1 \cdot 1 \cdot 21 = 21\ m/s$

Determination of the basic velocity pressure:

$$q_b = 0{,}5 \cdot \rho \cdot {v_b}^2 \tag{4.3.2}$$

Where:

q_b – basic velocity pressure.

v_b – basic wind velocity;

ρ – density of the air (1,25 kg/m^3 as recommended in Finnish National Annex 5 [7]);

$q_b = 0{,}5 \cdot 1{,}25 \cdot 21^2 = 0{,}28\ kN/m^2$

Determination of the peak velocity pressure:

$$q_p = [1 + 7l_v] \cdot 0{,}5 \cdot \rho \cdot v_m^2 \tag{4.3.3}$$

Where:

q_b – peak velocity pressure.

v_m – mean wind velocity;

ρ – density of the air;

l_v – turbulence intensity.

Calculation of the mean velocity pressure:

$$v_m = c_r \cdot c \cdot v_b \tag{4.3.4}$$

Where:

c_r – roughness factor;

c_o – orography factor $(\mathrm{c_o} = 1 \text{ in our case})$.

$$c_r = k_T \cdot \ln\left(\frac{z}{z_0}\right) \tag{4.3.5}$$

Where:

k_T – terrain factor, depending on the roughness length.

Calculation of the mean velocity pressure:

$$k_T = 0{,}19 \cdot \ln\left(\left(\frac{z_0}{z_{0,II}}\right)^{0{,}07}\right) \tag{4.3.6}$$

Where:

$z_{0,II}$ = 0,05 (terrain category II);

z_0 = 0,3 (terrain category III).

Calculation of the turbulence intensity:

$$l_v = \frac{k_I}{C_0 \cdot ln\left(\frac{z}{z_0}\right)} \tag{4.3.7}$$

Where:

k_I – turbulence factor (k_I = 1, as recommended in EN 1991-1-4 [2]);

z = 6,00 m in our case.

Thus,

$$q_p = \left(1 + \frac{7k_I}{c_0 \cdot ln\left(\frac{z}{z_0}\right)}\right) \cdot q_b \cdot \left(0{,}19 \cdot \left(\frac{z_0}{z_{0,II}}\right)^{0{,}07}\right) \cdot ln\left(\frac{z}{z_0}\right)\Bigg) =$$

$$= \left(1 + \frac{7 \cdot 1}{1 \cdot ln\left(\frac{6}{0{,}3}\right)}\right) \cdot 280 \cdot \left(0{,}19 \cdot \left(\frac{0{,}3}{0{,}05}\right)^{0{,}07}\right) \cdot ln\left(\frac{0{,}3}{0{,}05}\right)\Bigg) = 0{,}946 \text{ kN/m2};$$

External pressure coefficients:

$$w_e = q_p(z_e) \cdot c_{pe} \tag{4.3.8}$$

Where:

z_e – reference height of the external pressure;

c_{pe} - pressure coefficient for the external pressure depending on the size of the loaded area (in our case $c_{pe,10}$, because the loaded area for the structure is larger than 10m^2).

For vertical walls:

h ≤ b;

e = min(b,2h) = min (5,6; 6) = 5,6;

e < d = 7 m (11 m);

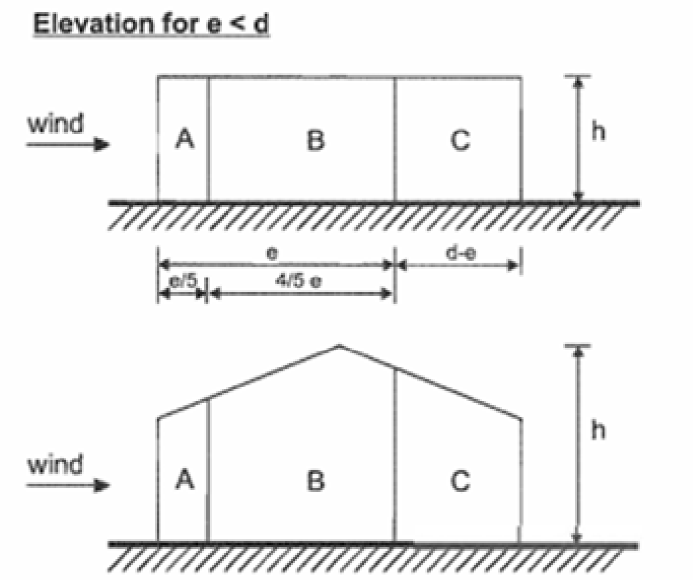


Figure 4.3.1 Key for vertical loads (SFS-EN 1991-1-4. Figure 7.5)

Plan view:

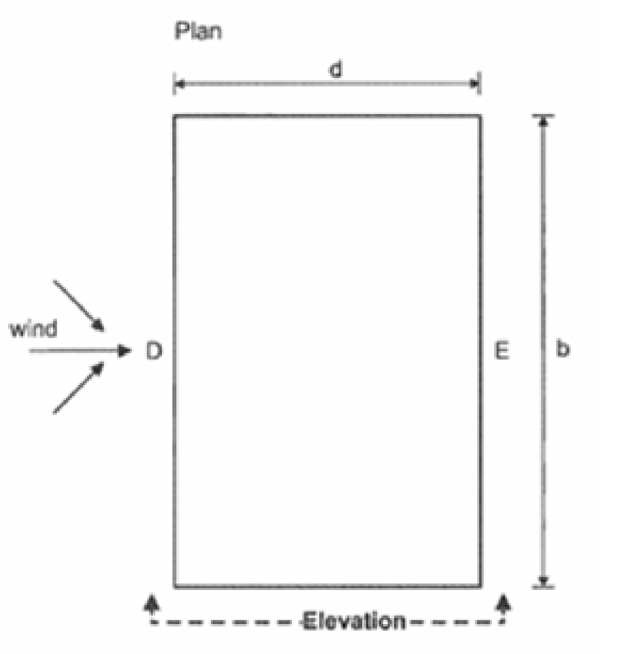


Figure 4.3.2 Zone plan for the vertical wall loads definition
(SFS-EN 1991-1-4 Figure 7.5)

To determine pressure coefficents in each zone it is necessery to know the h/d value. Coefficients value is given in the table:

Table 4.3.1 Recommended values of external pressure coefficients for vertical walls of rectangular plan buildings (SFS-EN 1991-1-4 Table 7.1)

Zone	A		B		C		D		E	
h/d	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,7	
1	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,5	
≤ 0,25	-1,2	-1,4	-0,8	-1,1	-0,5		+0,7	+1,0	-0,3	

In our case:

h/d = 6,00/7,00 = 0,86

Thus, the following coefficients $c_{pe,10}$ should be chosen:

Zone A: -1,2

Zone B: -0,8

Zone C: -0,6

Zone D: +0,8

Zone E: -0,5

For monopitch roofs:

In accordance with our design scheme pitch angle = 8°, such case is not shown in the table below. Thus, as recommended linear interpolation method used.

When wind direction angle θ = 0°:

e = min(b;2h), where b – is crosswind dimention.

In our case e = min(5,60 m; 12,00 m) = 5,60 m.

The following coefficients should be chosen:

Zone F: -1,46; 0,06;

Zone G: -0,8; 0,06;

Zone H: -0,3; 0,06.

When wind direction angle θ = 180°:

e = min(b;2h), where b – is crosswind dimention.

In our case e = min(7,00 m; 12,00 m) = 7,00 m.

The following coefficients should be chosen:

Zone F: -2,36;

Zone G: -1,3;

Zone H: -0,83

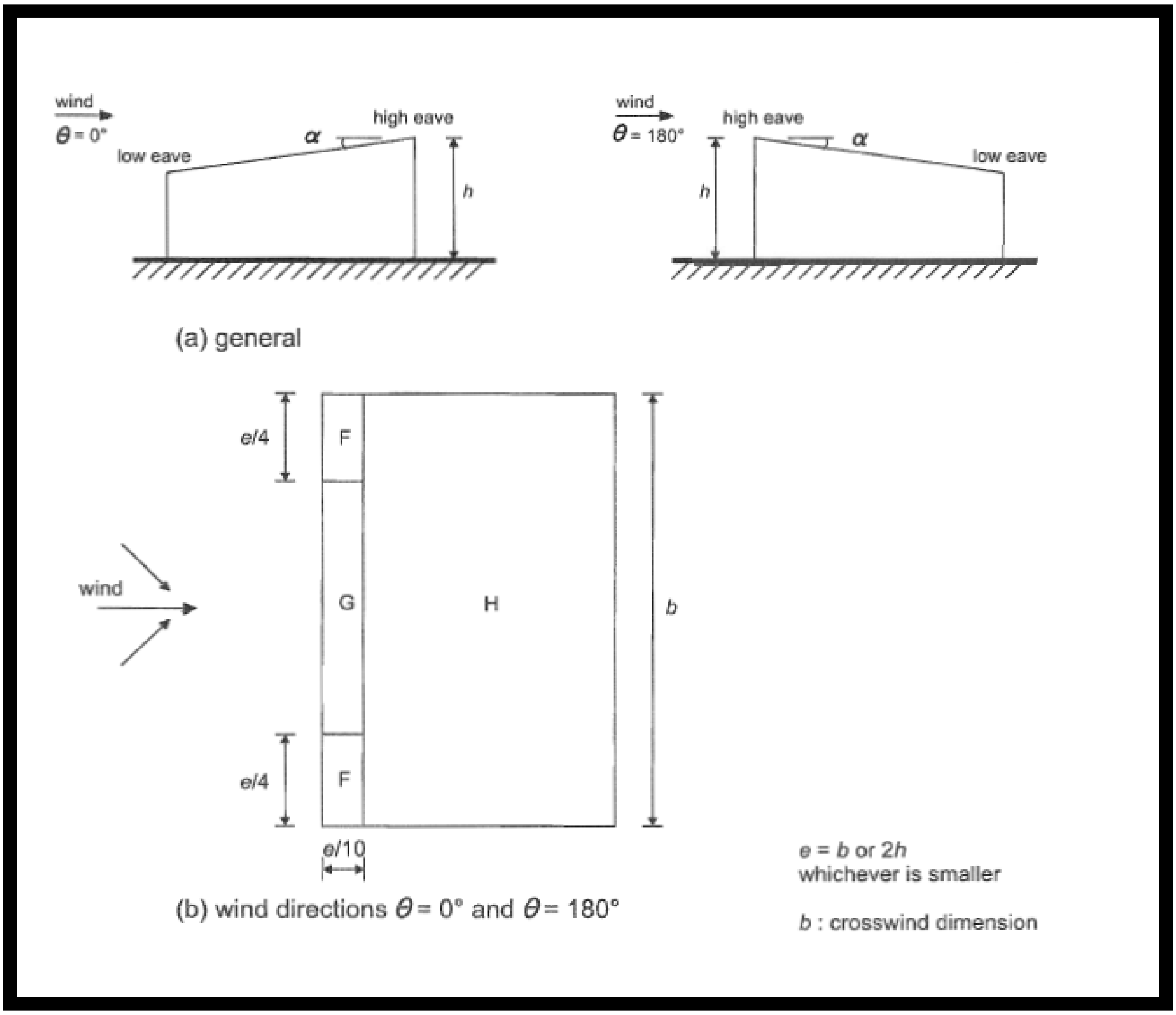


Figure 4.3.3 Zone plan for the mono-pitch roofs loads definition
with wind direction angle θ = 0°and θ = 180° (SFS-EN 1991-1-4 Figure 7.7)

When determining the influence area of the bearing elements, it should be remembered that vertical ties, half-timbers and other minor non-bearing elements do not perceive the wind load transmitted through the building envelope.

Thus, only the main parts of the building frame, namely columns, will participate in the load perception.

Linear interpolation coefficients were chosen from the table shown below.

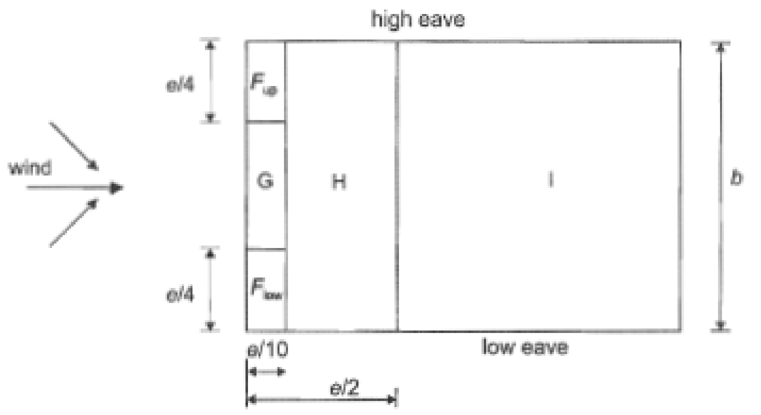
Table 4.3.2 Recommended values of external pressure coefficients for monopitch roofs (SFS-EN 1991-1-4 Table 7.3a)

Pitch Angle α	Zone for wind direction $\theta = 0°$						Zone for wind direction $\theta = 180°$					
	F		G		H		F		G		H	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5°	-1,7	-2,5	-1,2	-2,0	-0,6	-1,2	-2,3	-2,5	-1,3	-2,0	-0,8	-1,2
	+0,0		+0,0		+0,0							
15°	-0,9	-2,0	-0,8	-1,5	-0,3		-2,5	-2,8	-1,3	-2,0	-0,9	-1,2
	+0,2		+0,2		+ 0,2							
30°	-0,5	-1,5	-0,5	-1,5	-0,2		-1,1	-2,3	-0,8	-1,5	-0,8	
	+0,7		+0,7		+0,4							
45°	-0,0		-0,0		-0,0		-0,6	-1,3	-0,5		-0,7	
	+0,7		+0,7		+0,6							
60°	+0,7		+0,7		+0,7		-0,5	-1,0	-0,5		-0,5	
75°	+0,8		+0,8		+0,8		-0,5	-1,0	-0,5		-0,5	

When wind direction angle θ = 90°:

e = min(b;2h), where b – is crosswind dimention.

In our case e = min(7,00 m; 12,00 m) = 7,00 m.



(c) wind direction θ= 90°

Figure 4.3.4 Zone plan for the duo-pitch roofs loads definition with wind direction angle θ = 90°(SFS-EN 1991-1-4 Figure 7.7)

The following coefficients should be chosen:

Zone F_{up}: -2,19;

Zone F_{low}: -1,95;

Zone G: -1,83;

Zone H: -0,66;

Zone I: -0,56;

All coefficients are chosen in accordance with the table:

Table 4.3.3 Recommended values of external pressure coefficients for duopitch roofs (SFS-EN 1991-1-4 Table 7.3b)

Pitch Angle α	Zone for wind direction θ = 90°									
	F_{up}		F_{low}		G		H		I	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5°	-2,1	-2,6	-2,1	-2,4	-1,8	-2,0	-0,6	-1,2	-0,5	
15°	-2,4	-2,9	-1,6	-2,4	-1,9	-2,5	-0,8	-1,2	-0,7	-1,2
30°	-2,1	-2,9	-1,3	-2,0	-1,5	-2,0	-1,0	-1,3	-0,8	-1,2
45°	-1,5	-2,4	-1,3	-2,0	-1,4	-2,0	-1,0	-1,3	-0,9	-1,2
60°	-1,2	-2,0	-1,2	-2,0	-1,2	-2,0	-1,0	-1,3	-0,7	-1,2
75°	-1,2	-2,0	-1,2	-2,0	-1,2	-2,0	-1,0	-1,3	-0,5	

NOTE 1 At θ = 0° (see table a)) the pressure changes rapidly between positive and negative values around a pitch angle of α = +5° to +45°, so both positive and negative values are given. For those roofs, two cases should be considered: one with all positive values, and one with all negative values. No mixing of positive and negative values is allowed on the same face.

NOTE 2 Linear interpolation for intermediate pitch angles may be used between values of the same sign. The values equal to 0.0 are given for interpolation purposes

Internal pressure coefficients:

$$w_i = q_p(z_i) \cdot c_{pi} \qquad (4.3.9)$$

Where:

z_i – reference height of the internal pressure;

c_{pi} - pressure coefficient for the internal pressure.

The internal pressure coefficient depends on the size and distribution of the openings in the building envelope.

In our case c_{pi} taken as the more onerous of + 0,2 and - 0,3 as recommended in SFS-EN 1991-1-4 [3]. Such a decision was made because the structure should be able to be operated with different permeability levels of the building envelope (partly covered during the summer period or with enclosed building envelope during winter time or in case of extra precipitations in order to create more comfortable working conditions during the facility using period). In our case value c_{pi} = +0,2 would be more unfavorable than c_{pi} = -0,2.

Thus, the wind load per unit length (horizontal beam, vertical column in our case) would be determined as:

$$w \quad = q_p \cdot (c_{pi} + c_{pe}) \cdot a \tag{4.3.10}$$

Where w – is the wind load;

q_p – is the peak velocity pressure;

c_{pi} – pressure coefficient for the internal pressure;

c_{pe} – pressure coefficient for the external pressure;

a – influence area dimention, m (1,414 m for the horizontal beams (0,717 m for the edge beams) and 2,800 m for the vertical elements (1,400 for the edge ones). In addition, according to pressure wind load pressure distribution occurring on horizontal mounting beams reduce the internal stresses of the beam. Therefore, they may not be considered in load combinations in order to increase the safety margin.

This way, the wind load will be considered when it acts on the vertical components of the structure (bearing trusses and cover trusses).

The load distribution by the influence width would be:

1) Load case with wind direction angle θ = 0°:

For the windward side:

$w \quad = 0{,}946 \frac{kN}{m^2} \cdot 0{,}8 \cdot 2{,}8\, m = 2{,}12\, kN/m$ (For the central column);

$w \quad = 0{,}946 \frac{kN}{m^2} \cdot 0{,}8 \cdot 1{,}4\, m = 1{,}06\, kN/m$ (For the edge columns);

In the case of a load from the windward side, the internal pressure coefficient reduces the influence of wind pressure on structural elements from the windward side; therefore, it is not considered to increase the margin of safety.

For the leeward side:

$w \quad = 0{,}946 \frac{kN}{m^2} \cdot (0{,}6 + 0{,}2) \cdot 2{,}8\, m = 2{,}12\, kN/m$ (For the central column);

$w \quad = 0{,}946 \frac{kN}{m^2} \cdot (0{,}6 + 0{,}2) \cdot 1{,}4\, m = 1{,}06\, kN/m$ (For the edge columns);

2) Load case with wind direction angle θ = 180°:

For the windward side:

$w \quad = 0{,}946\frac{kN}{m^2} \cdot 0{,}8 \cdot 2{,}8\ m = 2{,}12\ kN/m$ (For the central column);

$w \quad = 0{,}946\frac{kN}{m^2} \cdot 0{,}8 \cdot 1{,}4\ m = 1{,}06\ kN/m$ (For the edge columns);

For the leeward side:

$w \quad = 0{,}946\frac{kN}{m^2} \cdot (0{,}6 + 0{,}2) \cdot 2{,}8\ m = 2{,}12\ kN/m$ (For the central column);

$w \quad = 0{,}946\frac{kN}{m^2} \cdot (0{,}6 + 0{,}2) \cdot 1{,}4\ m = 1{,}06\ kN/m$ (For the edge columns);

3) Load case with wind direction angle θ = 90°:

For the side facades:

$w \quad = 0{,}946\frac{kN}{m^2} \cdot (0{,}8 + 0{,}2) \cdot 2{,}8\ m = 2{,}8\ kN/m$ (For the central column);

$w \quad = 0{,}946\frac{kN}{m^2} \cdot (1{,}2 + 0{,}2) \cdot 1{,}4\ m = 1{,}96\ kN/m$ (For the edge columns (for both external pressure coefficient -1,2 was chosen in order to increase safety margin)).

In these two cases, the wind pressure acting on the vertical elements of the covering trusses is not considered due to its insignificant effect.

4.4 Jib cranes load

In the project following the model of the jib crane would be used:
«Seinäkääntö SKA 250 kg» with jib length 3,000 m and maximum lifting weigth 250 kg. In accordance with technical task two such cranes would be attached at two diagonally opposite corners of the conctruction to increase productivity and independence of use. The crane will be mounted to the steel plate using high-strength bolts. The steel plate would be welded to the vertical bearing element. The choice of such a crane is the most optimal for our situation and represents the best ratio of self-weight and load capacity. Also such cranes are the simplest during installation and commissioning. Another plus of choosing such a crane is the independence of the work of two opposite parts of the structure (located on two different containers). Unlike an overhead crane, where a small difference in the unevenness of the rail can cause additional loads, jib cranes are not sensitive to such cases and can be brought into working condition with much lower mounting accuracy.

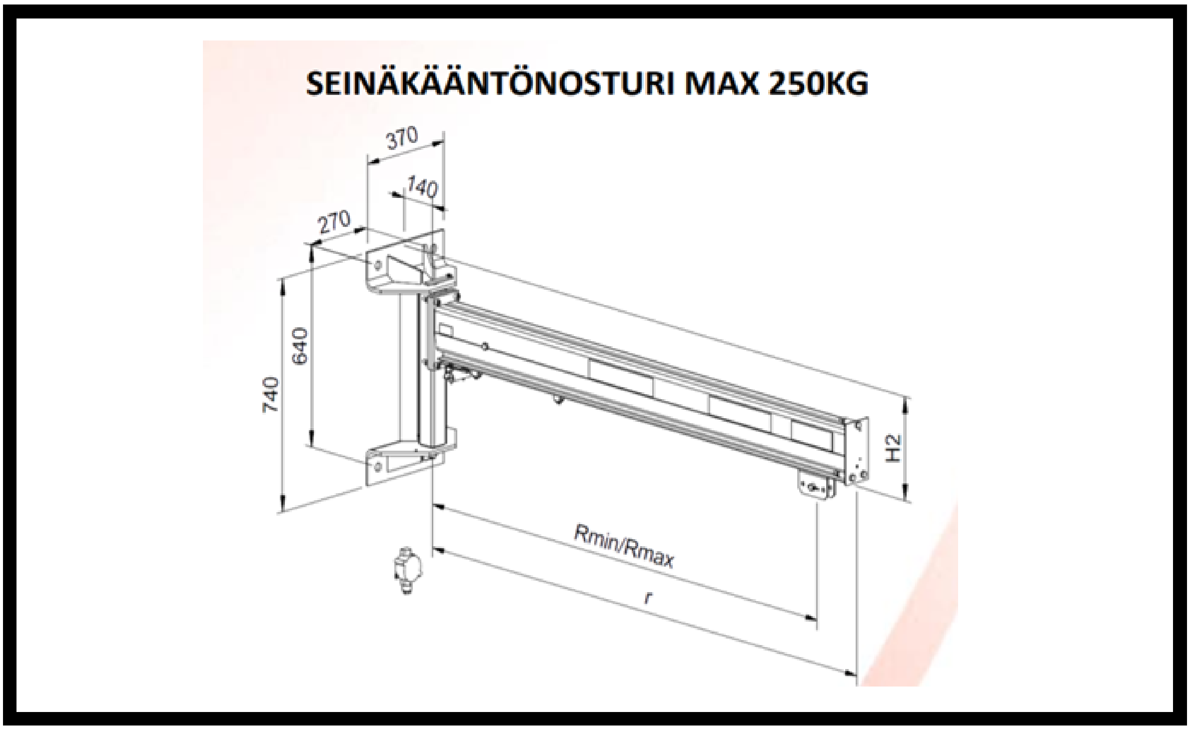


Figure 4.4.1 Jib crane drawing (Tuotetekno Oy cranes catalogue)

Load occurred by the Jib crane:

Load occurred by lifting goods and dead load of the crane construction (characteristic value):

$Q_k = m \quad + m_{s1} + m_{s2}$; (3.4.1)

Where:

Q_k – characteristic load value, kN;

m – maximum lifting weight, N;

In our case, due to the technical characteristics of the crane maximum lifting weight is 250 kg = 2,5 kN.

m_{s1} – weight of the crane's jib, N; m_{s1} = 67 kg = 0,67 kN.

m_{s2} – weight of the crane's plate, N; m_{s2} = 23 kg = 0,23 kN.

Thus, load occurred by crane would be:

$Q_k = 2{,}5 + 0{,}67 + 0{,}23$ = 3,4 kN.

Bending moment occurred by crane:

$M_k = M_c + M_j$ (3.4.2)

Where:

M_k – characteristic bending moment value, kN·m;

M_c – bending moment occurred by cargo lifting, kN·m;

M_j – bending moment occurred by crane's jib, kN·m;

The bending moment occurred by crane's jib could be represented as dead load distributed by the jib's length.

$M_c = 2{,}5\ kN \cdot 3\ m = 7{,}5\ kN \cdot m$;

$M_j = \frac{0{,}66\ kN \cdot 3\ m}{2} = 0{,}99\ kN \cdot m$;

$M_k = 7{,}5\ kN \cdot m + 0{,}99\ kN \cdot m = 8{,}49\ kN \cdot m$.

These loads will be applied together depending on the location of the crane jib.

5 Structural analysis

5.1 Analysis method

In this work, limiting states design method was used to select elements and analyse their bearing capacity. This method of calculating the structure implies the presence of groups of limit states that the structure must satisfy. For this thesis, an analysis of the structure was carried out according to structures two limiting states – ULS (Ultimate Limit State) and SLS (Serviceability Limit State). ULS group includes stress and stability analysis of the structure when SLS group includes deflection (in our case) as well as durability and cracking analysis. Such a method of structural design was chosen in accordance with rules and recommendations of EN-1990 [3] standard.

5.2 Load combinations

Load combinations were compiled in accordance with EN-1990 [3] general design principles as well as with Finnish National Annex rules that include necessary information about load partial factor and combination factor values. The partial load factor depends on load (dead, live, snow, wind, seismic, etc.), combination (favourable, unfavourable) and analysis group (SLS or ULS). In our design case seismic, emergency and temperature load cases application will not be considered in view of the features of the designed structure.
Thus, the following coefficients were used in load combinations considering consequences class (CC2) of the designed facility.

Table 5.2.1 Recommended values of combination and load partial factors

(Autodesk Robot Structural Analysis editor of code regulation table)

Code: SFS-EN 1990/A1 CC2 Version: 24

	Nature	Subnature	γ_{max}	γ_{min}	γ_s	γ_a	$\Psi_{0,1}$	$\Psi_{0,2}$	$\Psi_{0,3}$	$\Psi_{0,n}$	Ψ_1	$\Psi_{2,1}$	$\Psi_{2,n}$	Ψ_K	ξ_1	ξ_2
1	Dead	STRC	1.35	0.9	1	1									0.852	1
2	Dead	NSTR	1.35	0.9	1	1									0.852	1
3	Live	Luokka A	1.5		1		0.7				0.5	0.3				
4	Live	Luokka B	1.5		1		0.7				0.5	0.3				
5	Live	Luokka C	1.5		1		0.7				0.7	0.3				
6	Live	Luokka D	1.5		1		0.7				0.7	0.6				
7	Live	Luokka E	1.5		1		1				0.9	0.8				
8	Live	Luokka F	1.5		1		0.7				0.7	0.6				
9	Live	Luokka G	1.5		1		0.7				0.5	0.3				
10	Live	Luokka H	1.5		1											
11	Wind		1.5		1		0.6				0.2					
12	Snow		1.35		1		0.7				0.4	0.2				
13	Snow	Lumikuorma	1.5		1		0.7				0.4	0.2				
14	Snow	Lumikuorma	1.5		1		0.7				0.5	0.2				
15	Snow	Jääkuorma	1.5		1		0.7				0.3					

5.3 Assessment of the bearing capacity of profiled sheeting

As mentioned on the load evaluation part, some parts of the designed structure require verification after assessing the loads. In our case, the «Ruukki» load bearing profiled sheet T45-30L-905 was pre-selected.

T45-30L-905 properties (in accordance with manufacturer instruction):

Width: 905 mm;

Thickness: 0,7 mm;

Nominal weight of the m^2: 7,59 kg (0,076 kN/ m^2);

Steel yield strength: 350 MPa (S350 steel);

Calculated properties (the calculation is based on the geometric characteristics indicated on the manufacturer's website):

Moment of inertia (of one m): 29 cm^4;

Section modulus: 11,6 cm^3.

The design scheme in our case could be represented as a double span beam with maximum moment on support. Also, it should be noted that the calculation is carried out on 1 m of profiled sheeting:

$$M = -\frac{q(l \cdot cos\alpha)^2}{8}; \qquad (5.3.1)$$

Where:

q – load value, kN/m;

l – span length, m;

α – roof slope angle;

Thus, the design scheme can be represented as follows:

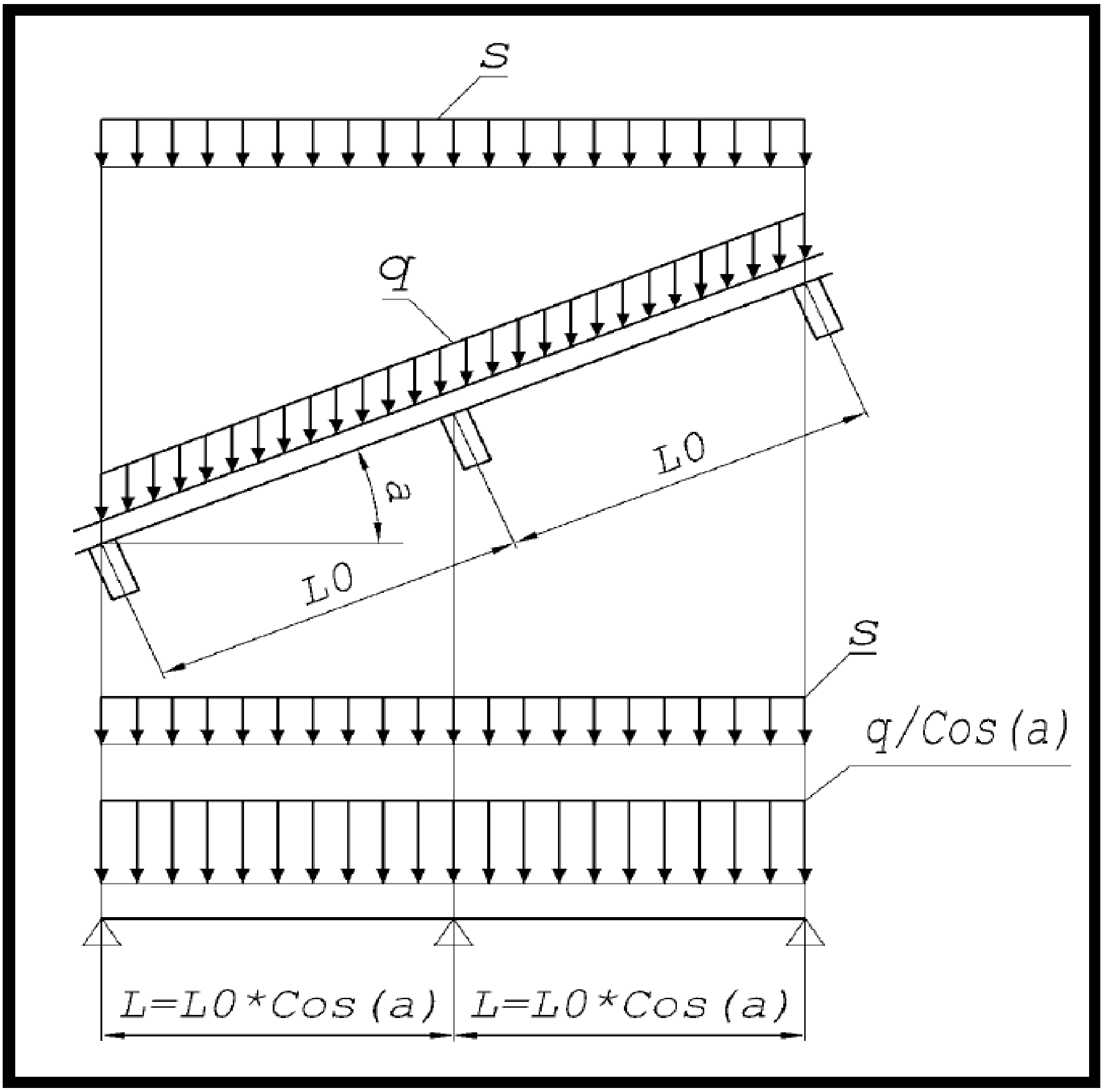


Figure 5.3.1 Transformation of the calculation scheme (Lightweight metal structures. Calculation Examples)

A uniformly distributed load will be composed of snow dead load, considering the load partial coefficients specified in the table 5.2.1.

Also, it is necessary to consider the direction of action of the dead load and the span length when calculating a uniformly distributed load.

This way, the uniformly distributed value could be represented:

$$q = 1{,}5 \cdot S + 1{,}35 \cdot \frac{g_k}{Cos(\alpha)}; \tag{5.3.2}$$

Where:

q – uniformly disributed load value, kN/m;

l – span length, m;

g_k – dead load value, kN/m;

S – snow load value, kN/m;

α – roof slope angle;

Thus:

$$q = 1{,}5 \cdot 2{,}2 + 1{,}35 \cdot \frac{0{,}076}{Cos(8°)} = 3{,}4 \text{ kN/m};$$

And the maximum bending moment value would be:

$$M = \frac{3{,}4 \cdot (1{,}414 \cdot 0{,}99)^2}{8} = 0{,}83\ kN \cdot m;$$

Strength analysis of the profiled sheeting:

$$\frac{M_{max}}{W \cdot \frac{fu}{\gamma m}} < 1; \tag{5.3.3}$$

Where:

W – section modulus, cm^3;

M_{max} – maximum bending moment, $0{,}83\ kN \cdot m$;

f_y – steel yield strength, MPa;

γ_m – material properties partial factor;

$$\frac{0{,}83 \cdot 10^3}{11{,}6 \cdot \frac{350}{1}} < 1;$$

$0{,}2 < 1$ – section ok;

Reducing the cross section is not usable because of design reasons.

And maximum deflection could be represented using formula:

$$\delta = \frac{q(l \cdot cos\alpha)^4}{185EI_x} < \frac{l}{200}; \tag{5.3.4}$$

Where:

δ – deflection, cm;

q – uniformly disributed load value, kN/m;

l – span length, m;

E – elasticity modulus, MPa;

I_x – moment of inertia, 29 cm^4;

In this case, uniformly distributed load should be represented as characteristic value:

$$q = 1 \cdot S + 1 \cdot \frac{g_k}{Cos(\alpha)} = 1 \cdot 2{,}2 + 1 \cdot \frac{0{,}076}{0{,}99} = 2{,}28\ \mathrm{kN/m};$$

$$\delta = \frac{\frac{0{,}0228kN}{cm} \cdot (141{,}4\ cm \cdot 0{,}99)^4}{185 \cdot 2{,}1 \cdot \frac{10^4\ kN}{cm^2} \cdot 29cm^4} = 0{,}08\ cm < \frac{140\ cm}{200} = 0{,}7\ cm;$$

From this we can conclude that the selected profiled sheet is suitable for use in our case.

5.4 Cantilever crane plate connection analysis

In this project, the cantilever jib crane would be fastened with high-strength bolts (as recommended by the manufacturer) to the plate, which will be welded to the column. Thus, the strength capacity of such weld connection should be checked.

Plate size: 370x740 mm;

Plate material: S355;

Weld fillet leg: 6 mm;

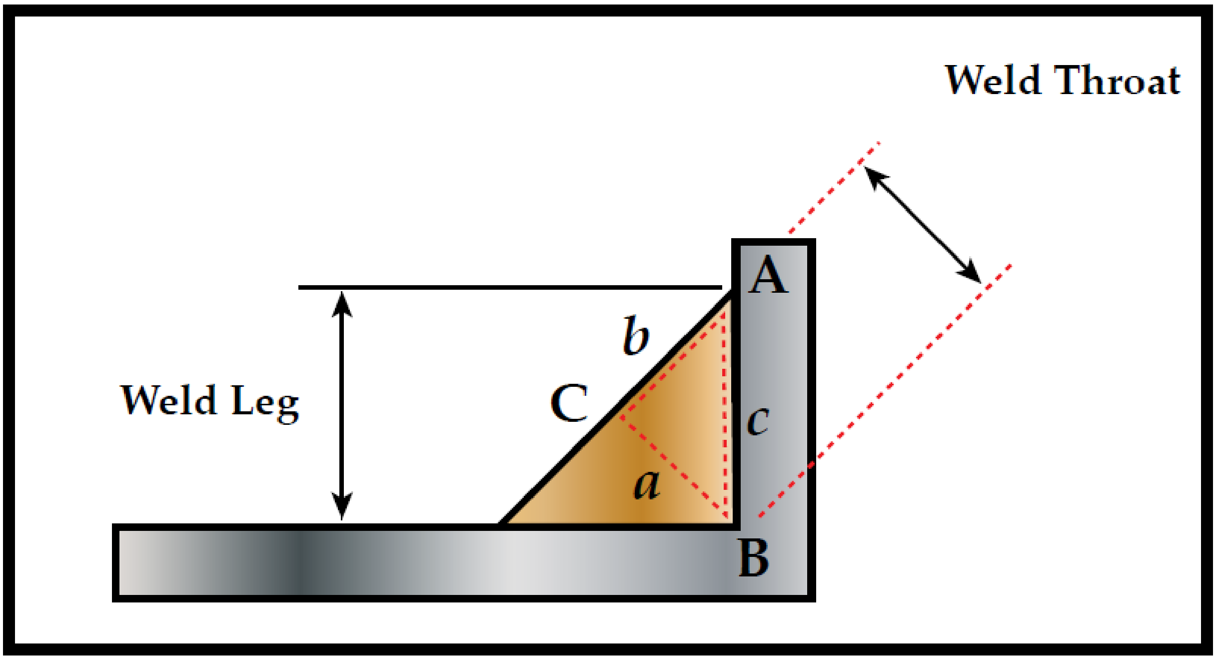


Figure 5.4.1 Weld fillet characteristics (Materials&Welding website)

The weld throat could be calculated as:

$$a = L \cdot Cos(45°) \tag{5.4.1}$$

Where:

L – weld fillet leg size, mm;

a – weld fillet throat size, mm;

$a = 6 \cdot Cos(45°) = 4{,}24\ mm;$

This way, bearing capacity of weld willet:

$$F_{w,Rd} = \frac{f_u}{\sqrt{3} \cdot \beta_w \cdot \gamma_{M2}} \tag{5.4.2}$$

Where:

$F_{w,Rd}$ – bearing capacity of weld fillet, kN/ cm^2 ;

f_u – steel ultimate tensile sthrengh, kN/cm^2;

β_w – correlation factor (EN1993-1-8) [4];

γ_{M2} – material partial factor (EN1993-1-8) [4];

$$F_{w,Rd} = \frac{47}{\sqrt{3} \cdot 0{,}9 \cdot 1{,}25} = 24\ \mathrm{kN/cm^2};$$

The following forces would act at the attachment point:

Shear forces:

$$Q = 1{,}35 \cdot Q_k \tag{5.4.3}$$

Where:

Q_k – characteristic load value, occurred by crane's plate, jib and by lifting goods;

Bending moments:

$$M = 1{,}35 \cdot M_k \tag{5.4.4}$$

M_k – characteristic bending moment value, occurred by crane's jib and by lifting goods (bending moment occurred by crane's plate is not significant and may not be taken into account in our case);

$Q = 1{,}35 \cdot 3{,}4 = 4{,}59\ kN;$

$M = 1{,}35 \cdot 8{,}49 = 11{,}46\ kN \cdot m;$

Determine the geometric characteristics of the weld fillet cross section at the plate to column attachment point:

Moment of inertia:

$$J_x = \frac{ba^3 - b_1 a_1^3}{12} = \frac{12 \cdot 75{,}2^3 - 1{,}08 \cdot 74^3}{12} = 60557\ cm^4; \tag{5.4.5}$$

Where:

a,b,a_1,b_1 – external and internal sides of the weld contour

Area of the welding fillet:

$$A = l \cdot a\ ; \tag{5.4.6}$$

Where:

$l -$ total length of the weld fillet (subject to weld defects of 10 mm on each side);

a – weld fillet throat size, mm;

$A = 2 \cdot (74 + 10{,}8) \cdot 0{,}42 = 71{,}2\ cm^2$;

Section modulus:

$$W_x = \frac{2 \cdot J_x}{h}; \quad (5.4.6)$$

Where:

$J_x -$ moment of inertia, cm^4;

$h -$ cross section height, cm;

$W_x = \frac{2 \cdot 60557}{75{,}2} = 1610\ cm^3$;

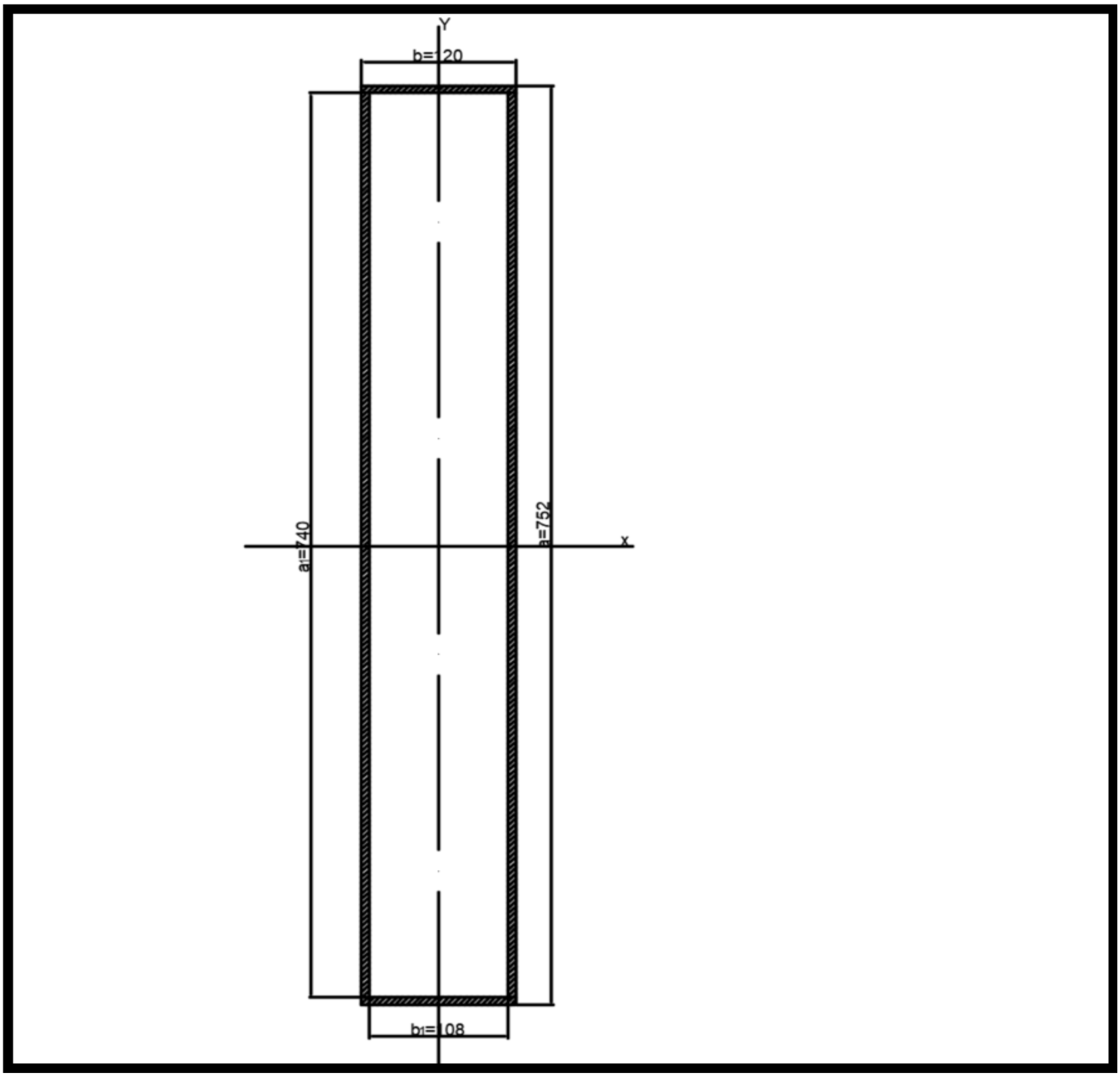


Figure 5.4.2 Weld fillet cross section dimensions (Autodesk AutoCAD student's version drawing)

Weld fillet shear stress from bending moment:

$$\tau_M = \frac{M}{W_x}; \tag{5.4.7}$$

Where:

M – maximum bending moment, $kN \cdot m$;

W_x – section modulus, cm^3;

$$\sigma = \frac{11{,}46 \cdot 10^2}{1610} = 0{,}7 \text{ kN/cm}^2;$$

Weld fillet shear stress from shear force:

$$\tau_Q = \frac{Q}{A}; \tag{5.4.8}$$

Where:

Q – maximum shear force, kN;

A – weld fillet section area, cm^2;

$$\tau_Q = \frac{4{,}59}{71{,}2} = 0{,}06 \text{ kN/cm}^2;$$

Total stress in the weld fillet:

$$\tau_T = \sqrt{\tau_Q^2 + \tau_M^2} = \sqrt{0{,}06^2 + 0{,}7^2} = 0{,}7\ kN/cm^2; \tag{5.4.9}$$

The stresses arising in the weld are much less than critical value for steel.

Thus, the welded attachment point is ok.

The strength of the weld when exposed to a force from the plane is lower than in the case of exposure in the plane of the weld. Thereby, the strength of the weld in the plane is obviously ensured.

5.5 Selection of the thickness of the base plate

In our case, the base plate will work as a bendable console with 35 mm overhang. The length and width of the bearing plate is accepted constructively (in order to weld the shipping container twist-lock). The plate works on bending as a plate supported on the end of the beam and loaded with uniformly distributed (conditionally) pressure of the support reaction N. Determine the maximum bending moment:

$$q = \frac{N}{B^2};$$

Where:

q – uniformly distributed load, kN/m^2;

N – support reaction, kN (the value taken from the calculation model, the results would be shown below);

B – bearing plate's side, m;

$$q = \frac{45{,}12}{0{,}21^2} = 1023\ \mathrm{kN/m2};$$

The moment value will be calculated in the plane per 1m:

$M = 0{,}5 \cdot q \cdot c^2;$

Where:

M – maximum moment, kN·m;

q – uniformly distributed load, kN/m;

c – console overhang, m;

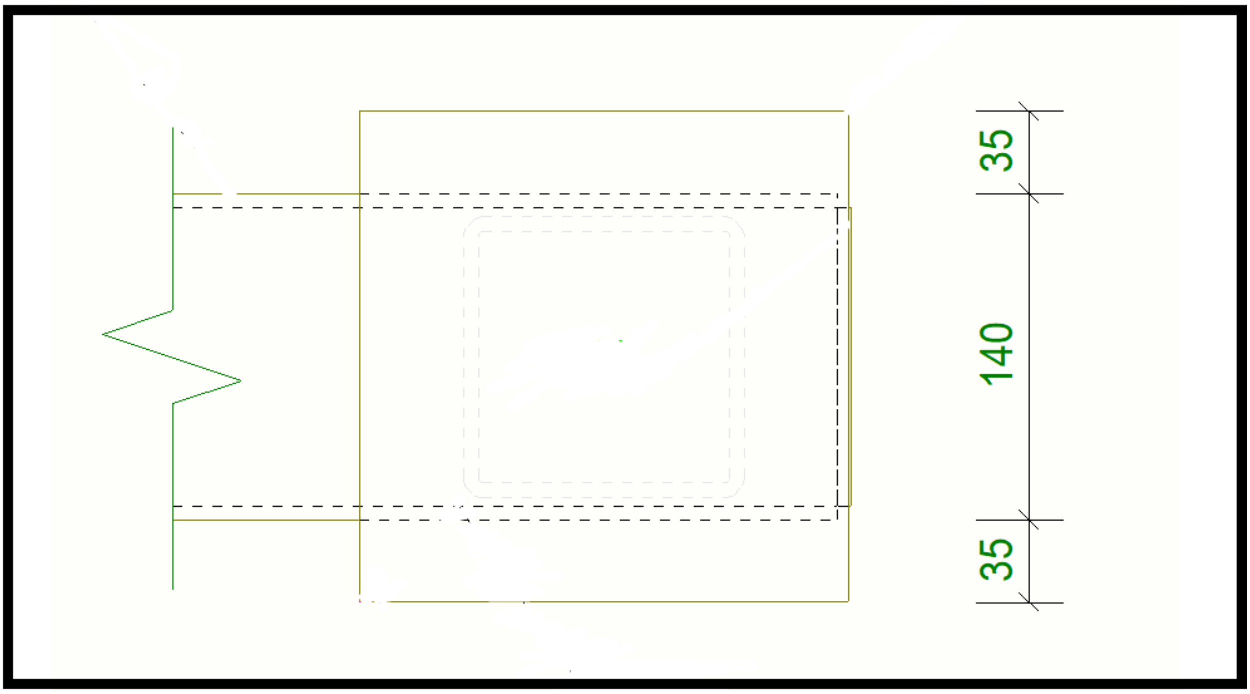


Figure 5.5.1 Bearing plate drawing (Autodesk AutoCAD student's version)

$M = 0{,}5 \cdot 1023 \cdot 0{,}35^2 = 62{,}65\ \mathrm{kN \cdot m};$

Thus, the plate thickness would be:

$$t = \sqrt{\frac{6M \cdot \gamma m}{f_y}};$$

Where:

t – plate's thickness, mm;

M – maximum moment, kN·m;

γm – material partial factor;

f_y – steel yield strength, MPa;

$$t = \sqrt{\frac{6 \cdot 62{,}65 \cdot 1{,}25}{235 \cdot 10^3}} = 0{,}04\ m = 4\ mm;$$

Thus, the minimum thickness of the base plate is 4 mm, but for structural purposes 10 mm thickness would be taken.

The base plate would be welded to the container twist-lock connector, the bearing capacity of which is many times greater than the pressure exerted on it (510 kN tension/compression, 420 kN shear strength).

5.6 Complete structural analysis

The bearing capacity of the whole structure was calculated in the program for the structural analysis of building constructions. All sections were selected based on the given loads as well as the material and the stress-strain state of each element. The analysis of permissible structural deflections was also carried out.

The results are described below:

Table 5.6.1 Support reactions. Global extremes

	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	1,29	16,87	45,12	0,00	0,00	0,00
Node	1	65	1	64	64	64
Case	203 (C)	142 (C)	193 (C)	205 (C)	203 (C)	180 (C)
MIN	-1,29	-15,72	-9,77	0,0	-0,00	-0,00
Node	15	63	1	1	63	63
Case	203 (C)	142 (C)	155 (C)	1	26 (C)	171 (C)

Table 5.6.2 Optimal cross-sections

Member	Section	Material	Lay	Laz	Ratio
Code group : 1 Columns					
4 Column1	OK RRHS 120x12	S 355 M/ML	52.02	52.02	0.33
Code group : 2 Beams					
15 Beam8	OK RRHS 80x80x	S 235	92.83	92.83	0.46
Code group : 3 Vertical bracings					
58 Vertical braci	OK RRHS 90x90x	S 235	100.90	100.90	0.19
Code group : 4 Horizontal bracings					
62 Horizontal bra	OK RRHS 100x10	S 355 M/ML	147.58	147.58	0.30
Code group : 5 Bearing beams					
11 Beam5	OK IPE 300	S 355 M/ML	56.74	211.10	0.85

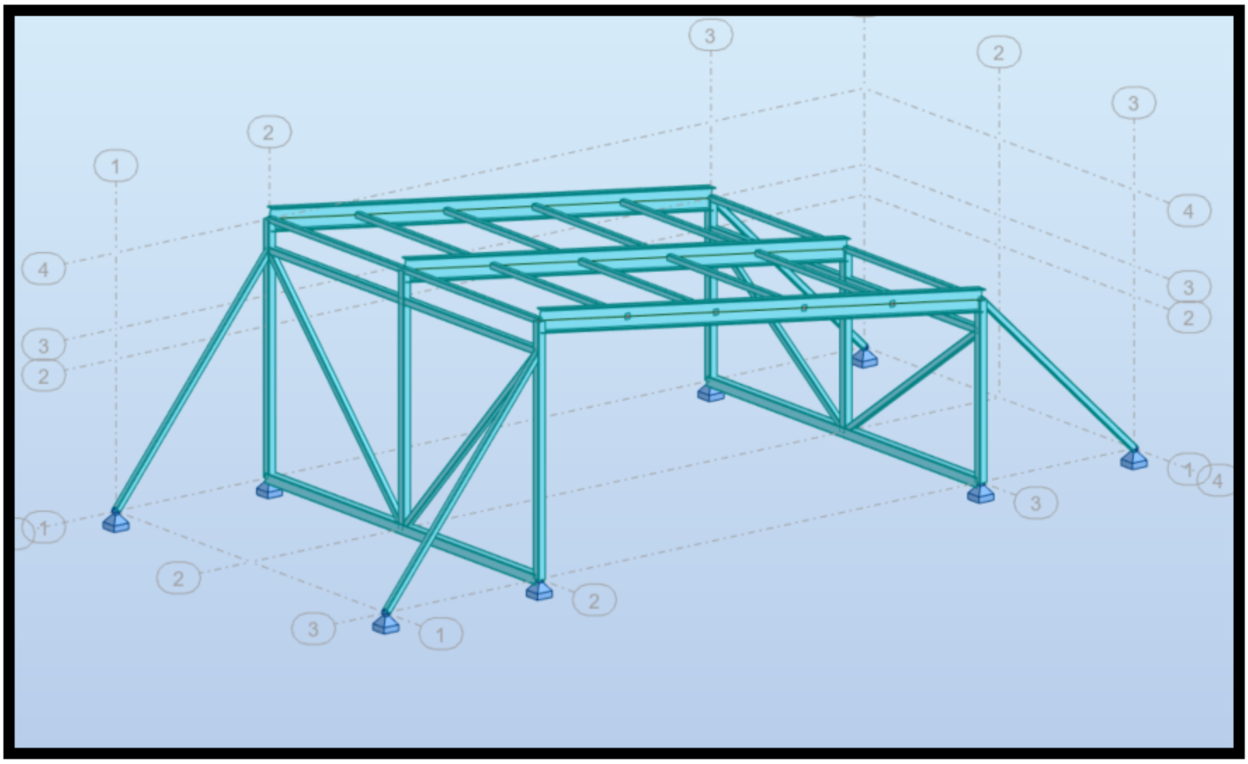


Figure 5.6.1 Global view (Autodesk Robot structural analysis student’s version)

Thus, the total weight of all parts of the structure is less than 3 tons what relates it to lightweight structures. Also, the use of factory fasteners and the grouping element groups into shipping marks will significantly ease the tasks of installation and transportation.

6 Conclusion

Based on the results of the work, we can conclude that it is possible to implement such a design solution. Also, this structure was calculated with a large margin (some reliability factors, data in the Eurocode and National Annexes were intentionally overstated). Thus, this design allows the use of a wide range of envelope structures and equipment that can be installed. Summing up, the thesis main goals were achieved: developing a lightweight structure using a flexible design scheme and withstanding a wide range of loads, which allows it to be used in various locations and positions. Thus, this scheme can serve as a basis for the development of structures of such kind.

References

1. EN 1991-1-3: Eurocode 1: Actions on structures – Part 1-3: General actions – Snow loads
2. EN 1991-1-4: Eurocode 1: Actions on structures – Part 1-4: General actions – Wind loads
3. EN 1990: Eurocode 0: Basics of structural design
4. EN 1993: Eurocode 3: Design of steel structures
5. Finish National Annex 2
6. Finnish National Annex 4
7. Finish National Annex 5