STRUCTURAL ANALYSIS AND DESIGN OF A LOW-RISE STEEL INDUSTRIAL HALL

A case study of Hunter's Hut project

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Bachelor's thesis

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ABSTRACT

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Subject	Structural analysis and design of a low-rise steel industrial hall	
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ABSTRACT

The purpose of this Bachelor's thesis was to demonstrate how to conduct structural analysis and design of a low-rise steel industrial building. The project was commissioned by HAMK Tech Research Unit as a practical summer project for trainees. The client was interested in constructing a single-storey industrial hall for processing moose carcasses delivered by hunting clubs of Finland. The building's architecture is based on the function of the building. Structural design of the building was assigned to a group of students who were guided by supervising teachers of HAMK University of Applied Sciences. This thesis serves as a demonstrative example of how structural analysis and design for this building was conducted.

The design work was started by calculation of loads and defining the load combinations. One of the first steps was to create a BIM model using Tekla Structures that has later been used for decisions on various design problems and for making shop drawings for production. Further on, the building structure was modelled in Dlubal RFEM where the structural analysis was made. Resistance check of joints was made using the calculation results from RFEM. Production of technical drawings for shop manufacturing was the final step of the project.

The thesis contains a detailed description of the design steps such as calculation of loads, structural analysis conducted in software, resistance check of joints and finally, examples of production drawings. The design was completed in accordance with Eurocodes, National Annexes and requirements of the client.

Keywords Structural analysis, steel structures, bracing

Pages 52 pages including appendices 50 page

CONTENTS

1 INTRODUCTION

Structural design process has been changing through the development history of construction engineering. Various methods of structural design come and go, shaping and modifying the work life of engineers. Methods applied in structural design and analysis today are quite different from what could be observed 50 years ago. Such aspects as buildability of the structure, the choice of shop or site connection of the component steel structures, economic sustainability and the environmental impact of the structure are becoming more crucial elements of the design process. In Finland and other European countries these aspects together with numerous other must satisfy the requirements of relevant Eurocodes and National Annexes.

Thanks to modern computers and powerful software, it is much more accurate and faster to conduct a design of massive structures nowadays. However, it does not mean that an engineer should rely entirely on the machines and let them do the whole work. There is still no computer that is able to replace human consciousness and creative thinking. Thus, engineers should learn to use software responsibly and understand the theory that lies behind numbers. Machines are the tools that require deep knowledge and understanding of the subject.

The purpose of this thesis is to demonstrate the way of conducting structural analysis and design of a building made of steel elements. The thesis is based on the summer project during which the structure of the building was analyzed. Problems related to foundation design are out of the scope of this thesis. The design is based on Ultimate limit state (ULS) and Serviceability limit state (SLS) approach. The design combines manual calculations as well as calculations done using software. This document will serve as a demonstration of how the design process works starting from the calculation of loads and performing structural analysis and ending with resistance checks of the joints and production of technical drawings. Another aim of the thesis is to study braced frames and to get an idea of what types of bracing there are and how it affects the whole structure.

2 DESCRIPTION OF THE PROJECT

2.1 Structural design

Structural design might have a complicated process with a number of steps and instruments. Material experts, BIM, fire and structural engineers and a number of other professionals might be involved. Figure 1 below shows the flow chart of a typical structural design process.

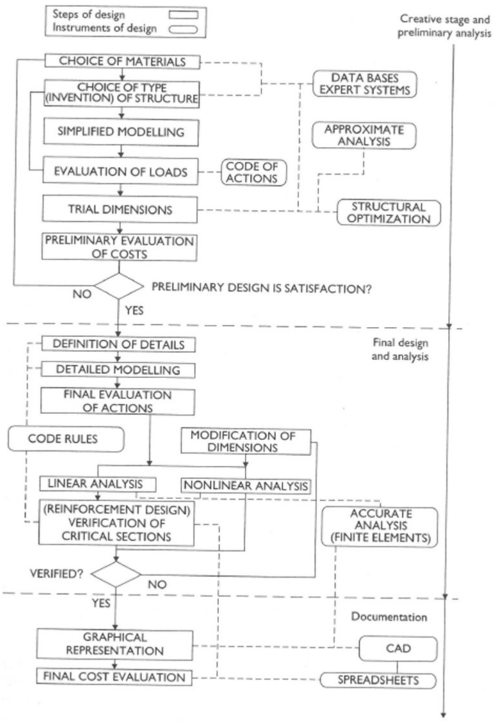


Figure 1. Flow-chart of the structural design process (Al Nageim & MacGinley, 2005)

Structural design means artistic invention and dimensioning. Invention is the creation of a structural form, dimensioning is to assign to every structural member adequate dimensions for stability, serviceability, suitability and sustainability (Al Nageim & MacGinley, 2005).

As for the Hunter's Hut project, structural analysis alone was conducted. There was no need to additionally design and change the dimensions of the structural components since the requirement was to use the structures of certain profiles. Nevertheless, the design resistance of structural members is checked and verified. Possible changes are considered.

The Hunter's Hut project consisted of the following steps:
1. Defining the structural solutions such as trusses and bracing
2. Calculating the loads acting on the structure
3. Performing structural analysis in Dlubal RFEM
4. Design resistance check of structural connections
5. Building a BIM model in Tekla Structures
6. Making shop drawings for production

2.2 **Location of the building**

It is planned to install the building 16 kilometres to the south-west from Hämeenlinna. The place is named Villinnotko and is located nearby Renko which is shown on the maps in Figures 2-3. The surroundings are represented primarily as fields with low construction density. As shown in Figures 4-5 below, the building will be erected in the middle of the sandy ditch that occurred due to excavation works.

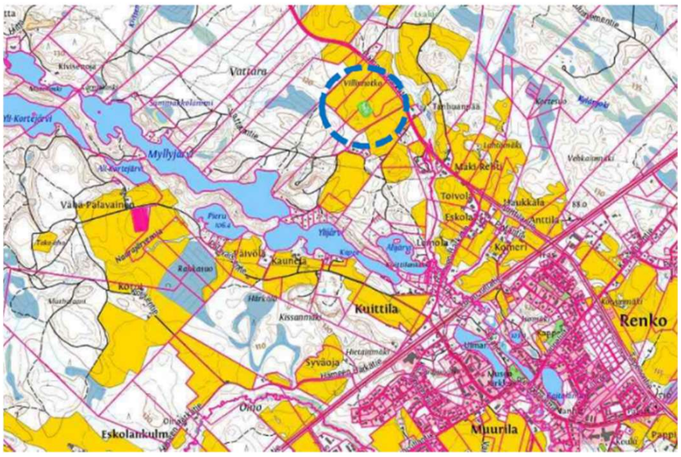


Figure 2. Municipal map of Renko area

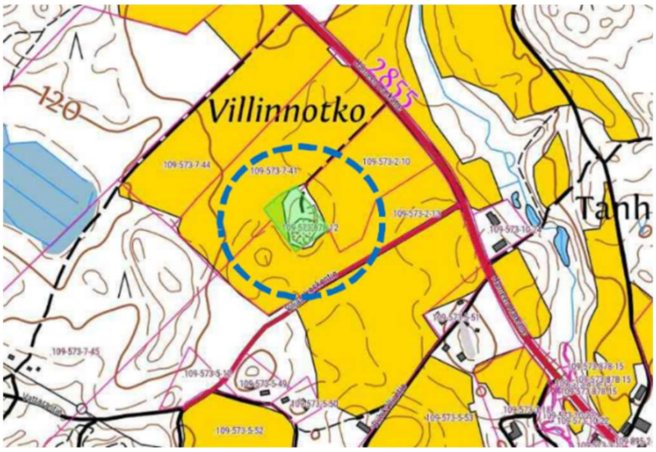


Figure 3. Municipal map of Villinnotko

Figure 4. Surrounding fields

Figure 5. Sandy ditch

2.3 **Architectural design**

Architectural design is based on the function of the building and is developed by the client. The hall consists of three rooms. The first room is meant for unloading of an animal’s carcass and for further skinning. The second room is a storage that also serves as a fridge. Finally, the third room is meant for cutting the carcass into smaller parts and packaging. Figures 6-7 demonstrate architectural drawings for a building permit which give information on the shape of the building, main dimensions and some structural components.

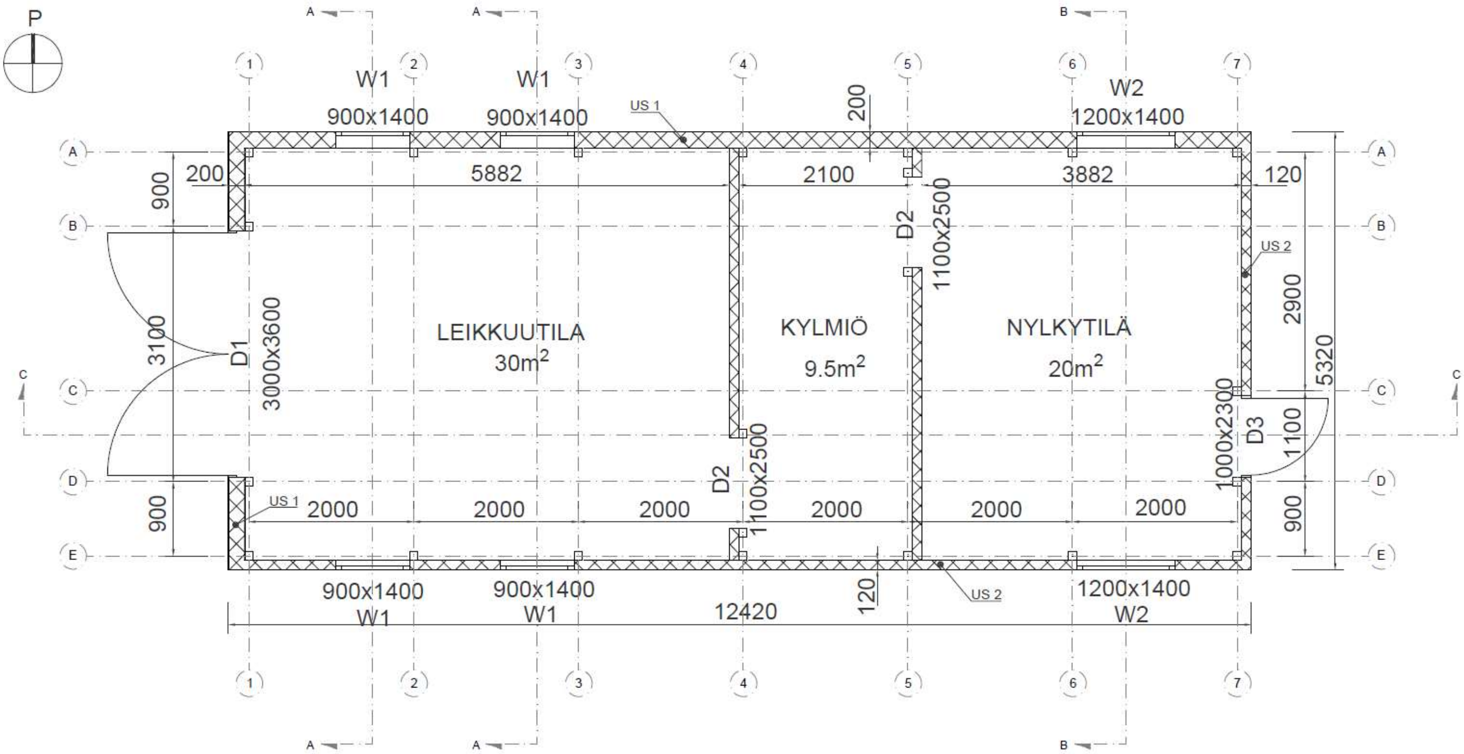


Figure 6. Floor plan

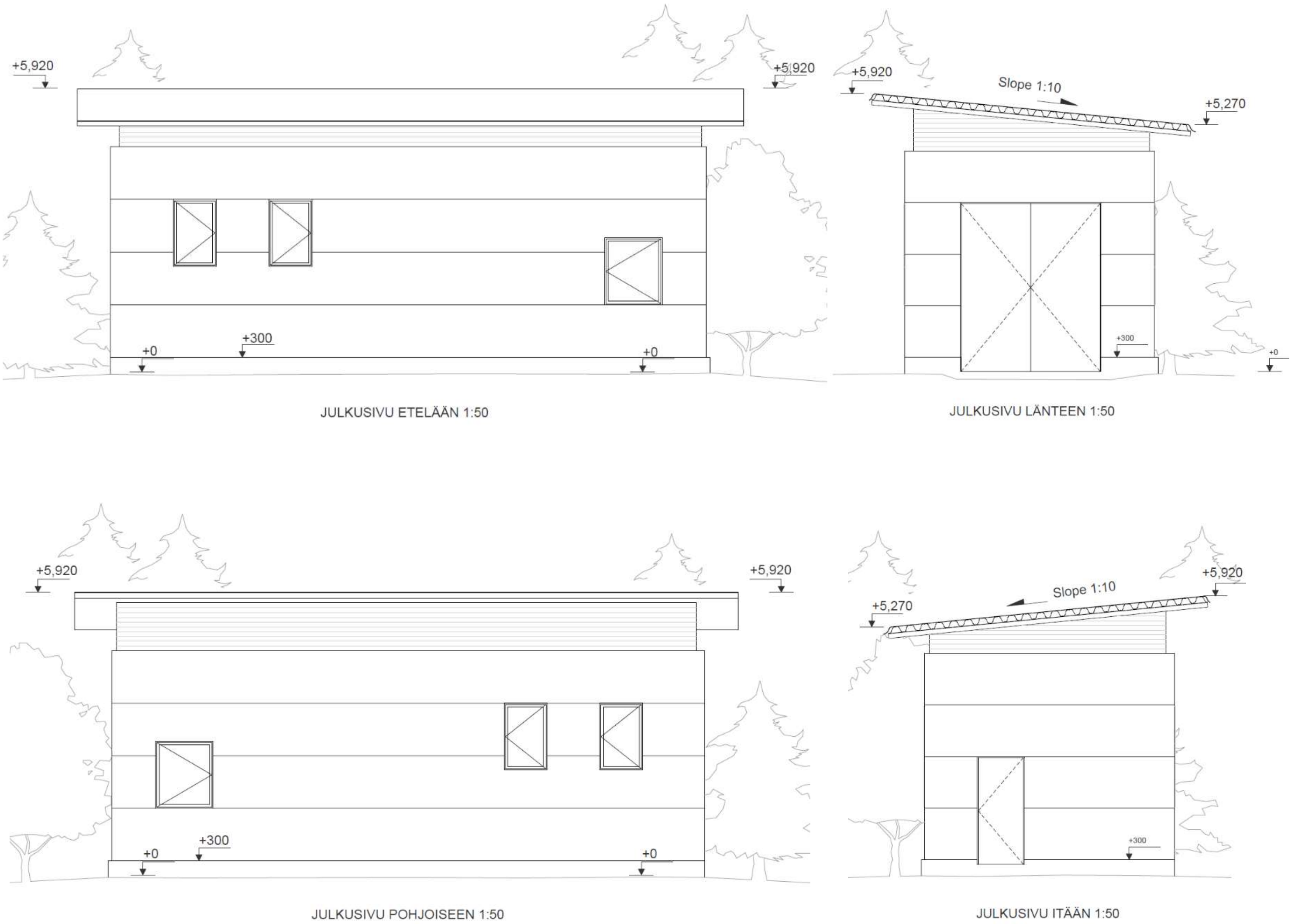


Figure 7. Façade drawings

2.4 **Methodology**

2.4.1 Codes and standards

Structural calculations in this project are based on the following standards:

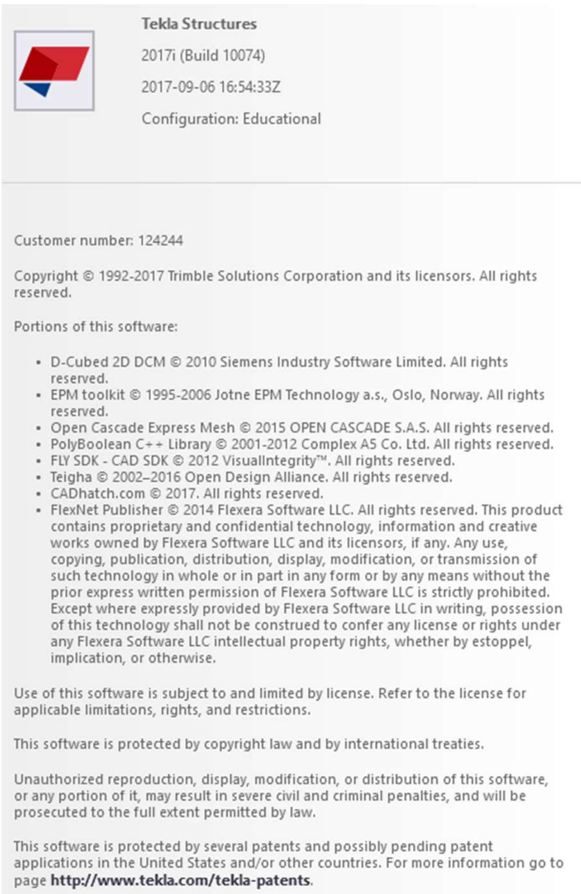
Basis of structural design	- EUROCODE 0 (SFS EN 1990 + Finnish NA)
Loadings	- EUROCODE 1 (SFS EN 1991 + Finnish NA)
Steel Structures	- EUROCODE 3 (SFS EN 1993 + Finnish NA)
Design of Joints	- EUROCODE 3 (SFS EN 1993-1-8)

2.4.2 Software used during the project

All the software used during the project is provided by HAMK University of Applied sciences and is run on the computers of the educational institution.

- Tekla structures

Tekla Structures is building information modelling software that is able to create structures and to store information on the components of the building. The software allows designers to make 3D models of structures as well as produce technical drawings and make calculations on the amount of materials needed for construction.
Figure 8 below shows the license information of the software.



Tekla Structures

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Configuration: Educational

Customer number: 124244

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Portions of this software:

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Figure 8. Tekla Structures license window

- Dlubal RFEM

Dlubal RFEM is finite element-based software that is used for structural analysis and design of structures made of steel, concrete, wood, glass and other materials. RFEM is able to produce reports that include internal forces, deflections and support reaction of structures. It is also possible to design building components based on building regulations like Eurocodes and National Annexes.
Figure 9 below shows the license information of the software.

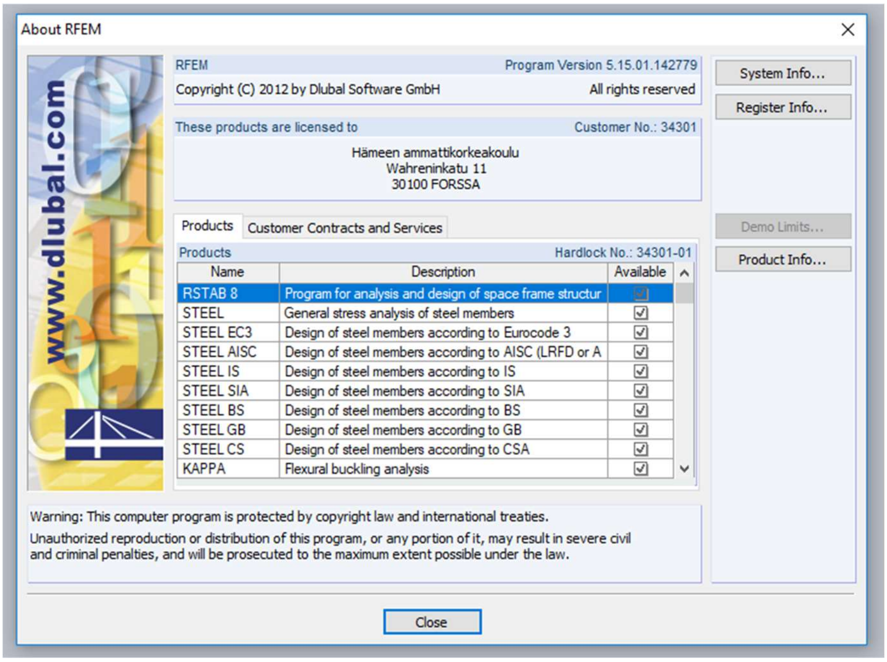


Figure 9. Dlubal RFEM license window

- Mathcad 15

Mathcad is software that is used for mathematical calculations. The software is used for calculation of loads, load combinations and resistance checks of joints.
Figure 10 below shows the license information of the software.

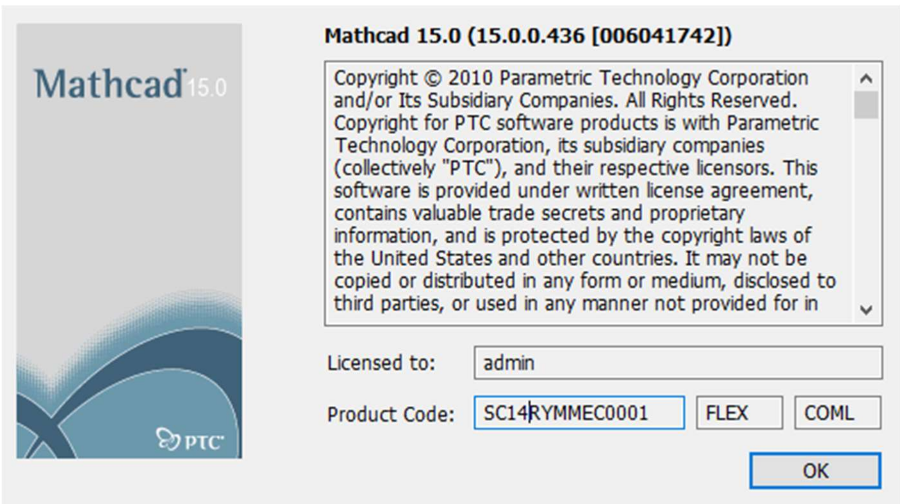


Figure 10. Mathcad license window

3 STRUCTURAL BUILDING COMPONENTS

Structural building components are described in this part of the report. The choice of material was limited because one of the client's demands was to design the whole structure only of square hollow sections (SHS) with sizes of 100x100x5 and 60x60x4. The sections are shown in Figure 11 below.

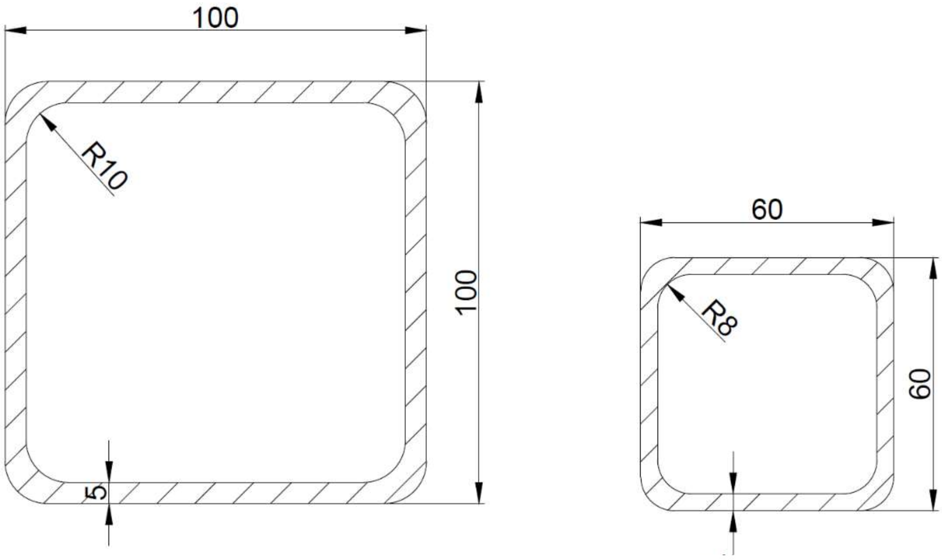


Figure 11. Profiles SHS100x100x5 and SHS60x60x4

Profiles of structural components together with their steel grades used in the project are presented in Table 1.

Table 1. Profiles and steel grade of the structural building components

Structural component	Profiles	Steel grade
Load-bearing columns	SHS100x100x5	S355
Horizontal beams	SHS60x60x4	S355
Beams at the ends of the roof structure	SHS100x100x5	S355
Bracing	SHS60x60x4	S355
Top and bottom chords of the trusses	SHS100x100x5	S355
Truss webs	SHS60x60x4	S355

3.1 **Foundation**

Even though the foundation is not in the scope of this report, it is important to know the initial dimensions. This information is needed to understand the support conditions of the structure and later to calculate the resistance of the column-base connection.

The foundation in a building serves for transferring loads from the superstructure to the ground. Load-bearing columns that transfer loads from the structure are placed on top of 200mm concrete foundation. The foundation transfers loads further to the footing. Figure 12 below shows the preliminary design of the foundations provided by the client.

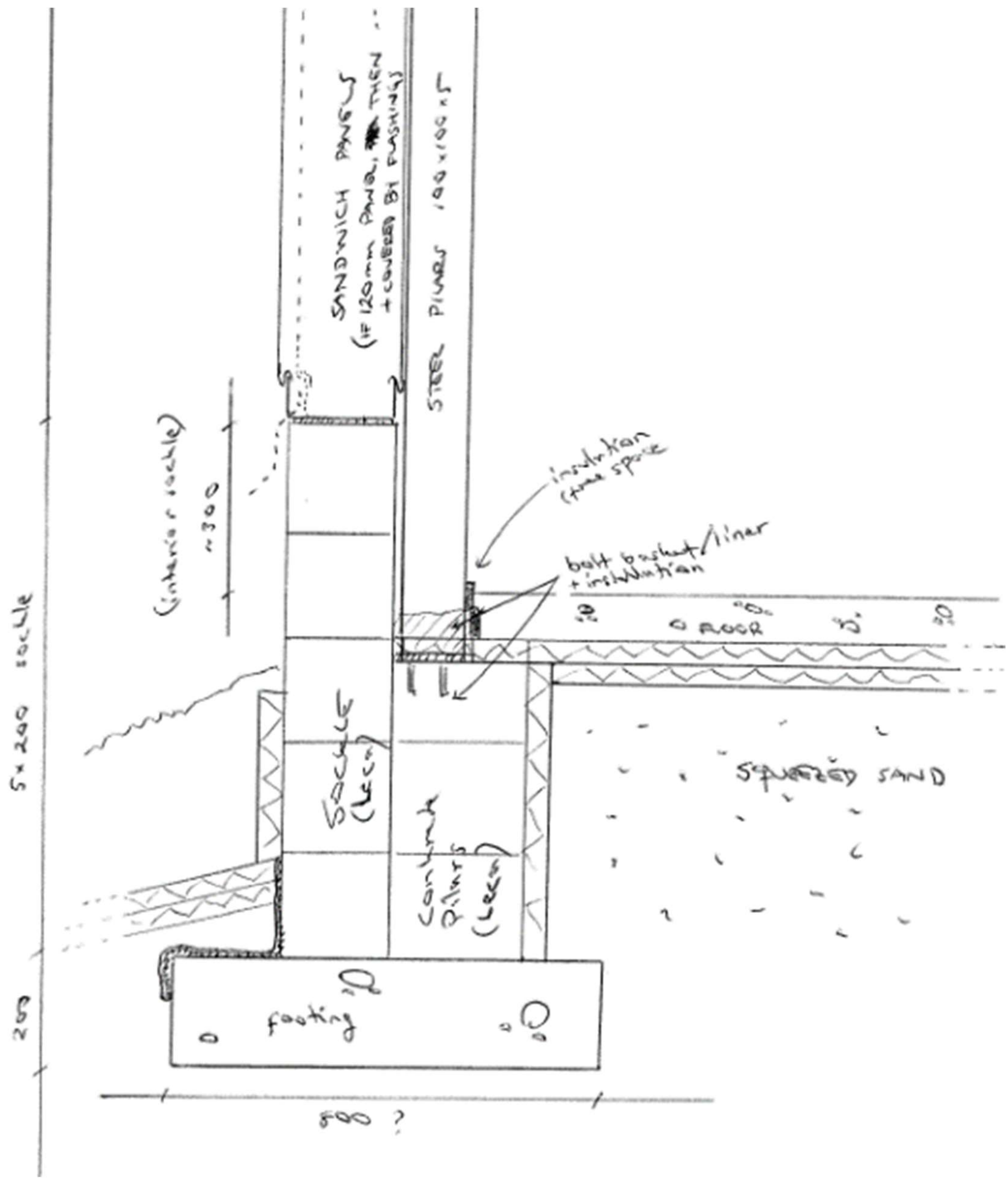


Figure 12. Detail drawing of the foundation

Foundation system modelled in Tekla Structures is shown in Figure 13.

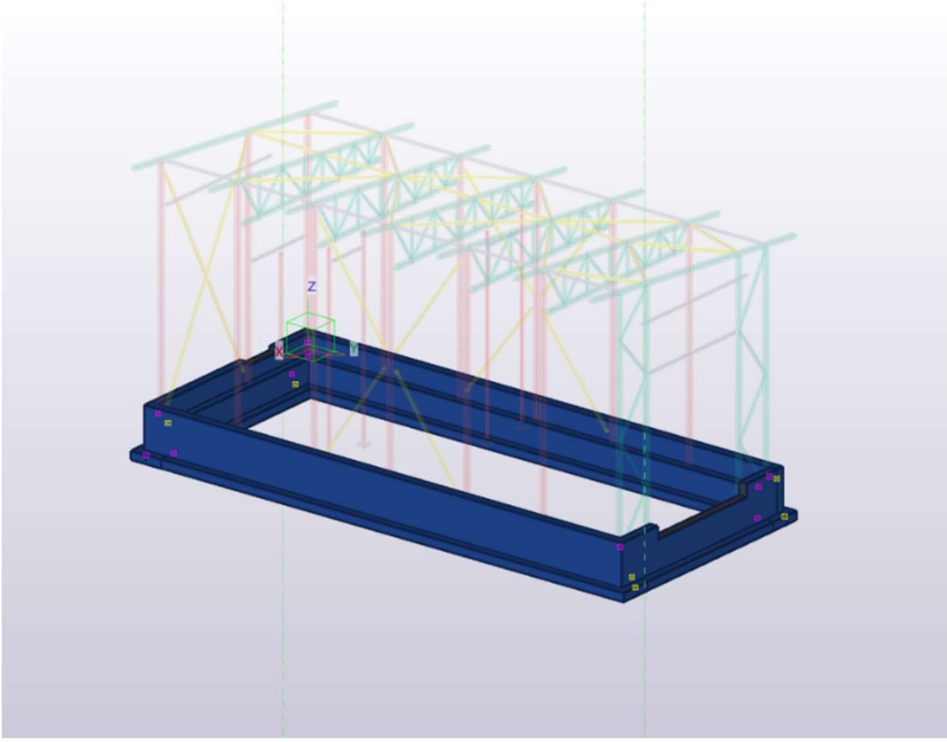


Figure 13. Foundation modelled using Tekla Structures

3.2 Columns

There is a total of 17 load-bearing columns in the structure. Columns that are non-loadbearing are used in door frames and are not considered in the calculations. Figure 14 demonstrates the placement of the columns in the structure.

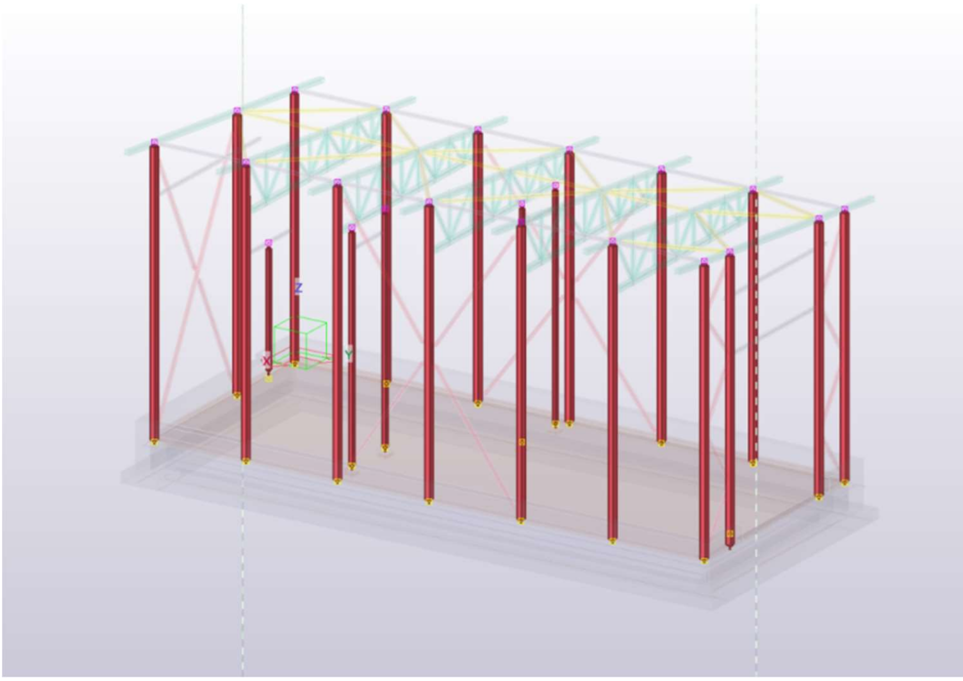


Figure 14. Columns modelled using Tekla Structures

3.3 **Beams**

Beams in the structure are connecting roof trusses and ensuring structural stability and integrity. It is decided to install beams of SHS100x100x5 profiles instead of the roof trusses in both ends of the roof structure. These beams are expected to take half of the external loads acting on the structure. Figure 15 demonstrates beams that are installed in the structure.

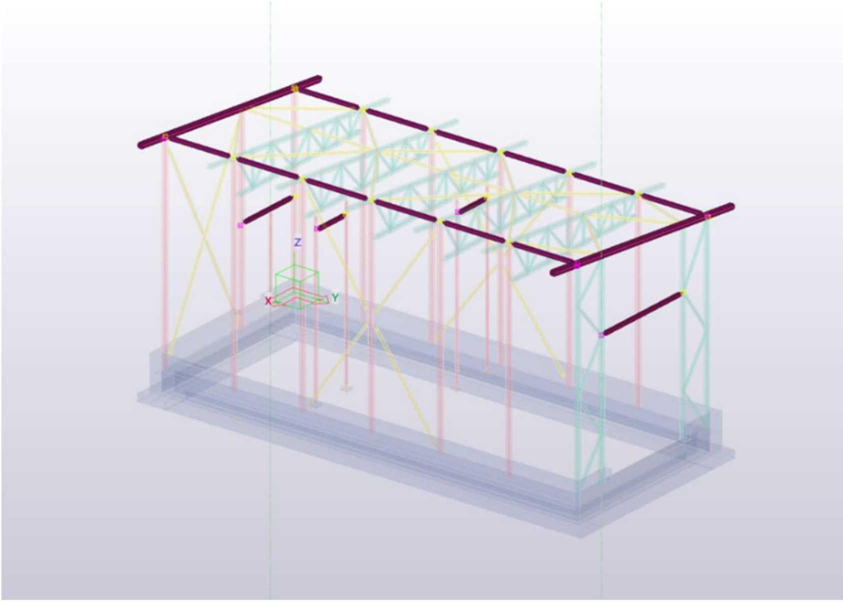


Figure 15. Beams modelled using Tekla Structures

3.4 **Trusses**

3.4.1 Roof trusses

Roof truss is a system of connected elements that are typically formed in the triangular pattern. Trusses are used to support the roof structure as well as take external loadings. Roof truss structure used in Hunter's Hut is shown in Figure 16.

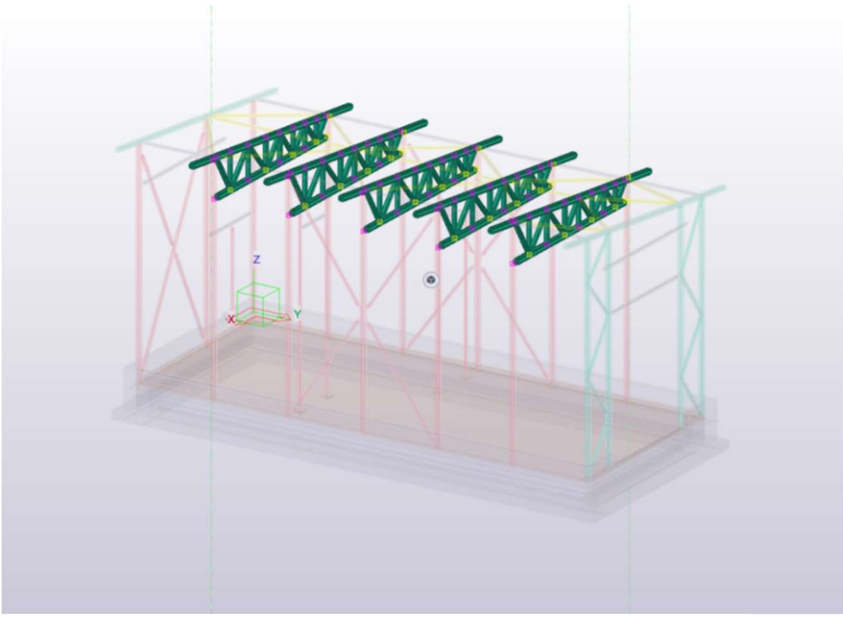


Figure 16. Roof trusses modelled in Tekla Structures

The truss is designed as a KT truss. Lateral support of the upper chord is denser than, for example in K or N trusses, that provides a better buckling resistance of the chord. Usually, truss structure is the most economic option; however, the overlapping joints in the top and bottom chords need bigger fabrication time. The trusses are also to be designed in such a way that it is easy to conduct welding without obstacles. It is important that the smaller angle θ between the truss members is kept not less than 30^{o}. Figure 17 illustrates the layout of the truss used in Hunter's Hut.

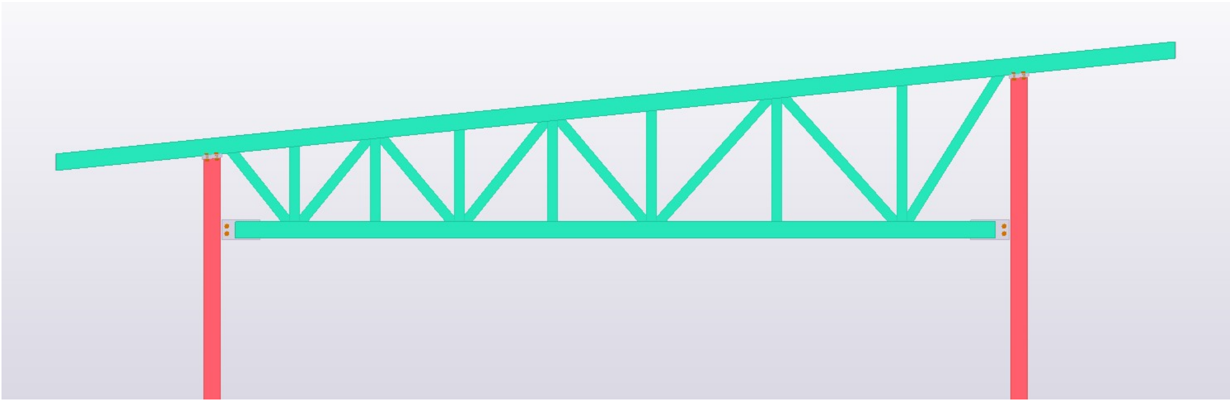


Figure 17. Layout of the truss

3.4.2 Wall trusses

It is also decided to use the truss system in one of the walls of the building. The vertical truss in this case is designed as K truss and follows the same design principles as the roof truss. The side of the building where the vertical truss is installed needs to be braced which is hard to implement because of the large door opening. The vertical truss ensures the stability of the structure and provides a proper transfer of applied horizontal loads to the foundation. Figure 18 illustrates the vertical truss modelled in Tekla Structures.

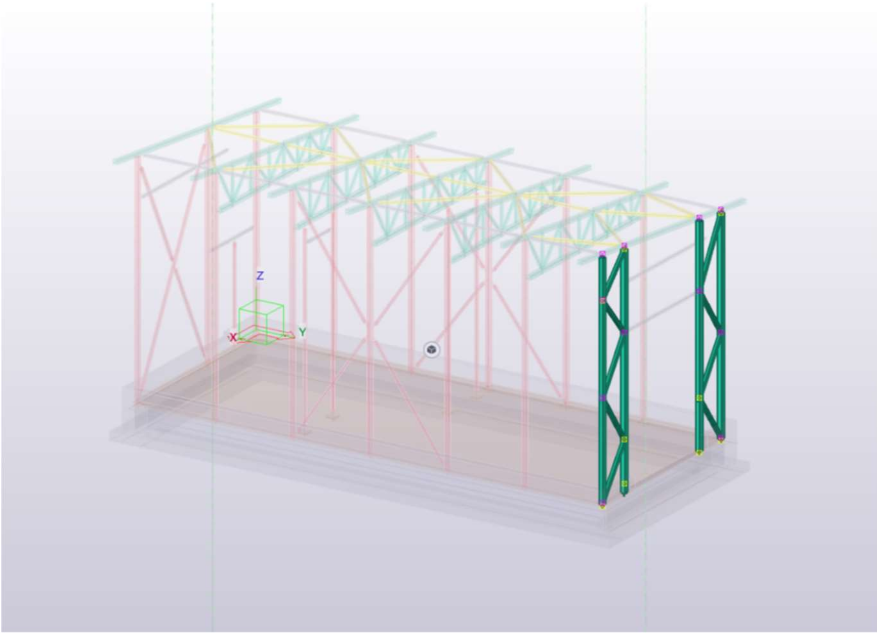


Figure 18. Wall trusses modelled in Tekla Structures

3.5 **Bracing**

This part of the report will give an introduction to different systems that may be used to provide structural stability. The choice of bracing system for the case-study will be discussed.

3.5.1 Basic systems for structural stability

There are different options to give stability to a building due to different loading modes. The basic systems are:

1. *Triangulation.* The system of interconnected beams and columns may be additionally braced by diagonal members. This will split the forces in members and ensure a smoother delivery of loads to the foundation. In order to make an on-site work easier, the connections can be designed as pinned joints. One of the biggest issues with this system is that it might be interacting with openings so it needs a more diligent design. Triangulation is considered to be the most efficient system of all because it transfers only axial forces in members avoiding taking moments. Figure 19 illustrates the principle of work of the triangulation system when the lateral load is applied.

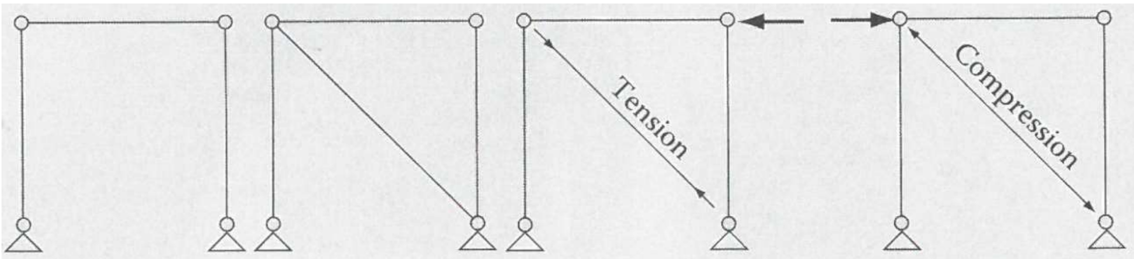


Figure 19. Triangulation system for stabilizing structures (Wyatt, K., Hough, R. 2013)

2. *Rigid Frames.* The structure is able to resist loads by bending of its members if the joints between column and beam (or footing) are not pinned but made rigid. Rigid frames are usually related to the presence of large bending moments. They resist lateral loads mainly by bending action. The connections in rigid frames are designed in such a way that they can take bending moments, shear and axial loads. An illustration of a rigid frame is shown in Figure 20.

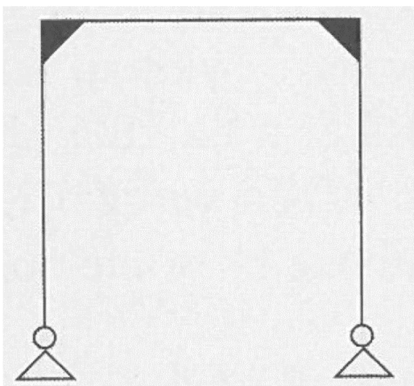


Figure 20. Rigid frame system (Wyatt, K., Hough, R. 2013)

3. *Shear Walls.* Walls can be used to resist lateral loads. This solution is easy to implement in the structures consisting of load bearing walls and slabs rather than columns and beams. Shear walls are called this way because they are placed in the same plane as the applied force and are stressed in shear. In terms of budget, this system is usually the cheapest because there is no need in the installation of other additional structural elements. Figure 21 illustrates how the shear wall acts when taking the lateral load.

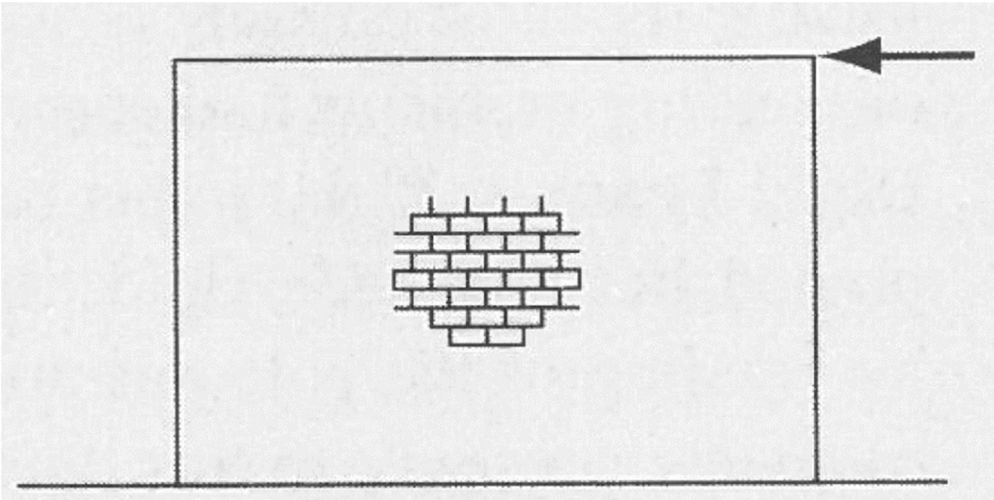


Figure 21. Shear wall for stabilizing structures (Wyatt, K., Hough, R. 2013)

In order to construct a multi-storey structure, the stability of which is ensured, the rigid frames are simply installed on top of each other like it is shown in Figure 22.

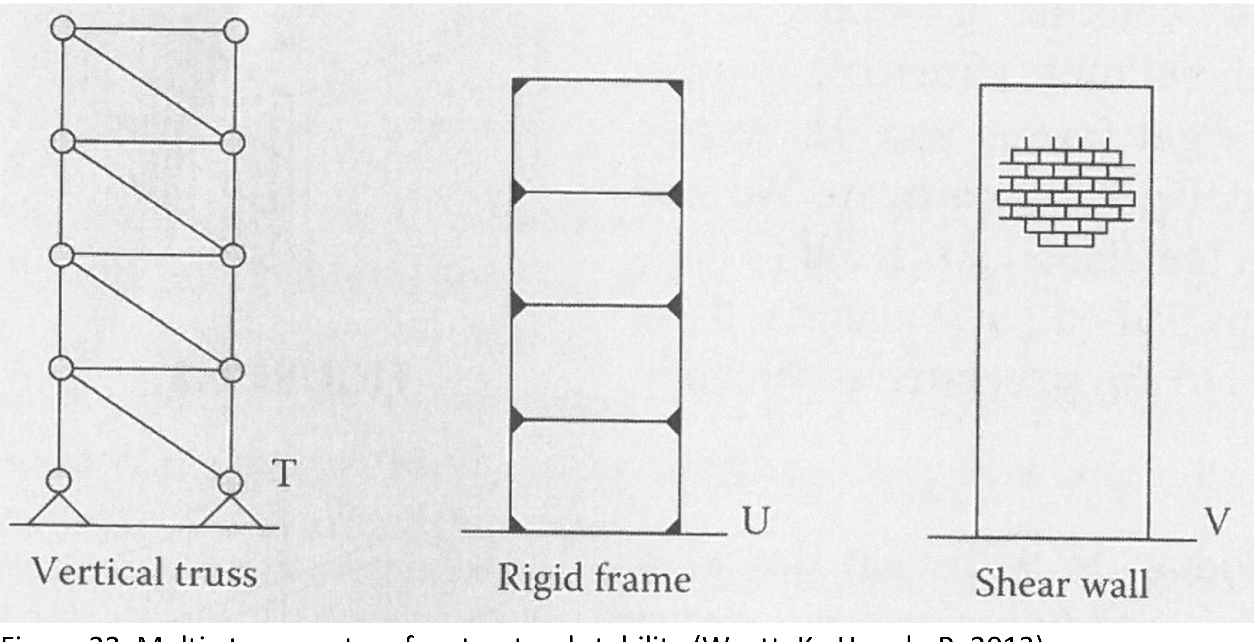


Figure 22. Multi-storey system for structural stability (Wyatt, K., Hough, R. 2013)

3.5.2 Bracing systems

There are generally two orthogonal bracing systems: vertical and horizontal.

1. *Vertical bracing* is used to transfer horizontal loads. The vertical bracing is usually placed between the columns and makes the horizontal forces to be transferred to the foundation as well as it provides the lateral stability of the structure. Figure 23 illustrates the options of vertical bracing installation.

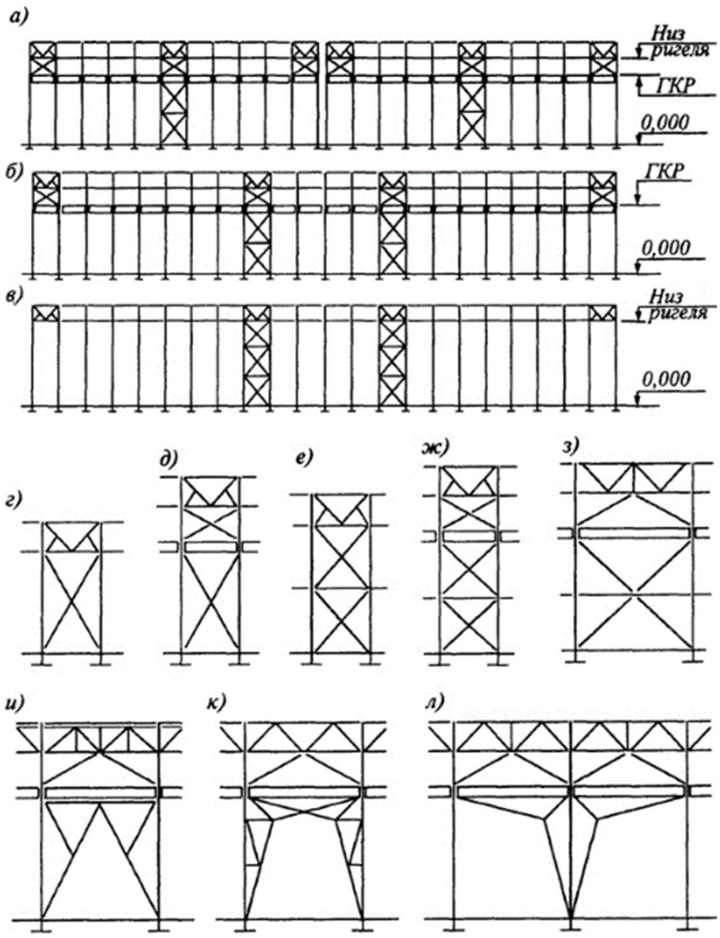


Figure 23. Vertical bracing installation options (Kudishin Y. 2006)

2. *Horizontal bracing* is required to transfer primarily horizontal loads to vertical bracing, columns and, later, foundations. The horizontal bracing usually connects the roof members and provides stability in the upper part of the structure. Options for horizontal bracing installation is shown in Figure 24.

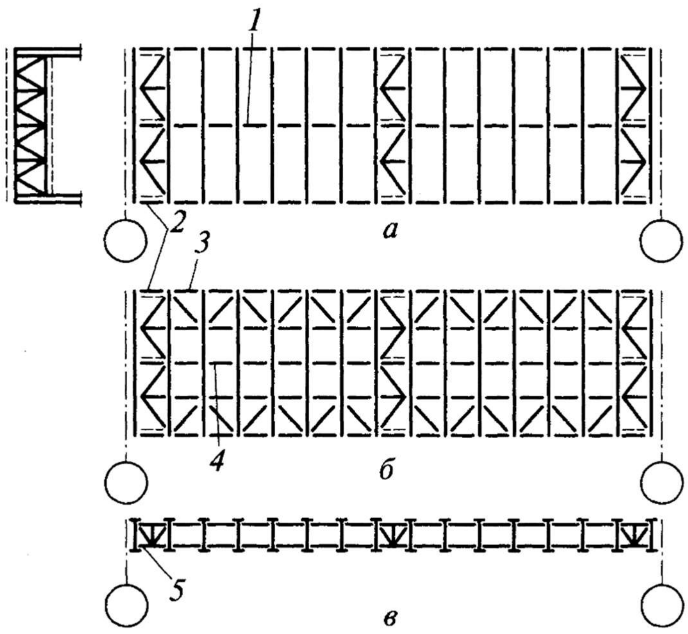


Figure 24. Horizontal bracing installation options (Kudishin Y. 2006)

3.5.3 Load paths

When designing a bracing system, it is important to understand how the structure works as a whole and what the load paths are. Load paths help designers see how the loads e.g. from the wind, are transferred through the structure to the foundation.

Wind loads produce stresses in a number of parts of the structure. In order to get an idea of how the load paths work, a lateral load might be applied to the façade system. Figure 25 illustrates how the loads are carried through the structure. The load starts its path by going vertically to the girts, then it splits to the columns. Part of the load is going straight to the foundation while the other part is transferred to the roof bracing, side frames and then to the foundation.

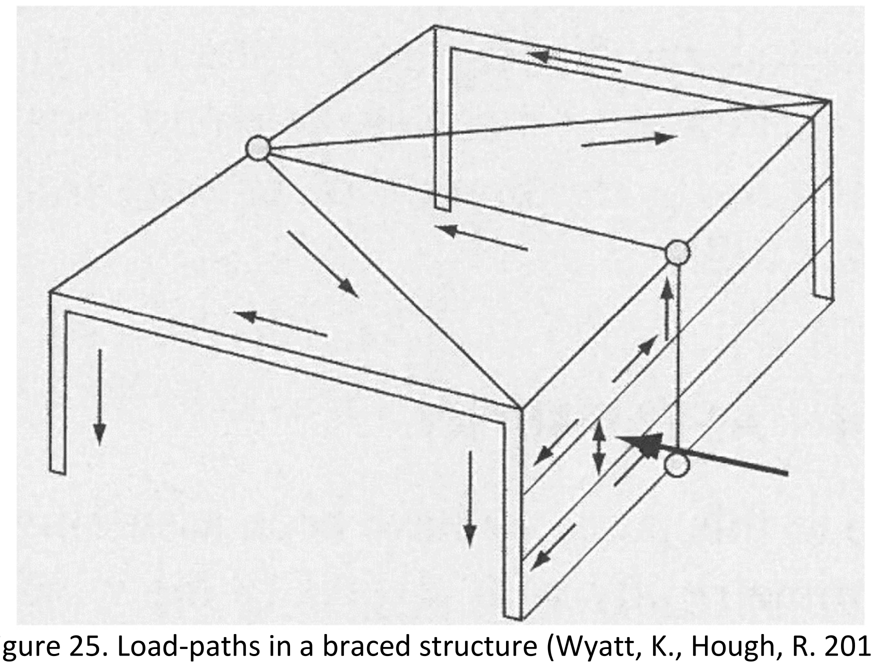


Figure 25. Load-paths in a braced structure (Wyatt, K., Hough, R. 2013)

3.5.4 Choice of bracing for the project

Roof bracing

Since the dimensions of the foundation do not allow to make proper moment-resisting connections, it is decided to design a roof structure that will take most of the horizontal loads. Triangulation method is used in the design of the structure. It is ensured that the load paths are established and there is as less eccentricity created in members as possible. Figure 26 shows how the roof structure is designed. In the figure it may be seen that the members of the bracing systems are connected in the same points in order to provide an efficient transfer of loads from member to member as well as from roof members to columns.

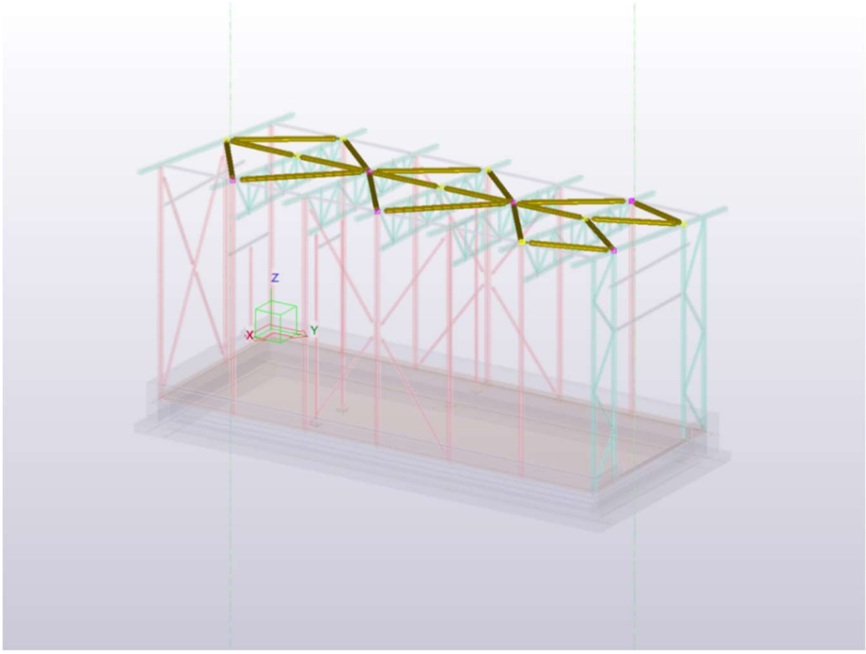


Figure 26. Roof bracing modelled in Tekla Structures

Figure 27 illustrates the top view of the roof's bracing system.

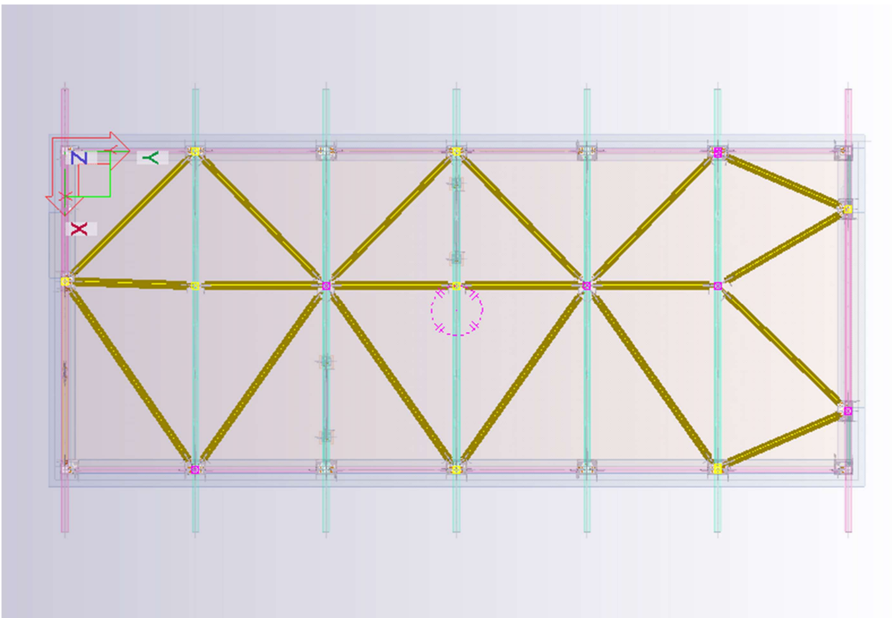


Figure 27. Top view of the roof's bracing

Wall bracing

While there are not many obstacles during the design of the roof bracing, the wall bracing might interfere with the openings of the building. As seen in Figure 28, there are three window openings on the longer façade of the building.

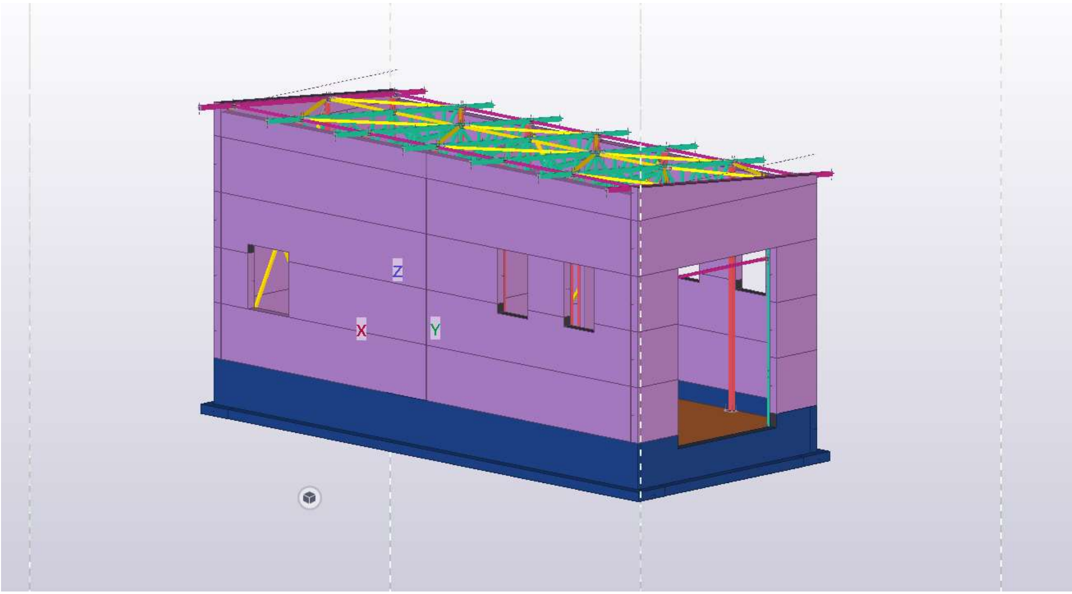


Figure 28. Illustration of the window openings on the longer façade

Since the architectural proposal cannot be changed, it is decided to place the cross-bracing (or X-bracing) on three sides of the structure as shown in Figure 29. Vertical trusses on the fourth side also serve as bracing. Depending on the direction of forces, the brace members need to be resistant to tension.

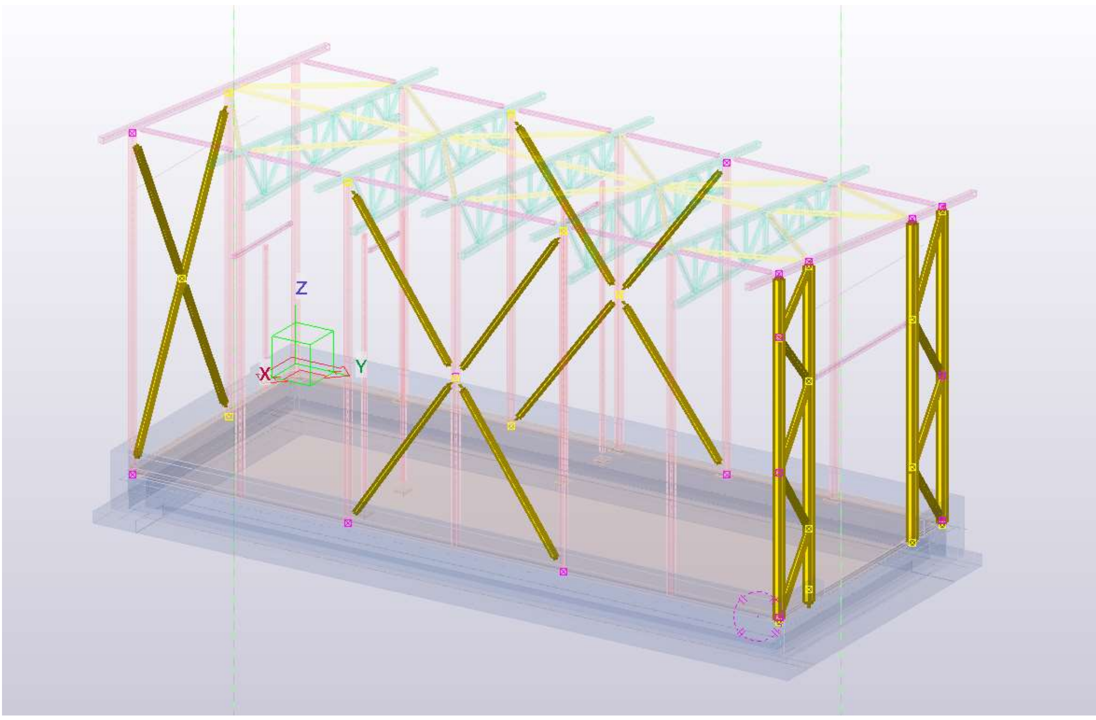


Figure 29. Wall bracing modelled in Tekla Structures

4 LOADS ON THE STRUCTURE

Like any building, Hunter's Hut is exposed to different external loads. The following loads are considered in the calculations:

1. Dead load (self-weight of the building)
2. Live load (load from the carcass of the animal, service of the roof)
3. Snow load
4. Wind load

Loads acting on the structure are calculated according to SFS-EN 1991 along with Finnish National Annexes. All the load calculations are done manually in Mathcad software. Detailed calculation of loads is presented in Appendix 1.

4.1 Dead load

Dead load calculation is based on Eurocode 1, Part 1-1 (SFS-EN 1991-1-1: 2002).
Dead load is the action that is likely to act throughout a given period and for which the variation in magnitude with time is negligible, or for which the variation is always in the same direction (monotonic) until the action attains a certain limit value (SFS-EN 1990, 2002).

Self-weight of the structure is considered as the dead load acting on the structure. Since the load calculations were done before the roof truss structure was designed, it was decided to calculate the weight of the roof with an estimated value $\sigma_{rf} = 5.8kg/m^2$ and the rest of the structure based on the Tekla model.

Weight of the steel frame without roof

The self-weight of the structure is based on the values obtained from the model in Tekla Structures. The following equation is used to calculate the weight of the steel frame without the roof:

$$S_{ws} = 300000kg \times 9.8\frac{m}{s^2}$$

Weight of the roof

Weight of the roof is calculated using the following equation:

$$q_{rf} = A_{rf}\sigma_{rf}\ 9.8\frac{m}{s^2}$$

Where:

A_{rf} is the area of the roof structure (m^2)
σ_{rf} is the estimated value for steel roof structure (kg/m^2)

Thus, the weight of the whole structure can be obtained from the following equation:

$$S_w = S_{ws} + q_{rf}$$

4.2 **Live load**

Live load is the action for which the variation in magnitude with time is neither negligible nor monotonic (SFS-EN 1990, 2002). This corresponds to loads related to the movement of people inside the building, movable furniture, etc.
Load coming from the maintenance of the roof and load from the moving rail system that holds and transfers the carcass of an animal are considered for live load calculations in the project.

Maintenance of the roof

Load from the maintenance of the roof based on EN 1990-1 can be calculated using the following equation:

$$G_{krf} = g_{krf}\ A_{rf}$$

Where:

g_{krf} is the characteristic value for roof maintenance ($g_{krf} = 0.4kN/m^2$)
A_{rf} is the area of the roof structure (m^2)

Imposed load from the rail system can be calculated as following:

$$\sigma_{rs} = 600kg \times 9.8\frac{m}{s^2}$$

Where:

600kg is the estimated weight of three lifted moose carcasses
$9.8\frac{m}{s^2}$ is the acceleration of gravity

4.3 Snow load

The snow load on the roof to be considered varies depending on the following points:

- The location (and orientation) of the building
- The slope of the roof
- The horizontal wind pressure on the roof

Snow load acting on the roof can be calculated using the following equation:

$$s = \mu_1 \, C_e \, C_t \, s_k$$

Where:

μ_1 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 the ground (kN/m^2)

The snow load shape coefficient can be obtained from the graph shown in Figure 30 below.

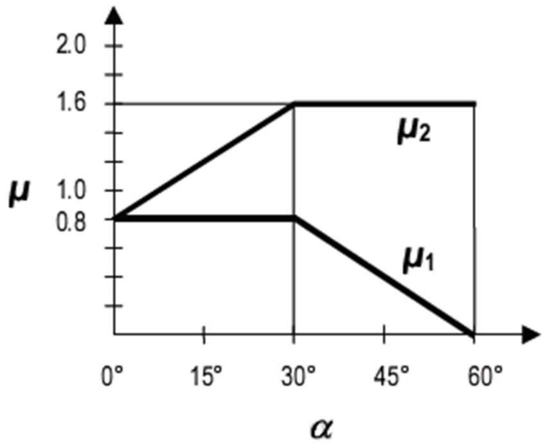


Figure 30. Snow load shape coefficient (SFS-EN 1991-1-3: 2003)

Since the building has a monopitch roof, the load arrangement shown in Figure 31 should be used for the calculation.

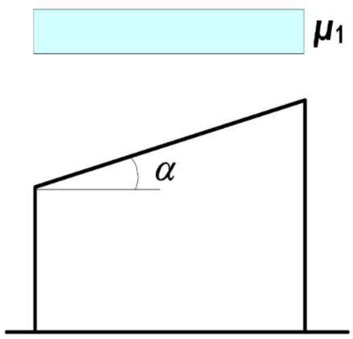


Figure 31. Snow loads shape coefficient – monopitch roof (SFS-EN 1991-1-3: 2003)

Roof's angle in the project is 5.71°; therefore the snow load shape coefficient $\mu_1 = 0.8$

Table 2 shows the conditions based on the angle of the roof to determine the snow load shape coefficients.

Table 2. Snow load shape coefficients (SFS-EN 1991-1-3:2003)

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	--

Finnish National Annex recommends $C_t = 1$ as a value for the thermal coefficient.

The exposure coefficient C_e can be obtained from Table 3 below. The location of the building is considered as "Normal topography".

Table 3. Recommended values of C_e for different topographies (SFS-EN 1991-1-3: 2003)

Topography	C_e
Windswept [a]	0,8
Normal [b]	1,0
Sheltered [c]	1,2

[a] *Windswept topography*: flat unobstructed areas exposed on all sides without, or little shelter afforded by terrain, higher construction works or trees.

[b] *Normal topography*: areas where there is no significant removal of snow by wind on construction work, because of terrain, other construction works or trees.

[c] *Sheltered topography*: areas in which the construction work being considered is considerably lower than the surrounding terrain or surrounded by high trees and/or surrounded by higher construction works.

According to Eurocode 1 (SFS-EN 1991-1-3: 2003) and Finnish National Annex, the characteristic value of snow load on the ground in Hämeenlinna area, s_k is 2.5 kN/m^2. A map of snow loads in Finland in shown in Figure 32.

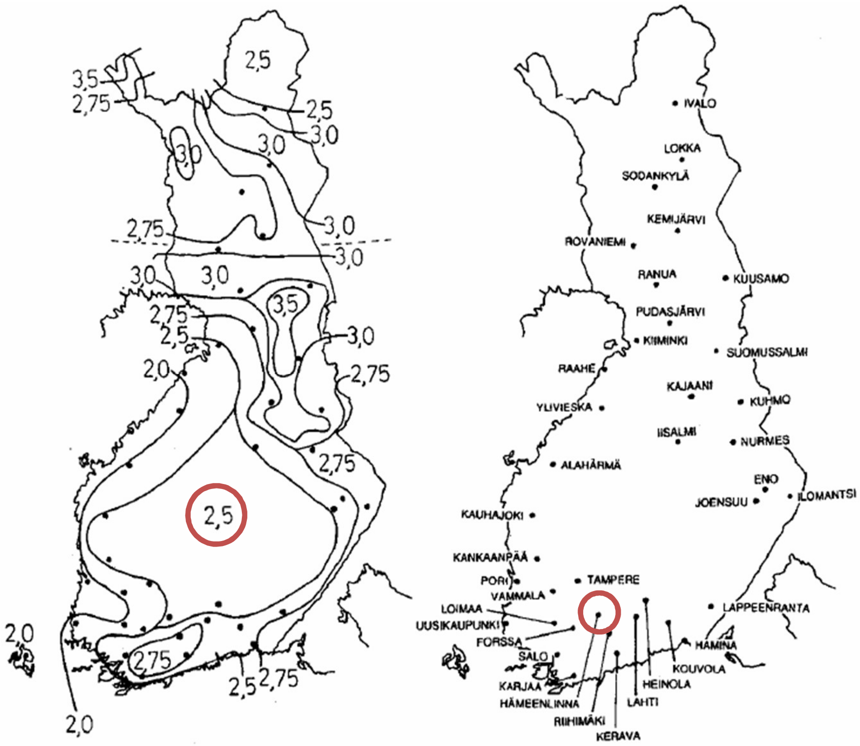


Figure 32. Snow loads on the ground in Finland (SFS-EN 1991-1-3: 2003)

4.4 Wind pressure

Wind pressure on buildings is based on EN 1991-1-4 along with Finnish National Annex. In order to calculate the wind pressure on the building, several steps that are mentioned in this report are to be followed.

Basic wind velocity

The basic wind velocity is calculated from the following equation:

$$v_b = v_{b,0}\, c_{dir}\, c_{season}$$

Where:

$v_{b,0}$ is the fundamental value of the basic wind velocity (m/s)
c_{dir} is the direction factor, recommended value 1.0
c_{season} is the season factor, recommended value 1.0

Therefore

$$v_b = v_{b,0}$$

It is assumed that the basic wind velocity v_b (obtained from meteorological data) is 21 m/s

Mean wind velocity

The mean wind velocity at height z above the terrain depends on the roughness and orography of the terrain and on the basic wind velocity v_b, and may be calculated from the following equation:

$$V_m(z) = v_b\, c_r(z)\, c_o(z)$$

Where:

$c_r(z)$ is the roughness factor of the ground roughness of the terrain upwind of the structure in the wind direction considered
$c_o(z)$ is terrain orography factor

Terrain roughness factor

The roughness factor at a height z can be calculated using the following equation:

$$C_r(z) = k_r\, ln(z/z_o)$$

Where:

z is the height of the structure above ground level (m)
z_o is the is the roughness height (m)
k_r is the terrain factor depending on the roughness length z_o

Roughness length z_o can be calculated using the following equation:

$$k_r = 0.19\left(z_o/z_{o,II}\right)^{0.07}$$

Where:

$z_{o,II}$ is the roughness height z_o at terrain category II

Terrain category III is considered for the project, so according to Table 4 below, $z_{o,II} = 0.05$ and $z_o = 0.3$

Table 4. Terrain categories and terrain parameters (SFS-EN 1991-1-4: 2005)

Terrain category		z_0 m	z_{min} m
0	Sea or coastal area exposed to the open sea	0,003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1,0	10
NOTE: The terrain categories are illustrated in A.1.			

Terrain orography factor

Where the orography (e.g. hills or cliffs) increases the wind velocity by more than 5%, the effects of this should be taken into account using the orography factor $c_o(z)$ (Ghosh, K. M. (2010). In case of Hunter's Hut, the terrain is flat so the orography may be neglected. Therefore, $c_o(z) = 1$

Wind turbulence

Wind turbulence can be calculated using the following equation:

$$I_v(z) = \frac{k_I}{c_0(z)\ln(\frac{z}{z_0})}$$

Where:

k_I is the tubulence factor, recommended value 1.0

Peak velocity pressure

The peak velocity pressure at a height z is given by the following equation:

$$q_p(z) = [1 + 7\, I_v(z)]\; 0.5\; \rho\; v_m^2(z)$$

Where:

ρ is the density of air ($\rho = 1.25\text{kg/m}^3$)

Wind pressures on surfaces

The external pressure acting on the surfaces is given by the following equation:

$$w_e = q_p(z_e)c_{pe}$$

Where:

w_e is the external pressure (N/m^2)
$q_p(z_e)$ is the peak velocity pressure (N/m^2)
z_e is the reference height for the external pressure (m)
c_{pe} is the pressure coefficient for the external pressure

The value of c_{pe} depends on the ratio *h/d* for the structure, where *h* is the height of the building up to the apex and *d* is the depth of the building.

Zones of action and wind direction for the external wind pressure are shown in Figure 33 and Figure 34 below.

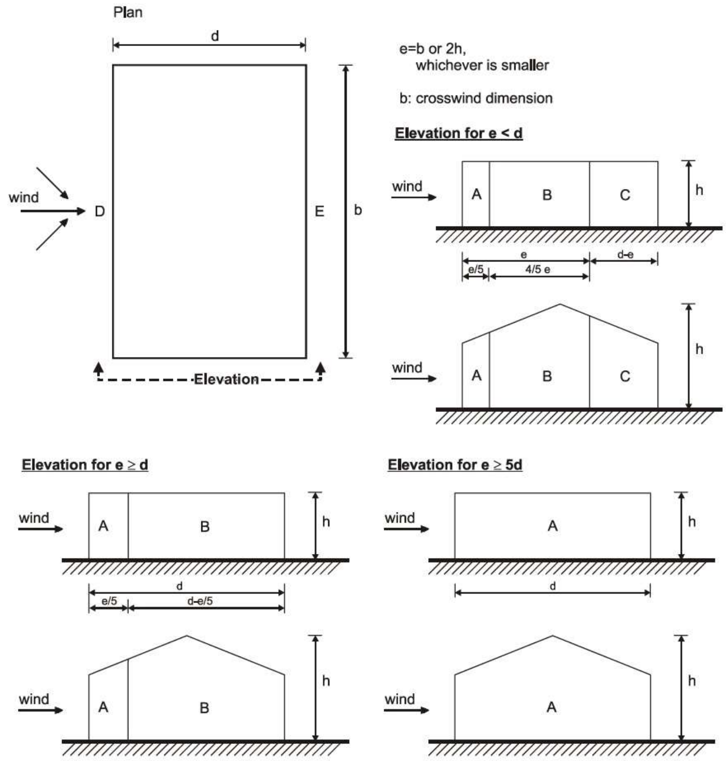


Figure 33. Key for vertical walls (SFS-EN 1991-1-4:2005)

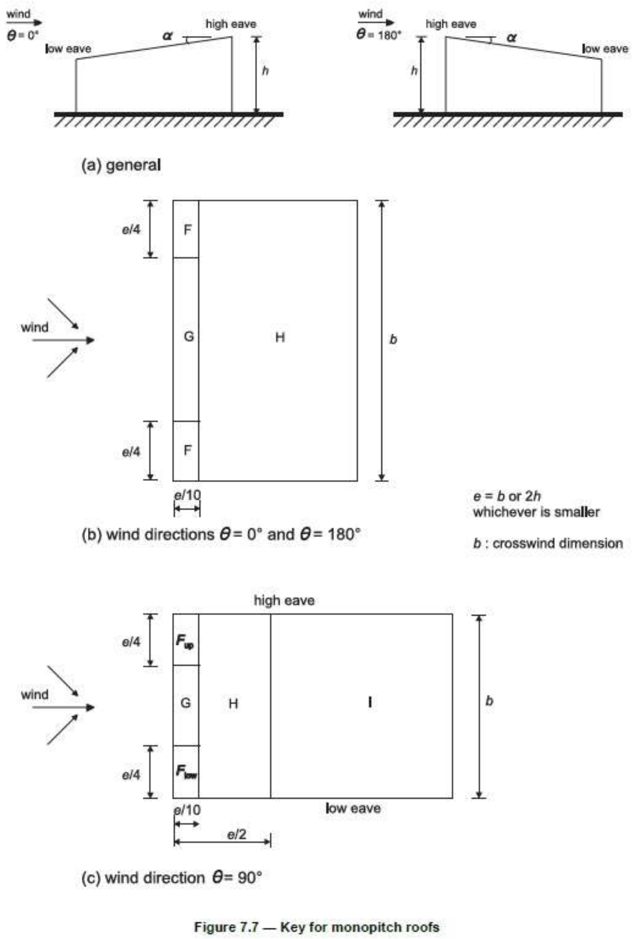


Figure 34. Key for monopitched roofs (SFS-EN 1991-1-4: 2005)

External pressure coefficients can be obtained from Table 5.

Table 5. External pressure coefficients for monopitch roofs (SFS-EN 1991-1-4: 2005)

Pitch Angle α	Zone for wind direction $\theta = 90°$									
	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 $\theta = 0°$ (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

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	

The wind pressure acting on the internal surfaces is expressed in the following form:

$$w_i = q_p(z_i)c_{pi}$$

Where:

w_i is the internal pressure (N/m^2)
$q_p(z_i)$ is the peak velocity pressure (N/m^2)
z_i is the reference height for the internal pressure (m)
c_{pi} is the internal pressure coefficient

Wind internal pressure action is not considered in the calculations for the Hunter's Hut since it is assumed that in case of storm, all windows and doors of the building are shut.

5 LOAD COMBINATIONS

Load combinations are calculated according to EN 1990:2002 "Eurocode. Basis of structural design" together with Finnish National Annex.

5.1 Ultimate limit state (ULS)

Ultimate limit states relate to the safety of the structure as a whole or of part of it. Relevant partial safety factors are applied for checking the strength of the structure. Thus, γ_G is the partial factor for permanent loads, and γ_Q is the partial factor for variable loads. The resulting factored loads should be applied in the most unfavourable combination so that the load-carrying capacity of the members sustains adequate strength without allowing any collapse.

Design values of actions for design of structural members not involving geotechnical actions are obtained from Table 6.

Table 6. Design values of actions (STR/GEO) (SFS-EN 1990: 2002, NA)

Persistent and transient design situations	Permanent actions		Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10a)	$1{,}35 K_{FI}\, G_{kj,sup}$	$0{,}90\, G_{kj,inf}$			
(Eq. 6.10b)	$1{,}15 K_{FI}\, G_{kj,sup}$	$0{,}90\, G_{kj,inf}$	$1{,}5 K_{FI}\, Q_{k,1}$		$1{,}5\, K_{FI}\, \psi_{0,i} Q_{k,i}$

(*) Variable actions are those considered in Table A1.1

Note 1. This can be expressed as a design formula in such a way that the most unfavourable of the two following expressions is used as a combination of loads when it should be noted that the latter expression only contains permanent loads:

$$\begin{cases} 1{,}15 K_{FI}\, G_{kj,sup} + 0{,}9\, G_{kj,inf} + 1{,}5 K_{FI}\, Q_{k,1} + 1{,}5 K_{FI} \sum_{i>1} \psi_{0,i}\, Q_{k,i} \\ 1{,}35 K_{FI}\, G_{kj,sup} + 0{,}9\, G_{kj,inf} \end{cases}$$

K_{FI} depends on the reliability class given in table B2 of Annex B as follows:

In reliability class RC3 $K_{FI} = 1{,}1$
In reliability class RC2 $K_{FI} = 1{,}0$
In reliability class RC1 $K_{FI} = 0{,}9$.

The reliability classes are associated with the consequence classes CC3 … CC1 given in Annex B.

Note 2: See also standards SFS-EN 1992 to SFS-EN 1999 for γ values to be used for imposed deformations.

Note 3: The characteristic values of all permanent actions from one source are multiplied by $\gamma_{G,sup}$ if the total resulting action effect is unfavourable and $\gamma_{Q,inf}$ if the total resulting action effect is favourable. For example, all actions originating from the self weight of the structure may be considered as coming from one source; this also applies if different materials are involved.

Note 4: For particular verifications, the values for γ_G and γ_Q may be subdivided into γ_g and γ_q and the model uncertainty factor γ_{sd}. A value of $\gamma_{sd} = 1{,}05 \ldots 1{,}15$ can be used in most common cases.

Note 5: In respect of geotechnical design of foundations, see standard SFS-EN 1997-1 with its National Annex.

Design values of actions for static equilibrium are obtained from Table 7.

Table 7. Design values of actions (EQU) (SFS-EN 1990: 2002, NA)

Persistent and transient design situations	Permanent actions		Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	1,10 $K_{FI}G_{kj,sup}$	0,90 $G_{kj,inf}$	1,50 $K_{FI}Q_{k,1}$		1,50K_{FI} $\psi_{0,i}Q_{k,I}$

(*) Variable actions are those considered in Table A1.1

K_{FI} depends on the reliability class given in table B2 of Annex B as follows:

In reliability class RC3 $K_{FI} = 1,1$
In reliability class RC2 $K_{FI} = 1,0$
In reliability class RC1 $K_{FI} = 0,9$.

The reliability classes are associated with the consequence classes CC3 … CC1 given in Annex B.

5.2 **Serviceability limit state (SLS)**

Serviceability limit states define the limit beyond which the specified service conditions are no longer fulfilled. In serviceability limit states, the specified loads are generally unfactored except in the case of combinations of imposed loads and wind loads

Design values of actions may be obtained from Table 8 below.

Table 8. Design values of actions for use in the combination of actions (SFS-EN 1990: 2002)

Combination	Permanent actions G_d		Variable actions Q_d	
	Unfavourable	Favourable	Leading	Others
Characteristic	$G_{kj,sup}$	$G_{kj,inf}$	$Q_{k,1}$	$\psi_{0,i}Q_{k,i}$
Frequent	$G_{kj,sup}$	$G_{kj,inf}$	$\psi_{1,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$
Quasi-permanent	$G_{kj,sup}$	$G_{kj,inf}$	$\psi_{2,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$

For load combinations using the ULS and SLS methods, values of partial factors ψ may be obtained from Table 9 below.

Table 9. Recommended values ψ of factors for buildings (SFS-EN 1990: 2002, NA)

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A : domestic, residential areas	0,7	0,5	0,3
Category B : office areas	0,7	0,5	0,3
Category C : congregation areas	0,7	0,7	0,6
Category D : shopping areas	0,7	0,7	0,6
Category E : storage areas	1,0	0,9	0,8
Category F : traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
NOTE The ψ values may be set by the National annex. * For countries not mentioned below, see relevant local conditions.			

Consequence class of the building is defined as CC2 and is obtained from the Table 10 below.

Table 10. Definition of consequence classes (SFS-EN 1990: 2002, NA)

Consequences Class	Description	Examples of buildings and civil engineering works
CC3	**High** consequence for loss of human life, or economic, social or environmental consequences **very great**	The load bearing system[1)] with its bracing parts in buildings which are often occupied by a large number of people for example – residential, office and business buildings with more than 8 storeys[2)] – concert halls, theatres, sports and exhibitions halls, spectator stands – heavily loaded buildings or buildings with long spans. Special structures such as high masts and towers. Ramps as well as embankments and other structures in areas of fine-grained soils in environments sensitive to adverse effects of displacements.
CC2	**Medium** consequence for loss of human life, economic, social or environmental consequences **considerable**	Buildings and structures not belonging to classes CC3 or CC1.
CC1	**Low** consequence for loss of human life and economic, social or environmental consequences **small or negligible**	1- and 2-storey buildings, which are only occasionally occupied by people for example warehouses. Structures, which when damaged, don't pose major risk, for example – low basement floors without cellar rooms – roofs, under which there is a load bearing floor and the loft is low – walls, windows, floors and other similar structures, which are mainly loaded horizontally by air pressure difference and which do not have a load bearing or stabilizing function in the load bearing system. – sheeting in structural classes II and III of SFS-EN 1993-1-3 – sheeting in structural class I of SFS-EN 1993-1-3 for loads vertical to surface causing bending [3)].

6 DLUBAL RFEM

Dlubal RFEM is a Finite Element Method – based software that is able to perform structural analysis. The workflow typically consists of six steps:

1. Building the geometry of the structure
2. Defining the boundary conditions such as supports and releases
3. Inputting loads and load combinations
4. Conducting structural analysis
5. Design of structural members based on utility ratios

The maximum internal forces in members obtained from the software are later used in resistance checks of joints.
The RFEM structural analysis that is performed for Hunter's Hut is described below step by step.

6.1 Geometry of the structure

The analysis was done for a three-dimensional structure. The shape and dimensions of the structure are based on the model built in Tekla Structures. The structure consists of lines that are connected with nodes. Each line contains information on the member it represents. Members have beginnings and ends that help to define the releases.

Figure 35 below illustrates the structural model built in Dlubal RFEM

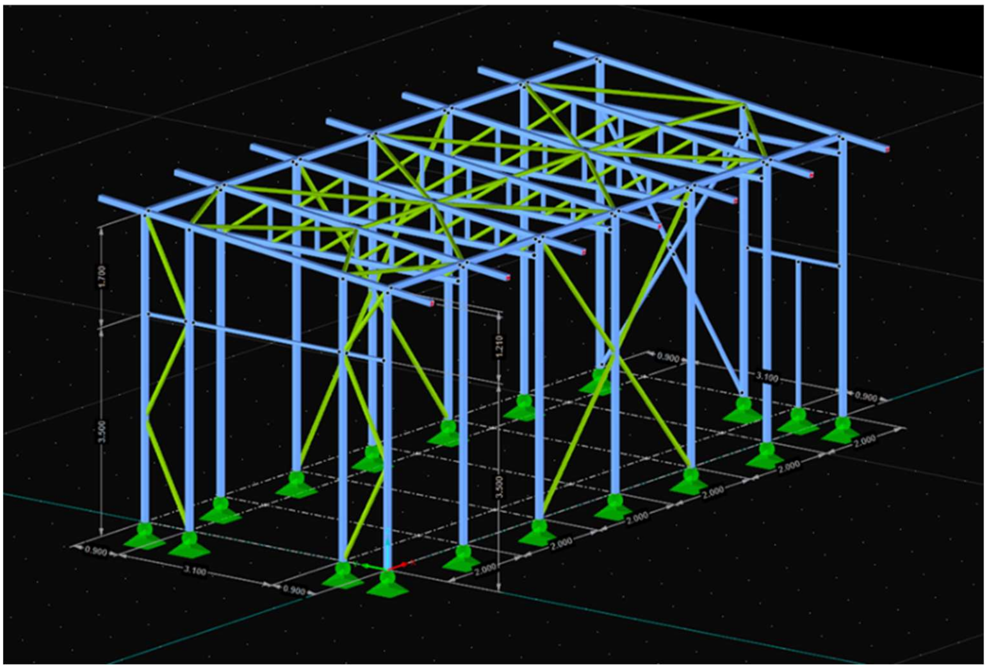


Figure 35. Three-dimensional structure modelled in RFEM

6.2 Boundary conditions of the structure

Supports and releases are defined after the geometry is built. Member releases are assigned to the beginning and the end nodes of a member. Border conditions must represent how the supports and connections will act when the loads are applied to the structure.

Since the concrete foundation is relatively narrow compared to the profile of the columns, it is assumed that the moment resistance of the foundation is neglectable. Therefore, the connection between the column and foundation is considered as pinned in the structural analysis. Figure 36 shows the pinned support assigned to the structure in RFEM.

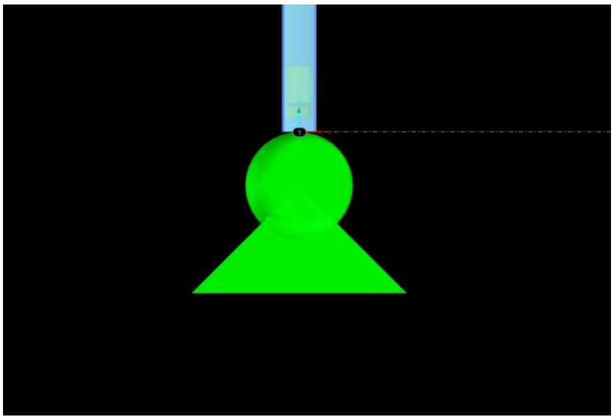


Figure 36. Illustration of pinned connection assigned in RFEM

Figure 37 shows the support conditions settings in the software.

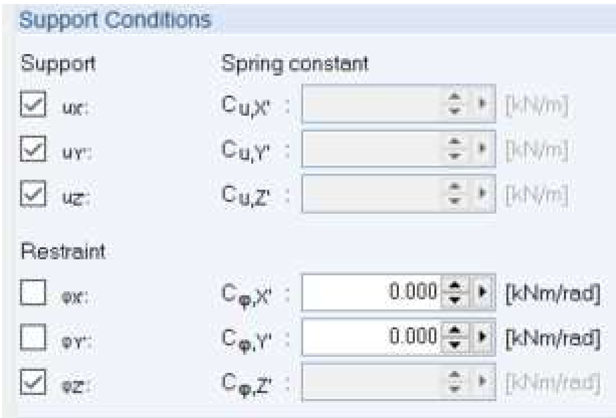


Figure 37. Settings for support conditions in Dlubal RFEM

Releases for other members are assigned to their nodes in a similar way depending on their function in the structure.

6.3 Loads and load combinations

Loads that are calculated manually are defined as shown in Table 11.

Table 11. Load cases input in Dlubal RFEM

	A	B	C	D	E	F	G
Load Case	Load Case Description	To Solve	EN 1990 | SFS Action Category	Self-Weight - Factor in Direction Active	X	Y	Z
LC1	Self-weight	☑	G Permanent	☑	0.000	0.000	-1.000
LC2	Snow	☑	Qs Snow - s-k < 2.75 kN/m^2	☐			
LC3	Wind in w+ AB	☑	Qw Wind	☐			
LC4	Wind in w- AB	☑	Qw Wind	☐			
LC5	Wind in w+ CD	☑	Qw Wind	☐			
LC6	Wind in w- CD	☑	Qw Wind	☐			
LC7	Wind in w+ BC	☑	Qw Wind	☐			
LC8	Wind in w- BC	☑	Qw Wind	☐			
LC9	Wind in w+ DA	☑	Qw Wind	☐			
LC10	Wind in w- DA	☑	Qw Wind	☐			
LC11	Imposed load	☑	Qi A Imposed - Category A: domestic, resider	☐			
LC12	Moose load	☑	Gq Permanent/Imposed	☐			

When applying wind loads to the structure, it is important to define which side of the structure is under pressure. Corners of the building are marked with letters A, B, C and D in the software. Thus, wind is acting perpendicularly on the sides AB, CD, BC and DA as shown in Figure 38.

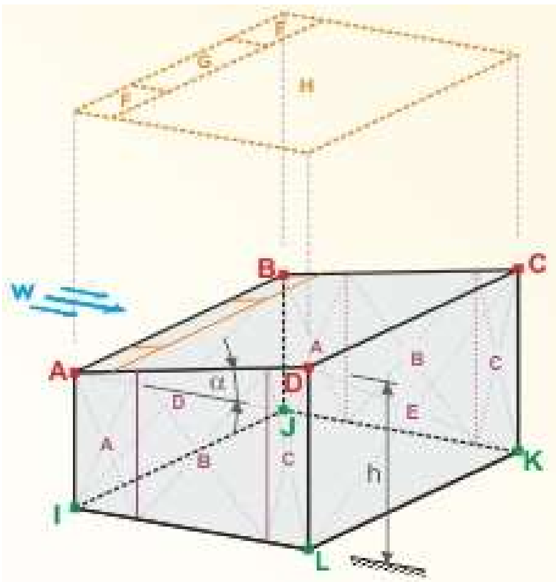


Figure 38. Designations of sides of the structure and pressure zones in Dlubal RFEM

Total of 32 load combinations are used for structural analysis. Load combinations input is shown in Table 12 below.

Table 12. Load combinations input in Dlubal RFEM

	A	B	C	D	E	F	G	H	I	J	K	L	M
Load Combin.	DS	Load Combination Description	To Solve	LC.1 Factor	No.	LC.2 Factor	No.	LC.3 Factor	No.	LC.4 Factor	No.	LC.5 Factor	No.
CO1		Main wind +AB	☑	1.35	G LC1	1.05	Qs LC2	1.50	Qw LC3	1.05	Gq LC12		
CO2		Main wind -AB	☑	1.35	G LC1	1.05	Qs LC2	1.50	Qw LC4	1.05	Gq LC12		
CO3		Main wind +CD	☑	1.35	G LC1	1.05	Qs LC2	1.50	Qw LC5	1.05	Gq LC12		
CO4		Main wind -CD	☑	1.35	G LC1	1.05	Qs LC2	1.50	Qw LC6	1.05	Gq LC12		
CO5		Main wind +BC	☑	1.35	G LC1	1.05	Qs LC2	1.50	Qw LC7	1.05	Gq LC12		
CO6		Main wind -BC	☑	1.35	G LC1	1.05	Qs LC2	1.50	Qw LC8	1.05	Gq LC12		
CO7		Main wind +DA	☑	1.35	G LC1	1.05	Qs LC2	1.50	Qw LC9	1.05	Gq LC12		
CO8		Main wind -DA	☑	1.35	G LC1	1.05	Qs LC2	1.50	Qw LC10	1.05	Gq LC12		
CO9		Main Snow +AB	☑	1.35	G LC1	1.50	Qs LC2	0.90	Qw LC3	1.05	Gq LC12		
CO10		Main Snow -AB	☑	1.35	G LC1	1.50	Qs LC2	0.90	Qw LC4	1.05	Gq LC12		
CO11		Main Snow +CD	☑	1.35	G LC1	1.50	Qs LC2	0.90	Qw LC5	1.05	Gq LC12		
CO12		Main Snow -CD	☑	1.35	G LC1	1.50	Qs LC2	0.90	Qw LC6	1.05	Gq LC12		
CO13		Main Snow +BC	☑	1.35	G LC1	1.50	Qs LC2	0.90	Qw LC7	1.05	Gq LC12		
CO14		Main Snow -BC	☑	1.35	G LC1	1.50	Qs LC2	0.90	Qw LC8	1.05	Gq LC12		
CO15		Main Snow +DA	☑	1.35	G LC1	1.50	Qs LC2	0.90	Qw LC9	1.05	Gq LC12		
CO16		Main Snow -DA	☑	1.35	G LC1	1.50	Qs LC2	0.90	Qw LC10	1.05	Gq LC12		
CO17		Main moose +AB	☑	1.35	G LC1	1.05	Qs LC2	0.90	Qw LC3	1.50	Gq LC12		
CO18		Main moose -AB	☑	1.35	G LC1	1.05	Qs LC2	0.90	Qw LC4	1.50	Gq LC12		
CO19		Main moose +CD	☑	1.35	G LC1	1.05	Qs LC2	0.90	Qw LC5	1.50	Gq LC12		
CO20		Main moose -CD	☑	1.35	G LC1	1.05	Qs LC2	0.90	Qw LC6	1.50	Gq LC12		
CO21		Main moose +BC	☑	1.35	G LC1	1.05	Qs LC2	0.90	Qw LC7	1.50	Gq LC12		
CO22		Main moose -BC	☑	1.35	G LC1	1.05	Qs LC2	0.90	Qw LC8	1.50	Gq LC12		
CO23		Main moose +DA	☑	1.35	G LC1	1.05	Qs LC2	0.90	Qw LC9	1.50	Gq LC12		
CO24		Main moose -DA	☑	1.35	G LC1	1.05	Qs LC2	0.90	Qw LC10	1.50	Gq LC12		
CO25		Main live load +AB	☑	1.35	G LC1	1.05	Qs LC2	0.90	Qw LC3	1.50	Qi A LC11	1.05	Gq LC12
CO26		Main live load -AB	☑	1.35	G LC1	1.05	Qs LC2	0.90	Qw LC4	1.50	Qi A LC11	1.05	Gq LC12
CO27		Main live load +CD	☑	1.35	G LC1	1.05	Qs LC2	0.90	Qw LC5	1.50	Qi A LC11	1.05	Gq LC12
CO28		Main live load -CD	☑	1.35	G LC1	1.05	Qs LC2	0.90	Qw LC6	1.50	Qi A LC11	1.05	Gq LC12
CO29		Main live load +BC	☑	1.35	G LC1	1.05	Qs LC2	0.90	Qw LC7	1.50	Qi A LC11	1.05	Gq LC12
CO30		Main live load -BC	☑	1.35	G LC1	1.05	Qs LC2	0.90	Qw LC8	1.50	Qi A LC11	1.05	Gq LC12
CO31		Main live load +DA	☑	1.35	G LC1	1.05	Qs LC2	0.90	Qw LC9	1.50	Qi A LC11	1.05	Gq LC12
CO32		Main live load -DA	☑	1.35	G LC1	1.05	Qs LC2	0.90	Qw LC10	1.50	Qi A LC11	1.05	Gq LC12

When the load cases and load combinations are defined, the loads may be assigned to the structure. Figure 39 shows the snow load applied to the structure.

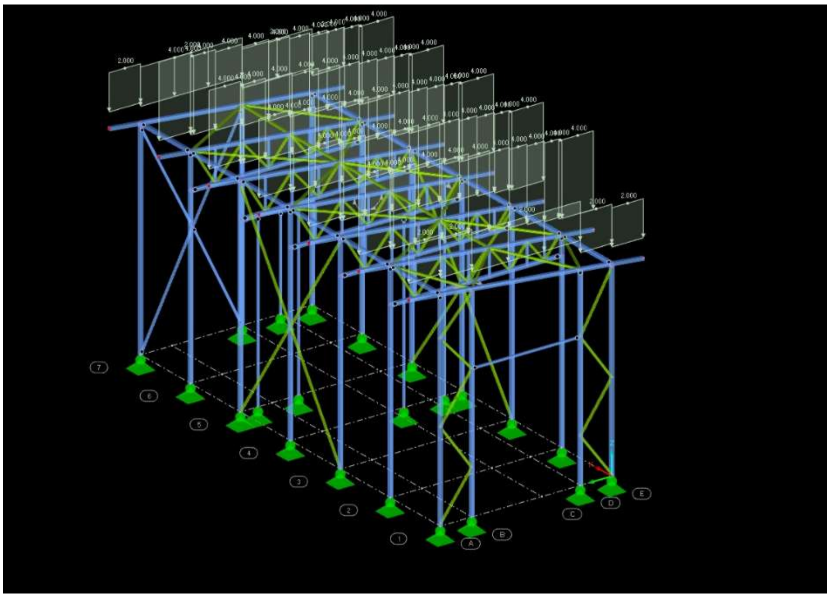


Figure 39. Snow load assigned to the structure in Dlubal RFEM

6.4 Structural analysis and results

After the definition of geometry, releases and loads, the structural calculation can be done. Different types of calculation results may be obtained from the software for further design. The maximum values of bending moments, shear and axial forces will be later used for resistance check of joints. Figure 40 below illustrates the calculation window in the software

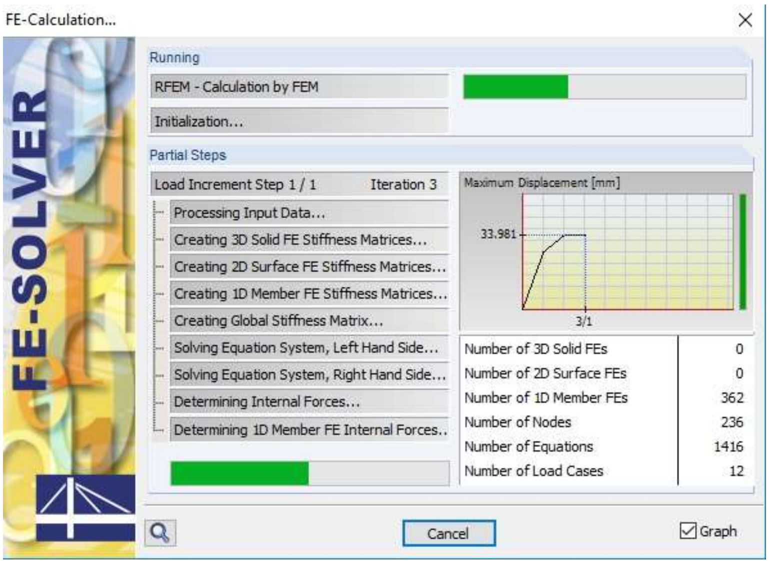


Figure 40. Calculation window in Dlubal RFEM

After the calculation is finished, it is possible to see the maximum loads in members depending on the load case. Maximum axial forces in the members of the roof truss for the worst load combinations are shown in Figure 41.

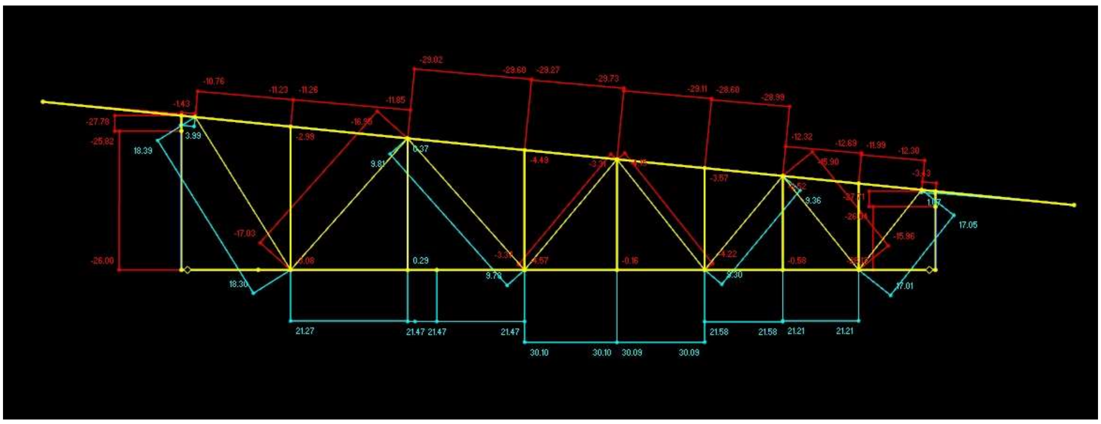


Figure 41. Maximum axial forces in roof truss members

The results from RFEM are presented in the form of report in Appendix 2.

6.5 Design resistance of structural members

The design of steel members in Dlubal RFEM is done according to SFS EN 1993-1-1

The member resistance design includes:

- Resistance of the cross-section
- Buckling of members under compression or lateral torsional buckling under bending
- Combined bending and axial compression
- Definition of the design ratio of the cross-section

The software analyses the most critical actions in members according to Ultimate Limit State and conducts relevant calculations. A total of 32 load combinations are used for the calculation. Figure 42 illustrates the design utility ratios of members in the structure.

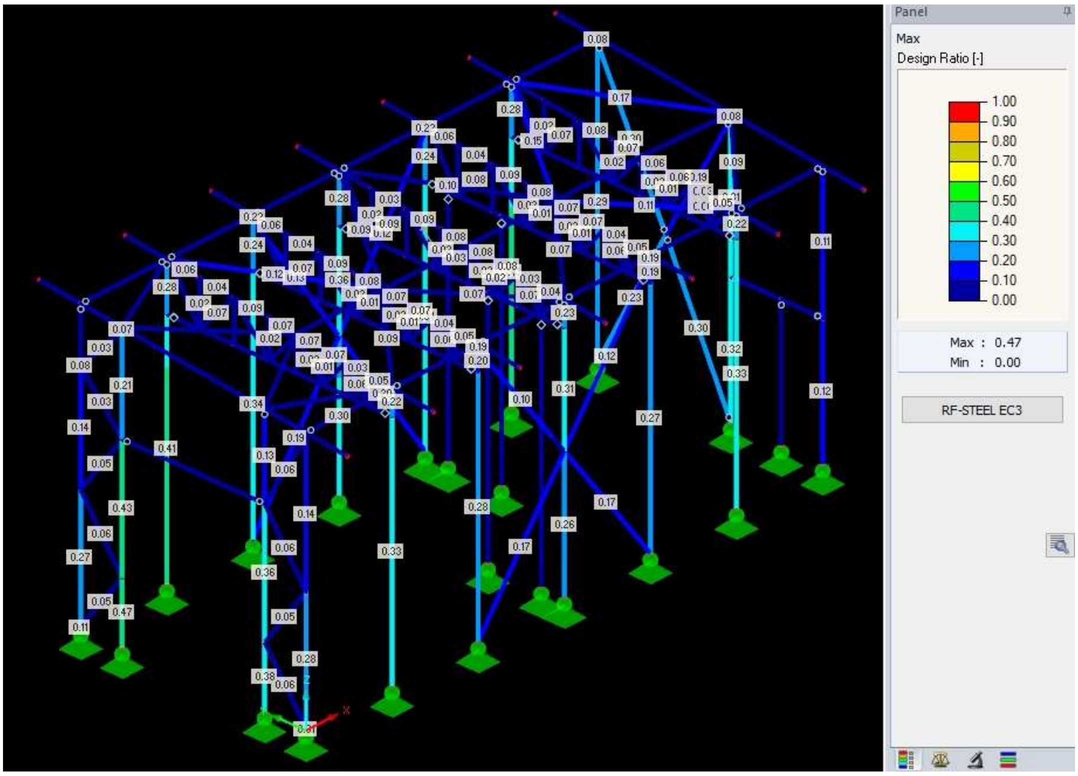


Figure 42. Utility ratios of structural members

The results of member design are shown in Table 13.

Table 13. Results of the member design

Section No.	Member No.	Location x [m]	Load-ing	Design Ratio		Design According to Formula
2	QRO 100x5 | EN 10219-2:2006					
	239	0,250	CO12	0,00	≤ 1	CS100) Negligible internal forces
	156	0,600	CO25	0,06	≤ 1	CS101) Cross-section check - Tension acc. to 6.2.3
	2	0,000	CO4	0,07	≤ 1	CS102) Cross-section check - Compression acc. to 6.2.4
	76	0,904	CO29	0,15	≤ 1	CS111) Cross-section check - Bending about y-axis acc. to 6.2.5 - Class 1 or 2
	28	1,125	CO27	0,03	≤ 1	CS116) Cross-section check - Bending about z-axis acc. to 6.2.5 - Class 1 or 2
	136	0,000	CO27	0,12	≤ 1	CS121) Cross-section check - Shear force in z-axis acc. to 6.2.6
	288	0,100	CO1	0,03	≤ 1	CS123) Cross-section check - Shear force in y-axis acc. to 6.2.6
	1	0,000	CO1	0,00	≤ 1	CS126) Cross-section check - Shear buckling acc. to 6.2.6(6)
	76	0,904	CO29	0,15	≤ 1	CS141) Cross-section check - Bending and shear force acc. to 6.2.5 and 6.2.8
	28	1,125	CO27	0,03	≤ 1	CS151) Cross-section check - Bending about z-axis and shear force acc. to 6.2.5 and 6.2.8
	286	2,650	CO18	0,03	≤ 1	CS161) Cross-section check - Biaxial bending and shear force acc. to 6.2.6, 6.2.7 and 6.2.9
	17	1,260	CO4	0,22	≤ 1	CS181) Cross-section check - Bending, shear and axial force acc. to 6.2.9.1
	74	3,063	CO5	0,21	≤ 1	CS201) Cross-section check - Bending about z-axis, shear and axial force acc. to 6.2.9.1
	17	0,630	CO4	0,08	≤ 1	CS221) Cross-section check - Biaxial bending, shear and axial force acc. to 6.2.10 and 6.2.9
	105	0,603	CO28	0,08	≤ 1	ST301) Stability analysis - Flexural buckling about y-axis acc. to 6.3.1.1 and 6.3.1.2(4)
	174	1,000	CO29	0,24	≤ 1	ST302) Stability analysis - Flexural buckling about y-axis acc. to 6.3.1.1 and 6.3.1.2
	26	0,000	CO17	0,07	≤ 1	ST311) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1.2(4)
	2	0,000	CO12	0,21	≤ 1	ST312) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1.2
	16	0,000	CO4	0,47	≤ 1	ST364) Stability analysis - Bending and compression acc. to 6.3.3, Method 2
3	QRO 60x4 | EN 10219-2:2006					
	7	0,000	CO23	0,00	≤ 1	CS100) Negligible internal forces
	168	0,000	CO27	0,07	≤ 1	CS101) Cross-section check - Tension acc. to 6.2.3
	165	0,000	CO25	0,07	≤ 1	CS102) Cross-section check - Compression acc. to 6.2.4
	277	1,767	CO21	0,02	≤ 1	CS111) Cross-section check - Bending about y-axis acc. to 6.2.5 - Class 1 or 2
	141	0,560	CO27	0,04	≤ 1	CS116) Cross-section check - Bending about z-axis acc. to 6.2.5 - Class 1 or 2
	277	3,535	CO4	0,00	≤ 1	CS121) Cross-section check - Shear force in z-axis acc. to 6.2.6
	141	0,280	CO25	0,01	≤ 1	CS123) Cross-section check - Shear force in y-axis acc. to 6.2.6
	277	0,000	CO4	0,00	≤ 1	CS126) Cross-section check - Shear buckling acc. to 6.2.6(6)
	277	1,767	CO21	0,02	≤ 1	CS141) Cross-section check - Bending and shear force acc. to 6.2.5 and 6.2.8
	141	0,560	CO27	0,04	≤ 1	CS151) Cross-section check - Bending about z-axis and shear force acc. to 6.2.5 and 6.2.8
	90	0,560	CO27	0,00	≤ 1	CS161) Cross-section check - Biaxial bending and shear force acc. to 6.2.6, 6.2.7 and 6.2.9
	277	1,767	CO4	0,03	≤ 1	CS181) Cross-section check - Bending, shear and axial force acc. to 6.2.9.1
	141	0,560	CO25	0,04	≤ 1	CS201) Cross-section check - Bending about z-axis, shear and axial force acc. to 6.2.9.1
	193	0,560	CO26	0,00	≤ 1	CS221) Cross-section check - Biaxial bending, shear and axial force acc. to 6.2.10 and 6.2.9
	165	0,000	CO25	0,09	≤ 1	ST301) Stability analysis - Flexural buckling about y-axis acc. to 6.3.1.1 and 6.3.1.2(4)
	292	0,000	CO1	0,23	≤ 1	ST302) Stability analysis - Flexural buckling about y-axis acc. to 6.3.1.1 and 6.3.1.2
	165	0,000	CO25	0,09	≤ 1	ST311) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1.2(4)
	292	0,000	CO1	0,23	≤ 1	ST312) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1.2
	293	0,000	CO4	0,30	≤ 1	ST364) Stability analysis - Bending and compression acc. to 6.3.3, Method 2

The maximum utility ratio in the Hunter’s Hut structure is 0.47. There is a possibility to increase the utility ratio by decreasing the sizes of the cross-sections. It will minimize the costs and material used for production.

As an example, the SHS100x100x5 profiles could be replaced by SHS80x80x6 and SHS60x60x4 – with 50x50x3. As can be seen in Table 14, the utility ratio is now equal to 0.82.

The optimization of members may be also achieved by changing the shape or materials of the cross-sections.

Table 14. Results of the design for members with smaller cross-section

Section No.	Member No.	Location x [m]	Load-ing	Design Ratio		Design According to Formula
2	QRO 80x6 | EN 10219-2:2006					
	239	0,500	CO5	0,00	≤1	CS100) Negligible internal forces
	156	0,600	CO25	0,06	≤1	CS101) Cross-section check - Tension acc. to 6.2.3
	2	0,000	CO4	0,07	≤1	CS102) Cross-section check - Compression acc. to 6.2.4
	76	0,904	CO29	0,20	≤1	CS111) Cross-section check - Bending about y-axis acc. to 6.2.5 - Class 1 or 2
	28	1,125	CO27	0,04	≤1	CS116) Cross-section check - Bending about z-axis acc. to 6.2.5 - Class 1 or 2
	136	0,000	CO27	0,13	≤1	CS121) Cross-section check - Shear force in z-axis acc. to 6.2.6
	288	0,100	CO4	0,04	≤1	CS123) Cross-section check - Shear force in y-axis acc. to 6.2.6
	1	0,000	CO1	0,00	≤1	CS126) Cross-section check - Shear buckling acc. to 6.2.6(6)
	76	0,904	CO29	0,20	≤1	CS141) Cross-section check - Bending and shear force acc. to 6.2.5 and 6.2.8
	28	1,125	CO27	0,04	≤1	CS151) Cross-section check - Bending about z-axis and shear force acc. to 6.2.5 and 6.2.8
	286	2,650	CO9	0,06	≤1	CS161) Cross-section check - Biaxial bending and shear force acc. to 6.2.6, 6.2.7 and 6.2.9
	17	1,260	CO4	0,34	≤1	CS181) Cross-section check - Bending, shear and axial force acc. to 6.2.9.1
	74	3,063	CO5	0,32	≤1	CS201) Cross-section check - Bending about z-axis, shear and axial force acc. to 6.2.9.1
	17	0,630	CO4	0,16	≤1	CS221) Cross-section check - Biaxial bending, shear and axial force acc. to 6.2.10 and 6.2.9
	257	0,603	CO18	0,06	≤1	ST301) Stability analysis - Flexural buckling about y-axis acc. to 6.3.1.1 and 6.3.1.2(4)
	174	1,000	CO29	0,39	≤1	ST302) Stability analysis - Flexural buckling about y-axis acc. to 6.3.1.1 and 6.3.1.2
	158	0,000	CO25	0,05	≤1	ST311) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1.2(4)
	271	0,000	CO32	0,35	≤1	ST312) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1.2
	16	0,000	CO4	0,82	≤1	ST364) Stability analysis - Bending and compression acc. to 6.3.3, Method 2
3	QRO 50x3 | EN 10219-2:2006					
	24	1,305	CO21	0,00	≤1	CS100) Negligible internal forces
	168	0,000	CO27	0,11	≤1	CS101) Cross-section check - Tension acc. to 6.2.3
	165	0,000	CO25	0,10	≤1	CS102) Cross-section check - Compression acc. to 6.2.4
	277	1,767	CO21	0,02	≤1	CS111) Cross-section check - Bending about y-axis acc. to 6.2.5 - Class 1 or 2
	90	0,560	CO31	0,06	≤1	CS116) Cross-section check - Bending about z-axis acc. to 6.2.5 - Class 1 or 2
	141	0,280	CO25	0,01	≤1	CS123) Cross-section check - Shear force in y-axis acc. to 6.2.6
	283	0,000	CO3	0,00	≤1	CS126) Cross-section check - Shear buckling acc. to 6.2.6(6)
	283	0,000	CO4	0,06	≤1	CS131) Cross-section check - Torsion acc. to 6.2.7
	277	1,767	CO21	0,02	≤1	CS141) Cross-section check - Bending and shear force acc. to 6.2.5 and 6.2.8
	90	0,560	CO31	0,06	≤1	CS151) Cross-section check - Bending about z-axis and shear force acc. to 6.2.5 and 6.2.8
	90	0,560	CO27	0,01	≤1	CS161) Cross-section check - Biaxial bending and shear force acc. to 6.2.6, 6.2.7 and 6.2.9
	277	1,767	CO4	0,03	≤1	CS181) Cross-section check - Bending, shear and axial force acc. to 6.2.9.1
	141	0,560	CO27	0,07	≤1	CS201) Cross-section check - Bending about z-axis, shear and axial force acc. to 6.2.9.1
	193	0,560	CO25	0,01	≤1	CS221) Cross-section check - Biaxial bending, shear and axial force acc. to 6.2.10 and 6.2.9
	143	0,784	CO27	0,12	≤1	ST301) Stability analysis - Flexural buckling about y-axis acc. to 6.3.1.1 and 6.3.1.2(4)
	292	0,000	CO1	0,48	≤1	ST302) Stability analysis - Flexural buckling about y-axis acc. to 6.3.1.1 and 6.3.1.2
	143	0,784	CO27	0,12	≤1	ST311) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1.2(4)
	292	0,000	CO1	0,48	≤1	ST312) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1.2
	293	0,000	CO4	0,61	≤1	ST364) Stability analysis - Bending and compression acc. to 6.3.3, Method 2

7 RESISTANCE CHECK OF JOINTS

Resistance check of structural joints is the final part of structural design. Three connections will be reviewed in this thesis: KT-joint connection of the roof truss, column-to-truss connection and column-to-foundation connection. Calculations are done according to Eurocode 3: Design of Steel Structures – Part 1-8: Design of Joints. Internal forces of structural members are taken from Dlubal RFEM calculation model. Mathcad software is used for calculations.

7.1 KT-joint welded truss connection

7.1.1 Description of the connection

The connection is represented by four tubular hollow sections joined in one point by welding. Member eccentricity is avoided and all angles between the members are kept not less than 30° to ensure that welding and assembly is done without obstacles.
There is a total of 20 KT-joints in the structure although only the one with the biggest internal force values is calculated to ensure the resistance. Detail drawing of the connection is shown in Figure 43.

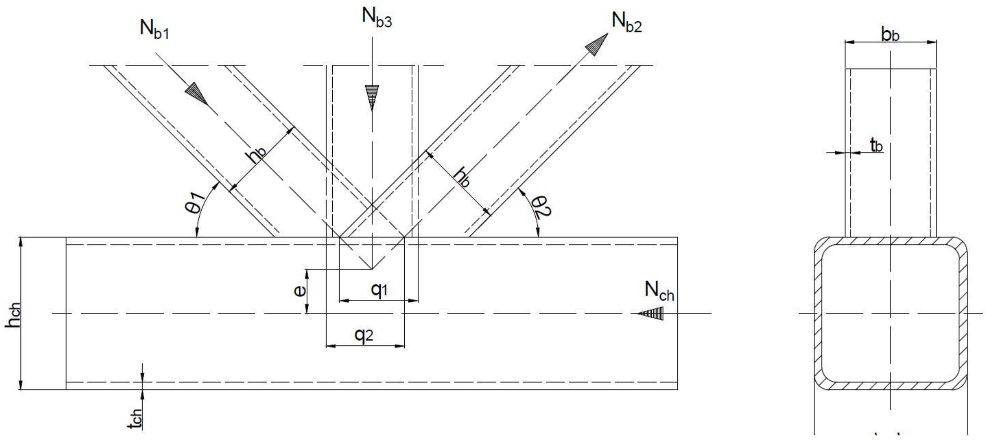


Figure 43. KT-joint detail drawing

Location of the connection in the structure is shown in Figure 44.

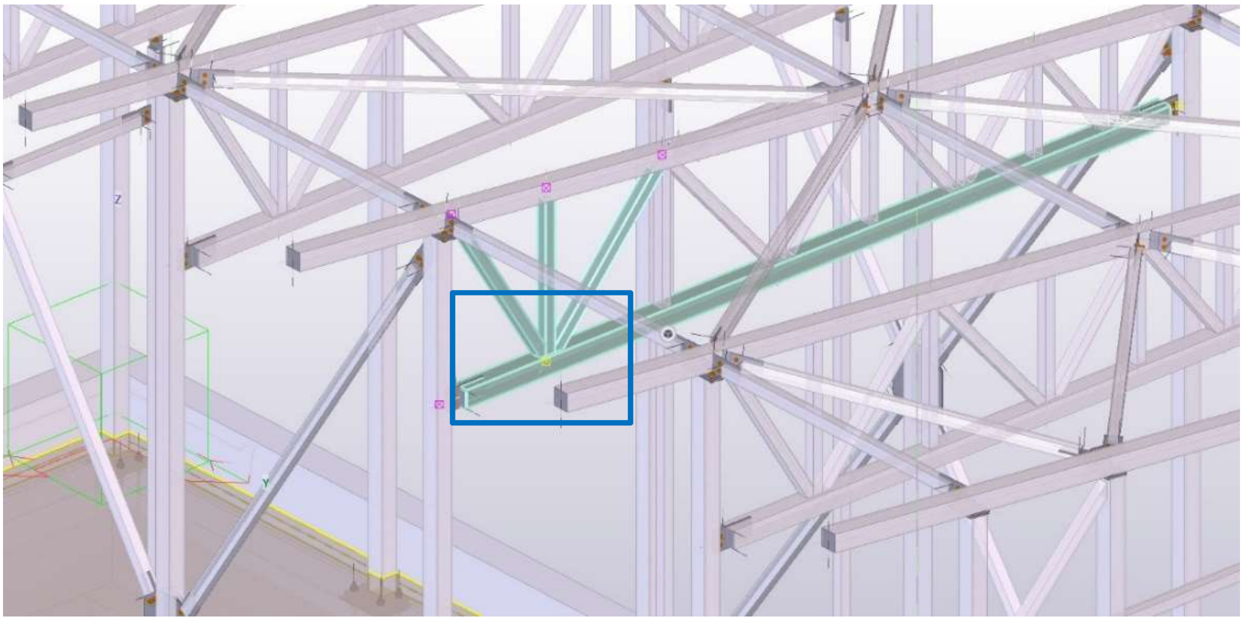


Figure 44. Location of the connection in the structure

7.1.2 Calculation procedure

The calculation was done according to the following steps:

- Define the detailed characteristics of the connection
- Determine the values of member forces applied to the joint
- Check the validity range of the brace and chord sizes
- Calculate the relative value for the overlap
- Verify resistance of a brace member failure. The following must be satisfied:

$$N_{i,Rd} = f_{y,b}\ t_b \frac{\left(b_{eff} + b_{e,ov} - 4t_b + \frac{\lambda_{ov,i}\ 100\ \ h_b\ 2}{50}\right)}{\gamma_{m5}} \geq N_{biEd}$$

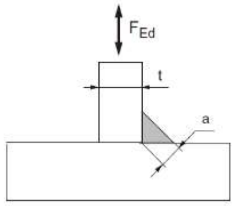
Where:

$f_{y,b}$ is the nominal yield strength of a brace member (N/mm^2)
t_b is the thickness of a brace member (mm)
b_{eff} is the effective width for a brace member to chord connection (mm)
$b_{e,ov}$ is the effective width for an overlapping joint (mm)
$\lambda_{ov,i}$ is the overlap ratio of the overlap joint
h_b is the depth of a brace member (mm)
γ_{m5} is the safety factor
$N_{i,Rd}$ is the design resistance of a brace member (N)
$N_{bi,Ed}$ is the design normal force in a brace member (N)

- Do the design calculations for the weld

Table 15 below is used to calculate the throat thickness of the weld.

Table 15. The required throat thickness for an equal strength fillet weld made around the perimeter of the hollow section which is subject to axial tension, compression and/or bending (SFS EN 1993-1-8: 2005).



Steel grade	Yield strength a) f_y (N/mm²)	Ultimate tensile strength a) f_u (N/mm²)	Throat thickness of the weld b)
S235H	235	360	0,92·t
S275H	275	430	0,96·t
S355H	**355**	**510**	**1,11·t**
S275NH	275	370	1,12·t
S355NH	355	470	1,20·t
S460NH	460	550	1,48·t
S275MH	275	360	1,15·t
S355MH	355	470	1,20·t
S420MH	**420**	**500**	**1,48·t**
S460MH	460	530	1,53·t

a) Nominal strength values according to Table 1.7.
b) However at least a ≥ 3 mm.

- The values in this table have been calculated using the recommended values $\gamma_{M0} = 1{,}0$ and $\gamma_{M2} = 1{,}25$ as given in Parts EN 1993-1-1 and EN 1993-1-8 of Eurocode. **National requirements must be checked from the National Annex of the relevant country.**
- The throat thickness values of this table are valid:
  - when the hollow section to be joined is subject to axial tension or compression and /or bending
  - when the weld is made around the perimeter of the hollow section to be joined
  - when the hollow section to be joined is of the same grade or lower grade than the adjacent member
  - when using a welding consumable having its yield strength and ultimate tensile strength at least equivalent minimum values as those of the hollow section to be joined.

See detailed calculation of the joint in Appendix 3.

7.2 **Column-to-truss connection**

7.2.1 Description of the connection

The structural column is connected to the upper chord of the truss. The weight of the whole truss is divided between two columns. The connection between the chord and the column is represented by a bolt connection with two plates. One plate is welded to the column and another one to the upper chord. Detail drawing of the connection is shown in Figure 45.

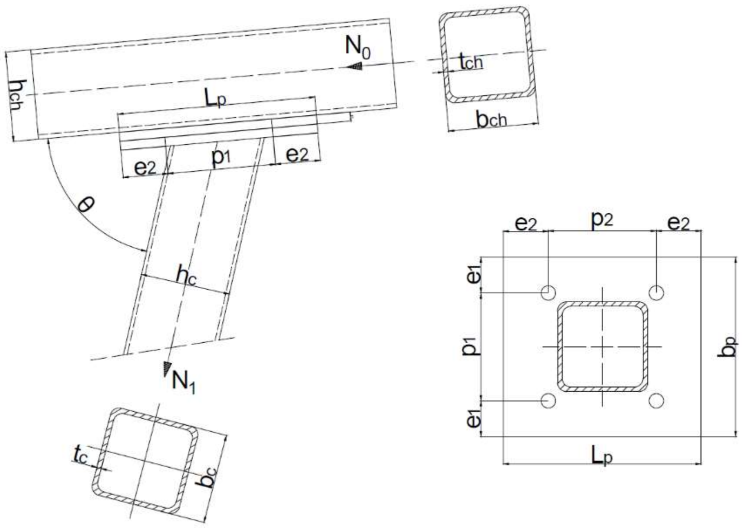


Figure 45. Column-to-truss detail drawing

The location of the connection in the structure is shown in Figure 46.

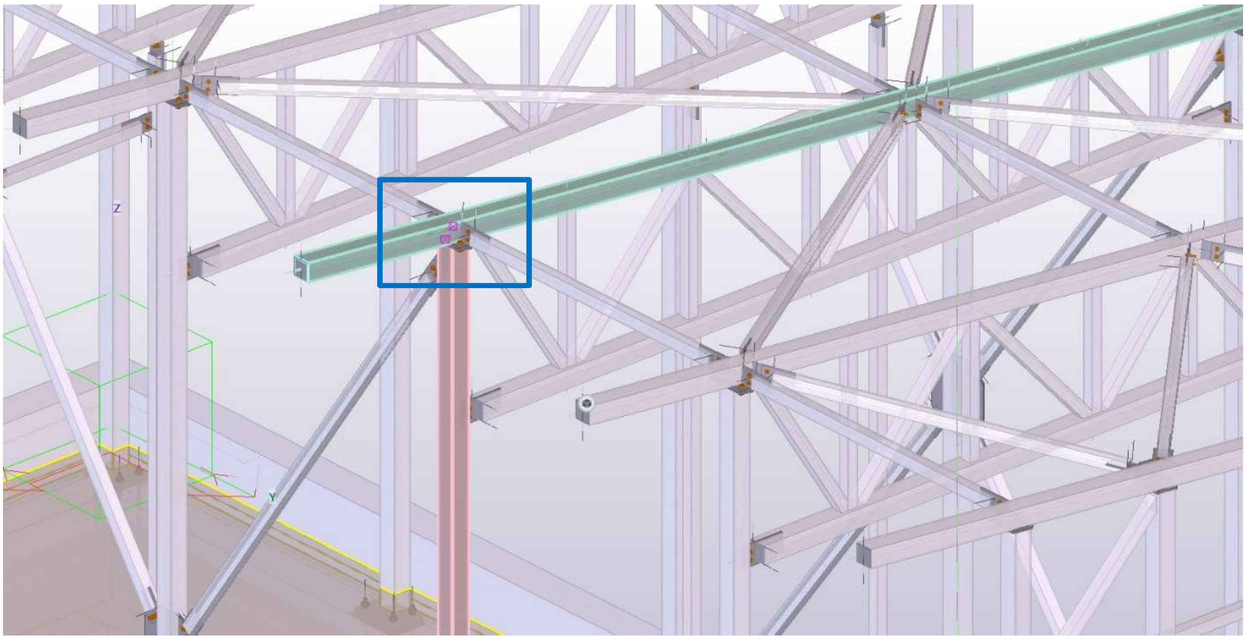


Figure 46. Location of the connection in the structure

7.2.2 Calculation procedure

The calculation was done according to the following steps:

- Define the detailed characteristics of the connection
- Determine the values of member forces applied to the joint
- Check the validity range of the plate's distances
- Verify the chord face failure resistance. The following must be satisfied:

$$N_{1,Rd} = \frac{f_{y,p}}{(1-\beta_p)\sin(\theta)}\, t_p^2\, \frac{\left(2\frac{\eta_p}{\sin(\theta)} + 4\sqrt{1-\beta_p}\right)}{\gamma_{M5}} \geq N_{Ed,f}$$

Where:

$f_{y,p}$ is the nominal yield strength of the plate (N/mm^2)
β_p is the ratio of the brace member width to the width of the plate
t_p is the thickness of the plate (mm)
η_p is the ratio of the brace member depth to the width of the plate
γ_{M5} is the safety factor
θ is the angle between the brace member and the chord
$N_{1,Rd}$ is the design value of the chord's resistance to face failure (N)
$N_{Ed,f}$ is the design concentrated load applied to the joint (N)

- Verify the chord side wall buckling resistance. The following must be satisfied:

$$N_{2,Rd} = \frac{f_{buckling}}{\sin(\theta)}\ t_{ch}\ \frac{\left(2\ \frac{h_{ch}}{\sin(\theta)} + 10\ t_{ch}\right)}{\gamma_{M5}} \geq N_{Ed,f}$$

Where:

$f_{buckling}$ is the buckling strength of the chord web (N/mm^2)
t_{ch} is the thickness of a brace member (mm)
h_{ch} is the effective width for a brace member to chord connection (mm)
t_{ch} is the effective width for an overlapping joint (mm)
γ_{M5} is the safety factor
θ is the angle between the brace member and the chord
$N_{2,Rd}$ is the design value of the chord's resistance to side wall buckling (N)
$N_{Ed,f}$ is the design concerntrated load applied to the joint (N)

- Verify the chord resistance to face punching shear. The following must be satisfied:

$$N_{3,Rd} = \frac{f_{y,p}}{\sqrt{3}\sin(\theta)}\ t_p\ \frac{\left(2\ b_{e,p} + 2\ \frac{h_{ch}}{\sin(\theta)}\right)}{\gamma_{M5}} \geq N_{Ed,f}$$

Where:

$f_{y,p}$ is the nominal yield strength of the plate (N/mm^2)
t_p is the thickness of the plate (mm)
$b_{e,p}$ is the effective width when calculating punching shear of the chord face (mm)
h_{ch} is depth of the chord (mm)
γ_{M5} is the safety factor
θ is the angle between the brace member and the chord
$N_{3,Rd}$ is the design value of the chord's resistance to face punching shear(N)
$N_{Ed,f}$ is the design concerntrated load applied to the joint (N)

- Verify the resistance of the column. The following must be satisfied:

$$N_{4,Rd} = f_{y,c}\ t_c\ \frac{\left(2\ h_c - 4\ t_c + 2\ b_{eff}\right)}{\gamma_{M5}} \geq N_{Ed,f}$$

Where:

$f_{y,c}$ is the nominal yield strength of the column (N/mm^2)
t_c is the thickness of the column profile (mm)
h_c is the depth of the column profile (mm)

b_{eff} is the the effective width for a brace member to chord connection (mm)
γ_{M5} is the safety factor
$N_{4,Rd}$ is the design value of the column's normal force resistance(N)
$N_{Ed,f}$ is the design concerntrated load applied to the joint (N)

- Verify tension resistance of bolts using the following equation:

$$F_{t,Rd} = 0.9\, f_{u,bolt} \frac{A_s}{\gamma_{M2}} = 90.432kN$$

Where:

$f_{u,bolt}$ is the ultimate tensile strength of a bolt (N/mm^2)
A_s is the tensile stress area of a bolt (mm^2)
γ_{M2} is the safety factor

The following must be satisfied:

$$n_{bolt}F_{t,Rd} \geq N_{Ed,f}$$

Where:

$F_{t,Rd}$ is the tensile resistance of a bolt (N)
$N_{Ed,f}$ is the design concentrated load applied to the joint (N)
n_{bolt} is the number of bolts in the connection

- Verify punching shear resistance of bolts using the following equation:

$$N_{p,Rd} = 0.6\, \pi\, d_m\, t_p \frac{f_{u,p}}{\gamma_{M2}}$$

Where:

d_m is the diameter of the hole (mm)
t_p is the thickness of the plate (mm)
$f_{u,p}$ is the ultimate tensile strength of the plate (N/mm^2)
γ_{M2} is the safety factor

The following must be satisfied:

$$n_{bolt}F_{t,Rd} \geq N_{Ed,f}$$

Where:

$F_{t,Rd}$ is the tensile resistance of a bolt (N)
$N_{Ed,f}$ is the design concentrated load applied to the joint (N)

n_{bolt} is the number of bolts in the connection

- Verify shear resistance of bolts using the following equations:

$$F_{v,Rd} = 0.6\, f_{u,bolt}\, \frac{A_{bolt}}{\gamma_{M2}}$$

And

$$V_d = \sqrt{V_{y,Ed}^2 + V_{z,Ed}^2}$$

Where:

$f_{u,bolt}$ is the ultimate tensile strength of a bolt (N/mm^2)
A_{bolt} is the area of a bolt (mm^2)
γ_{M2} is the safety factor
$V_{y,Ed}$ is the design shear force in y − direction (N)
$V_{z,Ed}$ is the design shear force in z − direction (N)

The following must be satisfied:

$$n_{bolt} F_{v,Rd} \geq V_d$$

Where:

$F_{v,Rd}$ is the shear resistance of a bolt (N)
V_d is the resultant shear force applied to the joint (N)
n_{bolt} is the number of bolts in the connection

- Verify the resistance of plates. The following must be satisfied:

$$N_{t,Rd} = t_p^2\, (1 + \delta\, a)\, \frac{n_{bolt}}{K\, \gamma_{M2}} \geq N_{Ed,f}$$

Where:

t_p is the thickness of the plate (mm)
δ is the factor that represents the relative net area of the bolt row
a is the throat thickness of the weld (mm)
γ_{M2} is the safety factor
n_{bolt} is the number of bolts in the connection
K is the auxiliary variable that is related to the plastic moment of the plate

- Do the weld design calculations

Chord-to-plate and column-to-plate welds have to satisfy the following:

$$\sigma_f \le \frac{f_{u,i}}{\gamma_{M2}\,\beta_w}$$

Where:

σ_f is the axial stress in the joint (N/mm^2)
$f_{u,i}$ is the ultimate strength of the column (N/mm^2)
γ_{M2} is the safety factor
β_w is the strength factor for the weld

Table 16 below is used to calculate the throat thickness of the weld.

Table 16. Welds between different steel grades. The required throat thickness for an equal strength fillet weld made around the perimeter of the hollow section which is subject to axial tension, compression and/or bending (SFS EN 1993-1-8: 2005).

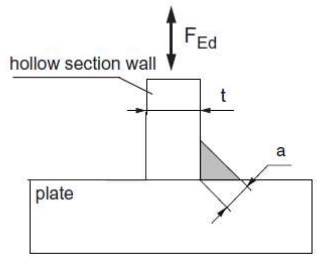


Plate		Throat thickness of the weld [b)]	
Steel grade	Ultimate tensile strength [a)] f_u (N/mm^2)	Structural hollow section S355H	Structural hollow section S420MH
S235	360	1,39·t	1,65·t
S275	430	1,24·t	1,48·t
S355	**490**	**1,15·t**	**1,36·t**
S275N	390	1,37·t	1,62·t
S355N	490	1,15·t	1,36·t
S420N	520	1,11·t	1,48·t
S460N	540	1,11·t	1,48·t
S275M	370	1,44·t	1,71·t
S355M	470	1,20·t	1,42·t
S420M	520	1,11·t	1,48·t
S460M	540	1,11·t	1,48·t

a) Nominal values of the ultimate tensile strength presented in EN 1993-1-1 for flat steels in grades S235-S460 conforming to EN 10025, when t ≤ 40 mm.
b) However at least a ≥ 3 mm.

- The values in this table have been calculated using the recommended values $\gamma_{M0} = 1,0$ and $\gamma_{M2} = 1,25$ as given in Parts EN 1993-1-1 and EN 1993-1-8 of Eurocode. **National requirements must be checked from the National Annex of the relevant country.**
- The throat thickness values of this table are valid:
  - when the hollow section to be joined is subject to axial tension or compression and /or bending
  - when the weld is made around the perimeter of the hollow section to be joined
  - when using a welding consumable having its yield strength and ultimate tensile strength at least equivalent minimum values as those of the weaker part to be joined.

See detailed calculation of the joint in Appendix 3.

7.3 **Column-to-foundation connection**

7.3.1 Description of the connection

Structural columns are connected to foundation by endplates with anchor rods. There are two types of column-to-foundation connections in the structure: corner connection and middle connection. This report reviews the middle connection. Detail drawing of the connection is demonstrated in Figure 47.

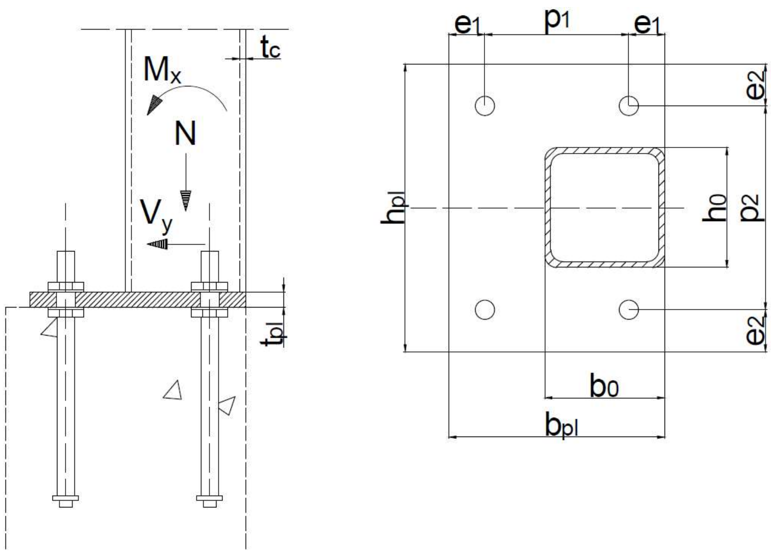


Figure 47. Beam-to-column detail drawing

The location of the connection in the structure is shown in Figure 48.

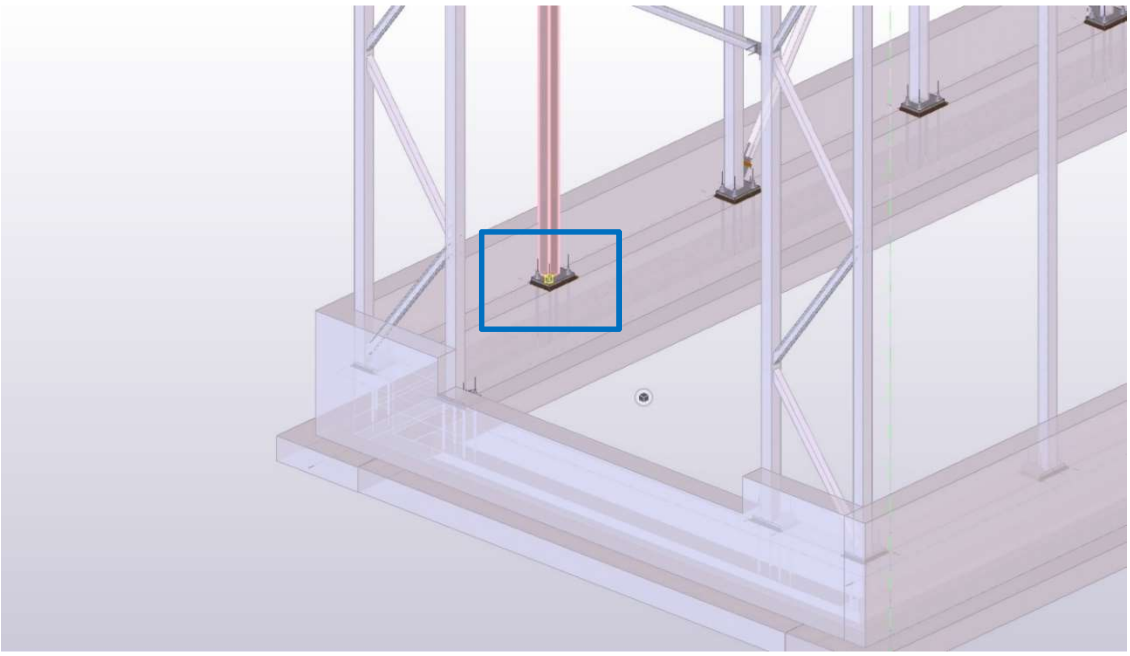


Figure 48. Location of the connection in the structure

7.3.2 Calculation procedure

The calculation was done according to the following steps:

- Define the detailed characteristics of the connection
- Determine the values of internal member forces applied to the joint
- Verify the combined stress ratio that has to satisfy the following:

$$Ratio = \frac{V_{Ed}^{\frac{4}{3}}}{V_{u,bolt}^{\frac{4}{3}}} + \frac{F_{Ed}^{\frac{4}{3}}}{F_{u,bolt}^{\frac{4}{3}}} \leq 1$$

Where:

V_{Ed} is the design shear force (N)
$V_{u,bolt}$ is the ultimate shear strength of a bolt (N)
F_{Ed} is the design concentrated load applied to the hollow section (N)
$F_{u,bolt}$ is the ultimate tensile strength of a bolt (N)

- Verify the resistance of the plate. The following must be satisfied:

$$f_d = \frac{M_{1,Ed}}{W_{el,p}} \leq f_{y,p}$$

Where:

$M_{1,Ed}$ is the design bending moment in the plate (Nmm)
$W_{el,p}$ is the elastic section modulus of the plate (mm^3)
$f_{y,p}$ is the nominal yield strength of the plate (N/mm^2)
f_d is the elastic bending stress in the plate (N/mm^2)

- Do the weld design calculations in x and y-directions. The following must be satisfied:

$$\sigma_{f,x/y} \leq \frac{f_u}{\gamma_{M2}\,\beta_w}$$

Where:

σ_f is the axial stress in the joint (N/mm^2)
f_u is the ultimate strength of a member (N/mm^2)
γ_{M2} is the safety factor
β_w is the strength factor for the weld

Table 17 below is used to calculate the throat thickness of the weld.

Table 17. Welds between different steel grades. The required throat thickness for an equal strength fillet weld made around the perimeter of the hollow section which is subject to axial tension, compression and/or bending (SFS EN 1993-1-8: 2005)

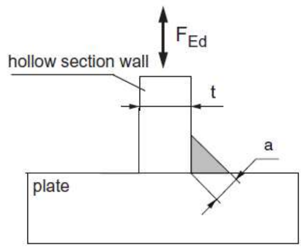


Plate		Throat thickness of the weld [b)]	
Steel grade	Ultimate tensile strength [a)] f_u (N/mm^2)	Structural hollow section S355H	Structural hollow section S420MH
S235	360	1,39·t	1,65·t
S275	430	1,24·t	1,48·t
S355	**490**	**1,15·t**	**1,36·t**
S275N	390	1,37·t	1,62·t
S355N	490	1,15·t	1,36·t
S420N	520	1,11·t	1,48·t
S460N	540	1,11·t	1,48·t
S275M	370	1,44·t	1,71·t
S355M	470	1,20·t	1,42·t
S420M	520	1,11·t	1,48·t
S460M	540	1,11·t	1,48·t

a) Nominal values of the ultimate tensile strength presented in EN 1993-1-1 for flat steels in grades S235-S460 conforming to EN 10025, when t ≤ 40 mm.
b) However at least a ≥ 3 mm.

- The values in this table have been calculated using the recommended values γ_{M0} = 1,0 and γ_{M2} = 1,25 as given in Parts EN 1993-1-1 and EN 1993-1-8 of Eurocode. **National requirements must be checked from the National Annex of the relevant country.**
- The throat thickness values of this table are valid:
  - when the hollow section to be joined is subject to axial tension or compression and /or bending
  - when the weld is made around the perimeter of the hollow section to be joined
  - when using a welding consumable having its yield strength and ultimate tensile strength at least equivalent minimum values as those of the weaker part to be joined.

See detailed calculation of the joint in Appendix 3.

8 PRODUCTION DRAWINGS

When the structural design process is finished and the dimensions of the members are defined, technical drawings for production can be made. The drawings are produced in Tekla Structures software based on the three-dimensional model of the structure. BIM software system allows to store information about every building component of the modelled structure, its properties and amounts.

8.1 Elements of production drawings

Production drawings are made for later manufacturing of building elements in the shop. Production drawings include assembly and part drawings and must contain relevant information for the components to be assembled in a correct way. The main elements of production drawings are as follows:

1. Sizes and shapes of components

2. Numbering of building components
3. Information about welding
4. Details of connection parts
5. Bill of quantities containing each part
6. Title block

8.2 **Assembly drawings**

Assembly drawings are made to present building components that consist of several parts. The drawings demonstrate how those parts are placed in the assembly, how they are connected and what their sizes are. Assembly drawings may contain orthogonal plans, sections, elevations and three-dimensional views. The location of an assembly in the structure may be shown in a general arrangement drawing. Figure 49 shows an example of the roof truss assembly drawing made for Hunter's Hut project.

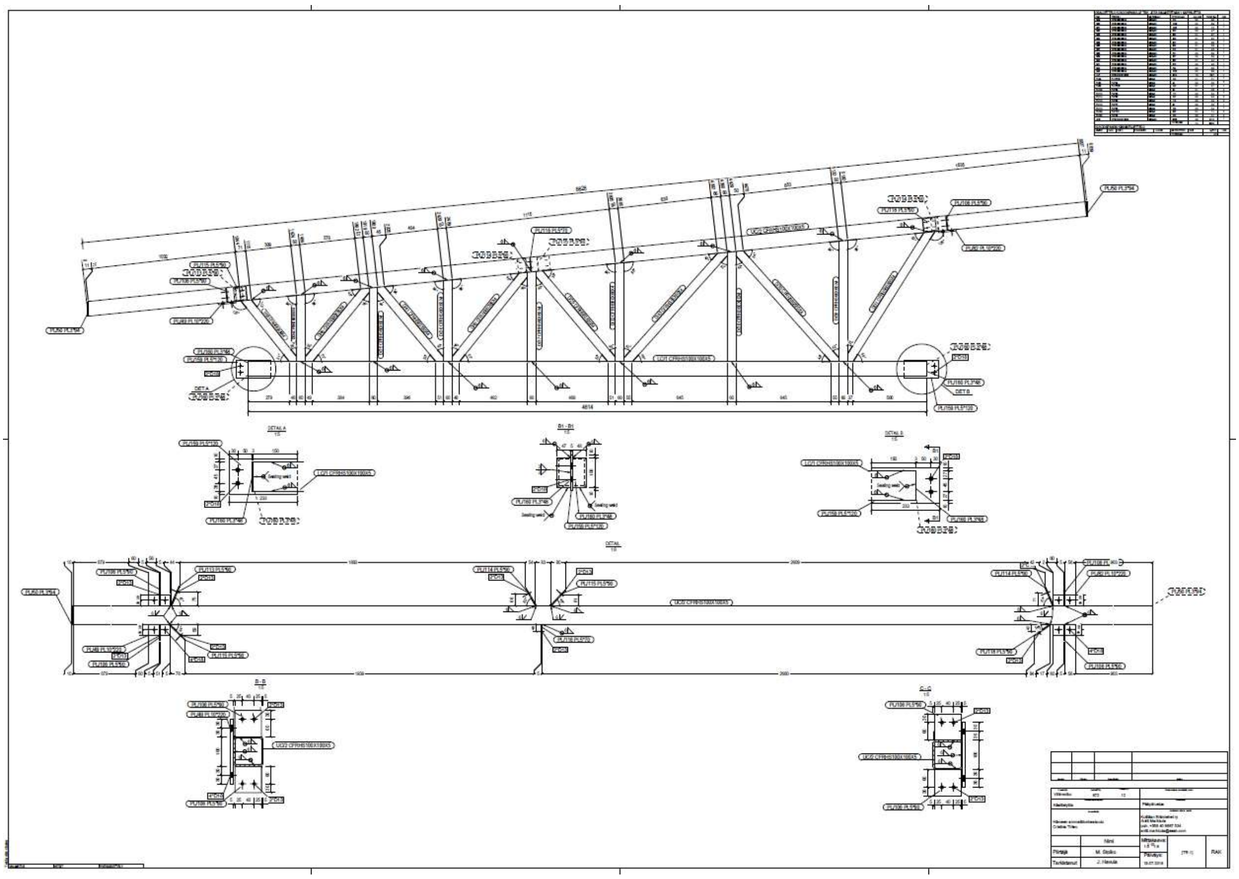


Figure 49. Roof truss assembly drawing

The drawing contains information about every single part of the assembly. Position, profile, material, weight, dimensions and amount of each part is presented in Table 18 below.

Table 18. Information on each part of the assembly

OSALUETTELO KOKOONPANOLLE TR/1, JOTA VALMISTETAAN 1 KAPPALETTA						
OSA	PROFIILI	MATERIAALI	PITUUS [mm]	ALA [m2]	PAINO [kg]	LKM
D/29	CFRHS60X60X4	S355J2H	818	0.2	5.5	1
D/30	CFRHS60X60X4	S355J2H	1026	0.2	6.9	1
D/31	CFRHS60X60X4	S355J2H	1067	0.2	7.2	1
D/32	CFRHS60X60X4	S355J2H	675	0.2	4.7	1
D/33	CFRHS60X60X4	S355J2H	560	0.1	3.7	1
D/34	CFRHS60X60X4	S355J2H	458	0.1	3.2	1
D/35	CFRHS60X60X4	S355J2H	507	0.1	3.5	1
D/36	CFRHS60X60X4	S355J2H	[illegible]	[illegible]	[illegible]	1

8.3 Single-part drawings

Drawings that present single parts of an assembly are called single part drawings. They present the shape and dimensions of the parts as well as information about which assembly they are element of. Figure 50 below illustrates an example of a single-part drawing of a truss brace made for Hunter's Hut project.

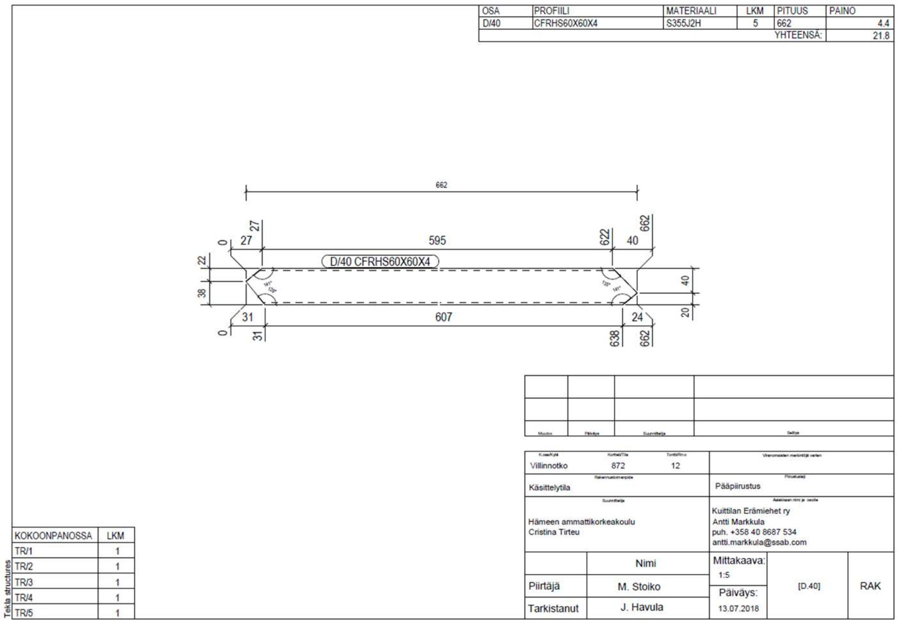


Figure 50. Truss brace single-part drawing

More examples of assembly and single-part drawings made for Hunter' Hut project are presented in Appendix 4.

9 CONCLUSION

The aim of the thesis was to describe the process of structural analysis and design using the example of a low-rise steel hall purposed for meat processing. The project was commissioned by HAMK Sheet Metal Centre and done in a group of four trainees during the summer 2018.

As the result of the project work, loads and load combinations were calculated, the structure was analyzed for stability, the joints were checked for resistance and finally, production drawings were made for further assembly. All the steps of the projects involved software such as Dlubal RFEM for structural analysis, Tekla Structures for modelling a 3D structure and later making production drawings and Mathcad for load calculations. Even though the sizes of the structural members were kept unchanged, possible modifications were discussed for the theoretical optimization of the structure.

In addition to the structural analysis, options for structural stability were studied. Bracing systems and how the load paths are working in the structure were discussed. It is useful to understand the way bracing affects the whole structure, which loads it may transfer and how to implement it in such a way that it complements the building in the most effective way.

Hopefully, this thesis will serve as an example to those who would like to get a general idea of how structural analysis and design can be performed with the help of modern software. The time and costs spent by designers can be minimized if the software is implemented wisely in the workflow. Despite the fact that nowadays computers are capable of performing work at a more precise and efficient level than humans, it is important to remember that there is still no replacement to creative human mind which is required for structural design.

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APPENDIX 1. CALCULATION OF LOADS

Dead load

Weight of the steel frame without roof

Weight of the steel frame without roof	$S_{ws} = 300000kg \times 9.8m/s^2 = 294kN$
Area of the roof	$A_{rf} = 90.3m^2$
Estimated value for steel roof	$\sigma_{rf} = 5.8kg/m^2$
Weight of the roof	$q_{rf} = A_{rf}\sigma_{rf}\ 9.8\frac{m}{s^2} = 5.1kN$
Self-weight of the whole structure	$S_w = S_{ws} + q_{rf} = \mathbf{299.1kN}$

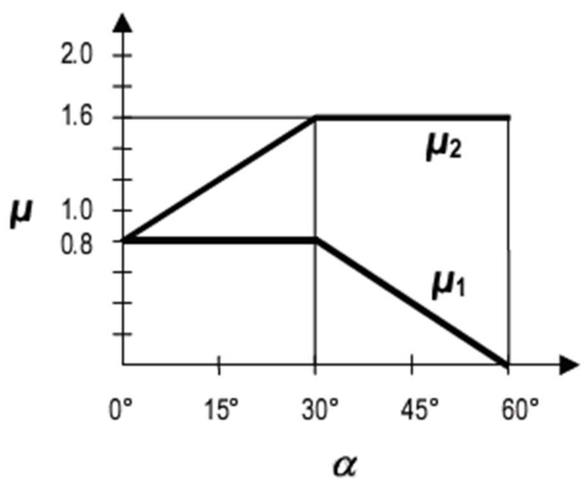
Live load

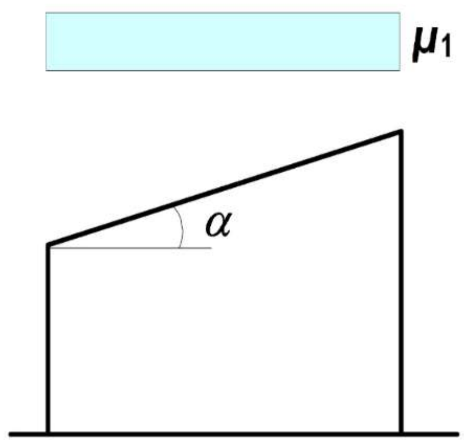
Maintenance of the roof

Characteristic value for roof maintenance	$g_{krf} = 0.4kN/m^2$
Imposed load from roof maintenance	$G_{krf} = g_{krf}\ A_{rf} = 36.1kN$
Estimated weight of three lifted carcasses	*600kg*
Imposed load from the rail	$\sigma_{rs} = 600kg \times 9.8m/s^2 = 5.9kN$

Snow load

Snow load shape coefficient	$\mu_1 = 0.8$





Exposure coefficient $C_e = 1.0$

Thermal coefficient $C_t = 1.0\ (recommended)$

Recommended values of C_e for different topographies (SFS-EN 1991-1-3: 2003)

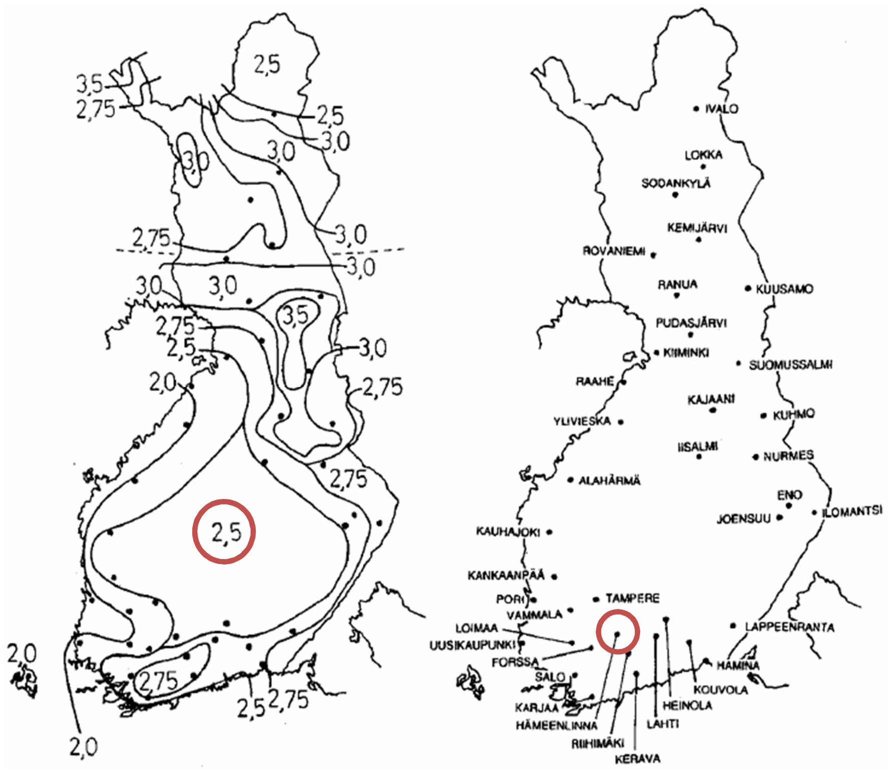
Topography	C_e
Windswept [a]	0,8
Normal [b]	1,0
Sheltered [c]	1,2

[a] *Windswept topography*: flat unobstructed areas exposed on all sides without, or little shelter afforded by terrain, higher construction works or trees.

[b] *Normal topography*: areas where there is no significant removal of snow by wind on construction work, because of terrain, other construction works or trees.

[c] *Sheltered topography*: areas in which the construction work being considered is considerably lower than the surrounding terrain or surrounded by high trees and/or surrounded by higher construction works.

Characteristic value of snow load on the ground $s_k = 2.8 kN/m^2$

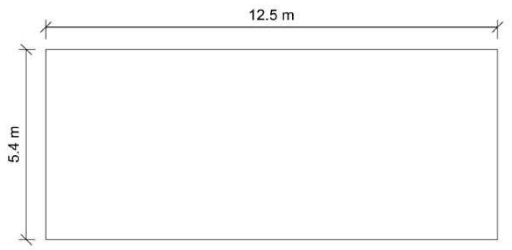


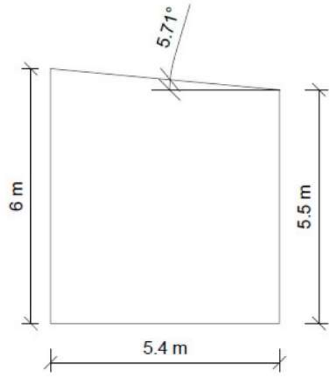
Snow loads on the ground in Finland (SFS-EN 1991-1-3: 2003)

Snow load $s = \mu_1\ C_e\ C_t\ s_k = \mathbf{2kN/m^2}$

Wind pressure

Dimensions of the building for wind calculations are illustrated below:





Characteristics of the structure

Height	$h_{ch} = 100mm$
Depth	$d = 12.5m$
Width	$b = 5.4m$
Reference height of the structure	$z = 6m$

Basic wind velocity

The basic wind velocity is calculated from the following equation:

Fundamental value of basic wind velocity	$v_{b,0} = 1$
Direction factor	$c_{dir} = 1$
Season factor	$c_{season} = 1$
Basic wind velocity	$v_b = v_{b,0}\, c_{dir}\, c_{season} = \mathbf{21m/s}$

Mean wind velocity

Roughness height	$z_o = 0.3$
Factor	$z_{o,II} = 0.05$

Terrain category		z_0 m	z_{min} m
0	Sea or coastal area exposed to the open sea	0,003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1,0	10
NOTE: The terrain categories are illustrated in A.1.			

Terrain factor $k_r = 0.19(z_o/z_{o,II})^{0.07} = 0.215$
Terrain roughness factor $c_r = k_r\, ln(z/z_o) = 0.645$
Terrain orography factor $\mathrm{c_o} = 1$
Season factor $c_{season} = 1$
Mean wind velocity $v_m = v_b\, c_r\, c_o = \mathbf{13.55}\boldsymbol{m/s}$

Wind turbulence

Factor $k_I = 1$
Wind turbulence $I_v(z) = \frac{k_I}{c_0 \ln(\frac{z}{z_0})} = \mathbf{0.334}$

Peak velocity pressure

Density of air $\rho = 1.25kg/m^3$
Peak velocity pressure $q_p = [1 + 7\, I_v]\, 0.5\, \rho\, v_m^2 = \mathbf{0.383}\ \boldsymbol{kN/m^2}$

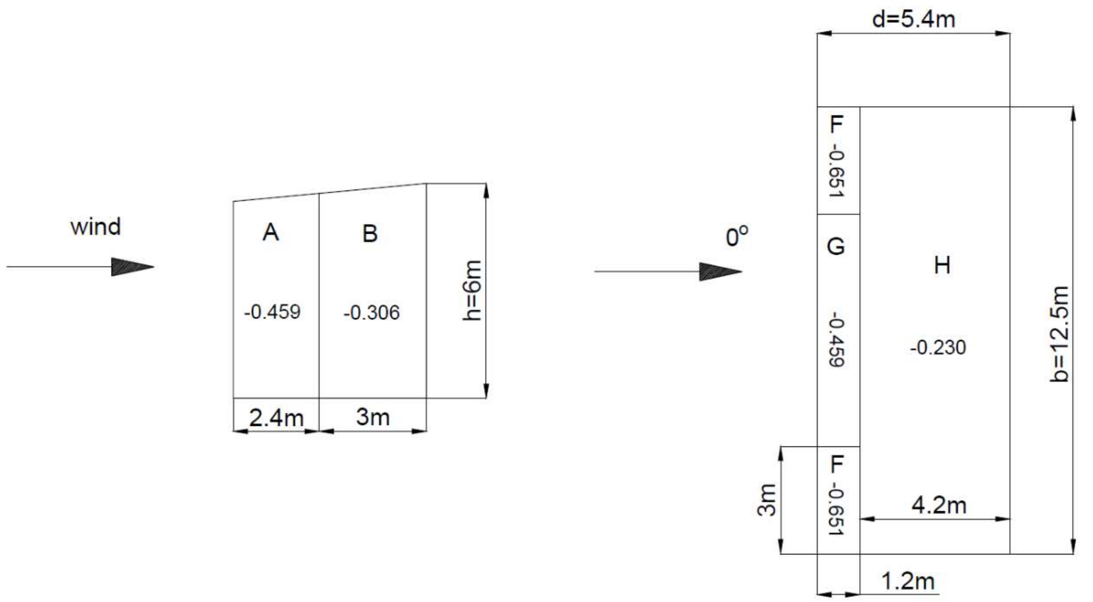
Wind pressures on surfaces

Wind 1, $\theta = 0°$

Crosswind dimension $b = 12.5m$
Height of the structure $h = 6m$
Depth of the structure $d = 5.4m$
Length parameter $e = \min(b, 2h) = 12m$
Ratio h/d $\frac{h}{d} = 1.1$

External pressure on Zone A $q_p \times (-1.2) = -0.459\ kN/m^2$
External pressure on Zone B $q_p \times (-0.8) = -0.306\ kN/m^2$
External pressure on Zone D $q_p \times 0.8 = 0.306\ kN/m^2$
External pressure on Zone E $q_p \times (-0.5) = -0.191\ kN/m^2$
External pressure on Zone F $q_p \times (-1.7) = -0.651\ kN/m^2$
External pressure on Zone G $q_p \times (-1.2) = -0.459\ kN/m^2$
External pressure on Zone H $q_p \times (-0.6) = -0.23\ kN/m^2$

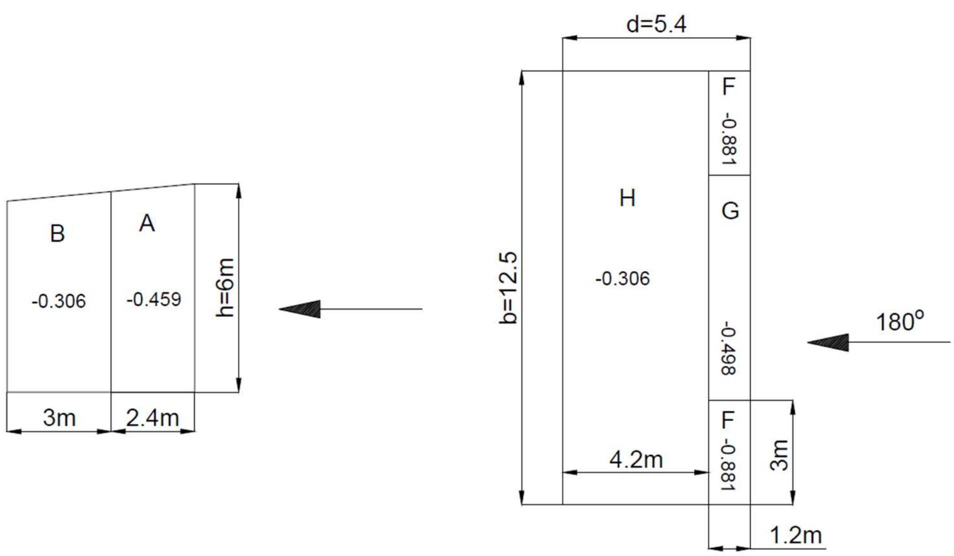
Dimensions of zones and wind pressure values are shown in the figure below:



Wind 2, $\theta = 180°$

Crosswind dimension	$b = 12.5m$
Height of the structure	$h = 6m$
Depth of the structure	$d = 5.4m$
Length parameter	$e = \min(b, 2h) = 12m$
Ratio h/d	$\frac{h}{d} = 1.1$
External pressure on Zone A	$q_p \times (-1.2) = -0.459\ kN/m^2$
External pressure on Zone B	$q_p \times (-0.8) = -0.306\ kN/m^2$
External pressure on Zone D	$q_p \times (0.8) = 0.306\ kN/m^2$
External pressure on Zone E	$q_p \times (-0.5) = -0.191\ kN/m^2$
External pressure on Zone F	$q_p \times (-2.3) = -0.881\ kN/m^2$
External pressure on Zone G	$q_p \times (-1.3) = -0.498\ kN/m^2$
External pressure on Zone H	$q_p \times (-0.8) = -0.306\ kN/m^2$

Dimensions of zones and wind pressure values are shown in the figure below:

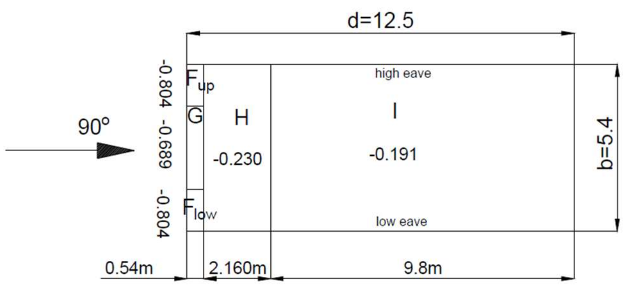


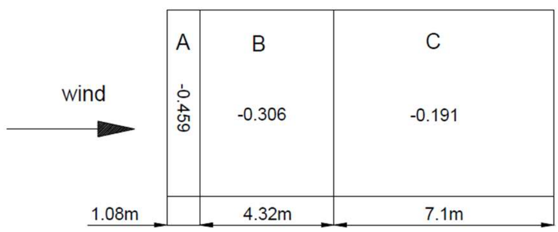
Wind 3, $\theta = 90°$

Depth of the structure	$d = 12.5m$
Height of the structure	$h = 6m$
Crosswind dimension	$b = 5.4m$
Length parameter	$e = \min(b, 2h) = 5.4m$
Ratio h/d	$\frac{h}{d} = 0.48$

External pressure on Zone A	$q_p \times (-1.2) = -0.459\ kN/m^2$
External pressure on Zone B	$q_p \times (-0.8) = -0.306\ kN/m^2$
External pressure on Zone C	$q_p \times (-0.5) = -0.191\ kN/m^2$
External pressure on Zone D	$q_p \times 0.8 = 0.306\ kN/m^2$
External pressure on Zone E	$q_p \times (-0.5) = -0.191\ kN/m^2$
External pressure on Zone F	$q_p \times (-2.1) = -0.804\ kN/m^2$
External pressure on Zone G	$q_p \times (-1.8) = -0.689\ kN/m^2$
External pressure on Zone H	$q_p \times (-0.6) = -0.23\ kN/m^2$
External pressure on Zone I	$q_p \times (-0.5) = -0.191\ kN/m^2$

Dimensions of zones and wind pressure values are shown in the figure below:





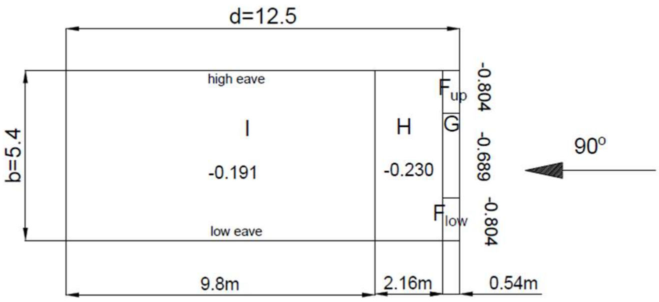
Wind 4, $\theta = 90°$

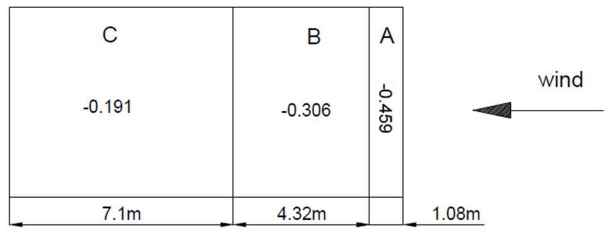
Depth of the structure	$d = 12.5m$
Height of the structure	$h = 6m$
Crosswind dimension	$b = 5.4m$
Length parameter	$e = \min(b, 2h) = 5.4m$
Ratio h/d	$\frac{h}{d} = 0.48$

External pressure on Zone A	$q_p \times (-1.2) = -0.459\ kN/m^2$
External pressure on Zone B	$q_p \times (-0.8) = -0.306\ kN/m^2$
External pressure on Zone C	$q_p \times (-0.5) = -0.191\ kN/m^2$

External pressure on Zone D $q_p \times 0.8 = 0.306\ kN/m^2$
External pressure on Zone E $q_p \times (-0.5) = -0.191\ kN/m^2$
External pressure on Zone F $q_p \times (-2.1) = -0.804\ kN/m^2$
External pressure on Zone G $q_p \times (-1.8) = -0.689\ kN/m^2$
External pressure on Zone H $q_p \times (-0.6) = -0.23\ kN/m^2$
External pressure on Zone I $q_p \times (-0.5) = -0.191\ kN/m^2$

Dimensions of zones and wind pressure values are shown in the figure below:





Keys and tables for defining the sizes of the pressure zones and external pressure coefficients are presented below:

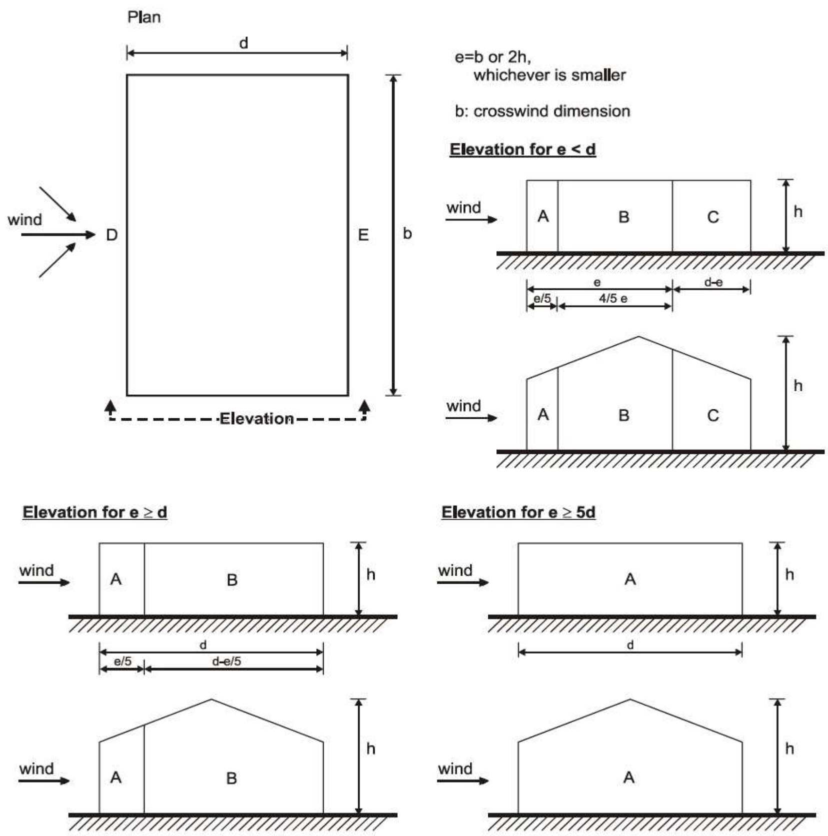
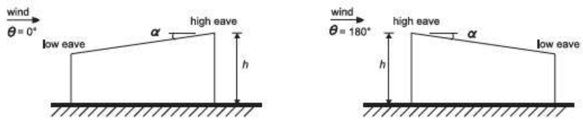
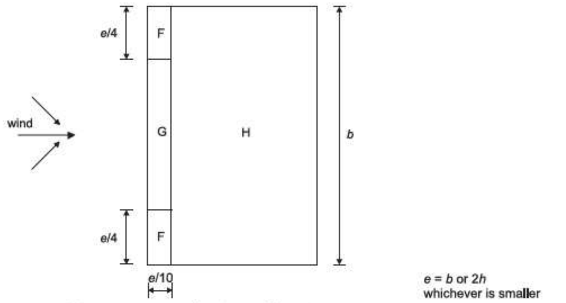


Table 7.1 — Recommended values of external pressure coefficients for vertical walls of rectangular plan buildings

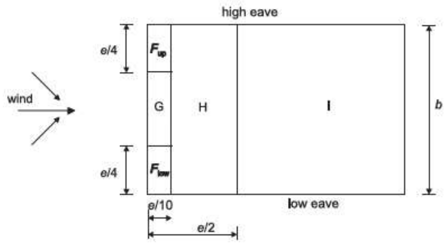
Zone	A		B		C		D		E	
h/d	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,7	
1	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,5	
≤ 0,25	-1,2	-1,4	-0,8	-1,1	-0,5		+0,7	+1,0	-0,3	



(a) general



(b) wind directions θ= 0° and θ= 180°



(c) wind direction θ= 90°

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

<table>
<tr><th rowspan="3">Pitch Angle α</th><th colspan="6">Zone for wind direction θ = 0°</th><th colspan="6">Zone for wind direction θ = 180°</th></tr>
<tr><th colspan="2">F</th><th colspan="2">G</th><th colspan="2">H</th><th colspan="2">F</th><th colspan="2">G</th><th colspan="2">H</th></tr>
<tr><th>$c_{pe,10}$</th><th>$c_{pe,1}$</th><th>$c_{pe,10}$</th><th>$c_{pe,1}$</th><th>$c_{pe,10}$</th><th>$c_{pe,1}$</th><th>$c_{pe,10}$</th><th>$c_{pe,1}$</th><th>$c_{pe,10}$</th><th>$c_{pe,1}$</th><th>$c_{pe,10}$</th><th>$c_{pe,1}$</th></tr>
<tr><td rowspan="2">5°</td><td>-1,7</td><td>-2,5</td><td>-1,2</td><td>-2,0</td><td>-0,6</td><td>-1,2</td><td rowspan="2">-2,3</td><td rowspan="2">-2,5</td><td rowspan="2">-1,3</td><td rowspan="2">-2,0</td><td rowspan="2">-0,8</td><td rowspan="2">-1,2</td></tr>
<tr><td colspan="2">+0,0</td><td colspan="2">+0,0</td><td colspan="2">+0,0</td></tr>
<tr><td rowspan="2">15°</td><td>-0,9</td><td>-2,0</td><td>-0,8</td><td>-1,5</td><td colspan="2">-0,3</td><td rowspan="2">-2,5</td><td rowspan="2">-2,8</td><td rowspan="2">-1,3</td><td rowspan="2">-2,0</td><td rowspan="2">-0,9</td><td rowspan="2">-1,2</td></tr>
<tr><td colspan="2">+0,2</td><td colspan="2">+0,2</td><td colspan="2">+ 0,2</td></tr>
<tr><td rowspan="2">30°</td><td>-0,5</td><td>-1,5</td><td>-0,5</td><td>-1,5</td><td colspan="2">-0,2</td><td rowspan="2">-1,1</td><td rowspan="2">-2,3</td><td rowspan="2">-0,8</td><td rowspan="2">-1,5</td><td rowspan="2" colspan="2">-0,8</td></tr>
<tr><td colspan="2">+0,7</td><td colspan="2">+0,7</td><td colspan="2">+0,4</td></tr>
<tr><td rowspan="2">45°</td><td colspan="2">-0,0</td><td colspan="2">-0,0</td><td colspan="2">-0,0</td><td rowspan="2">-0,6</td><td rowspan="2">-1,3</td><td rowspan="2" colspan="2">-0,5</td><td rowspan="2" colspan="2">-0,7</td></tr>
<tr><td colspan="2">+0,7</td><td colspan="2">+0,7</td><td colspan="2">+0,6</td></tr>
<tr><td>60°</td><td colspan="2">+0,7</td><td colspan="2">+0,7</td><td colspan="2">+0,7</td><td>-0,5</td><td>-1,0</td><td colspan="2">-0,5</td><td colspan="2">-0,5</td></tr>
<tr><td>75°</td><td colspan="2">+0,8</td><td colspan="2">+0,8</td><td colspan="2">+0,8</td><td>-0,5</td><td>-1,0</td><td colspan="2">-0,5</td><td colspan="2">-0,5</td></tr>
</table>

APPENDIX 2. SUMMARY OF STRUCTURAL ANALYSIS IN DLUBAL RFEM

The following diagrams exported from Dlubal RFEM are presented in the Appendix:

1. Structural model

2. Global deformations

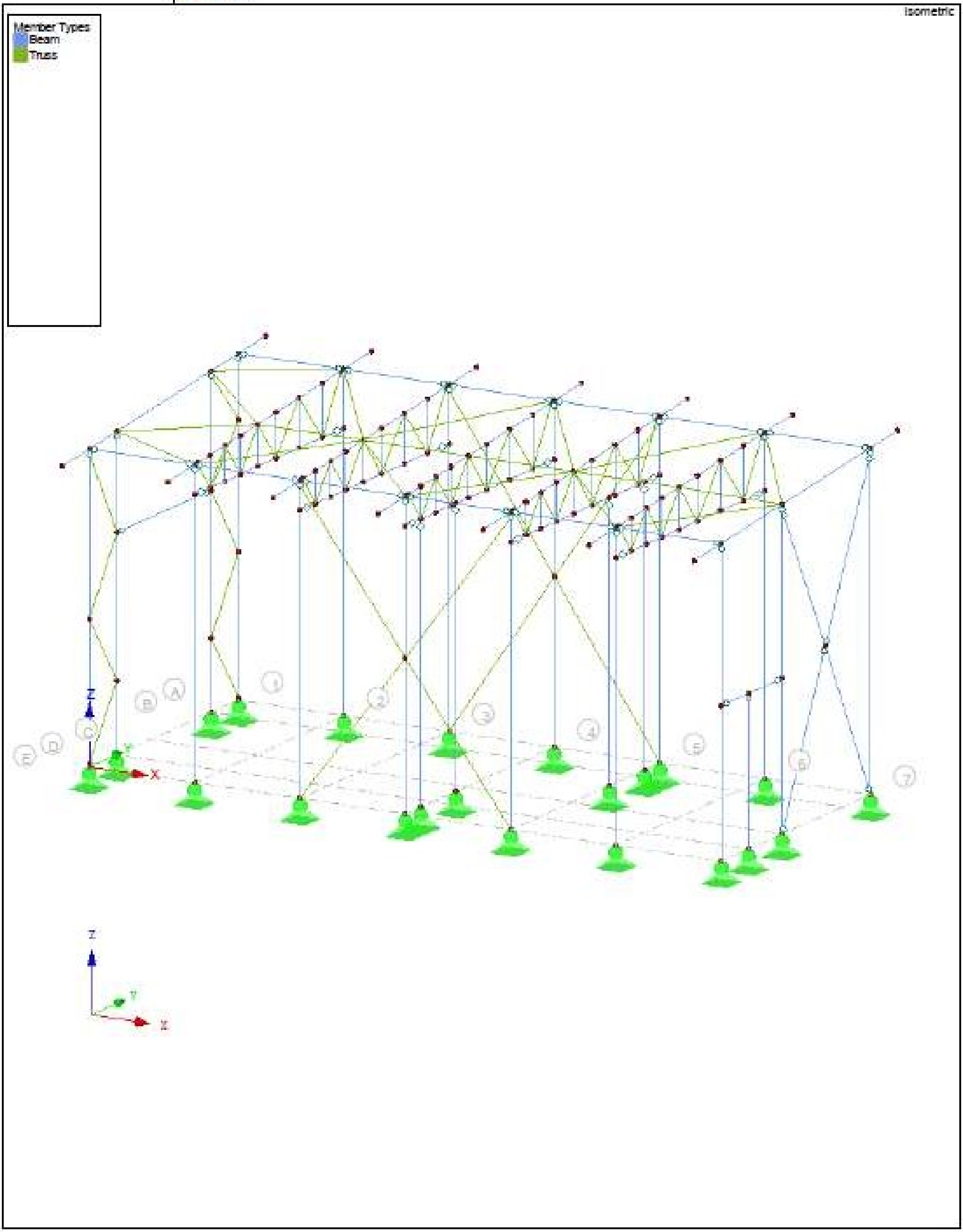
    - $max\ u_x = 26.4mm$
    - $max\ u_y = 31.0mm$

3. Internal forces

    - $max\ N = 38.78kN$
    - $max\ V_y = 6.13kN$
    - $max\ V_z = 23.08kN$
    - $max\ M_y = 4.59kNm$
    - $max\ M_Z = 4.85kNm$

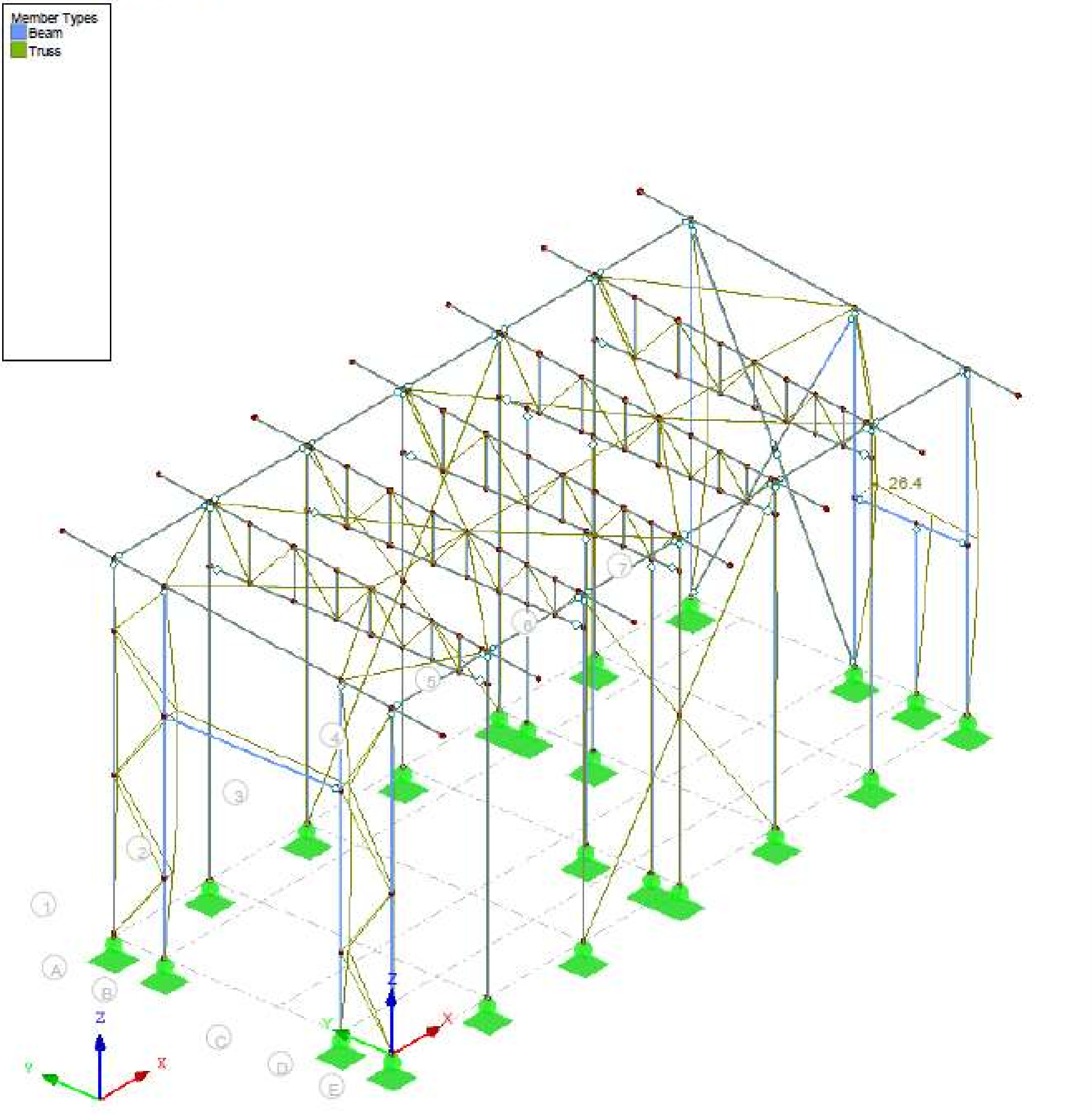
4. Design ratio

MODEL



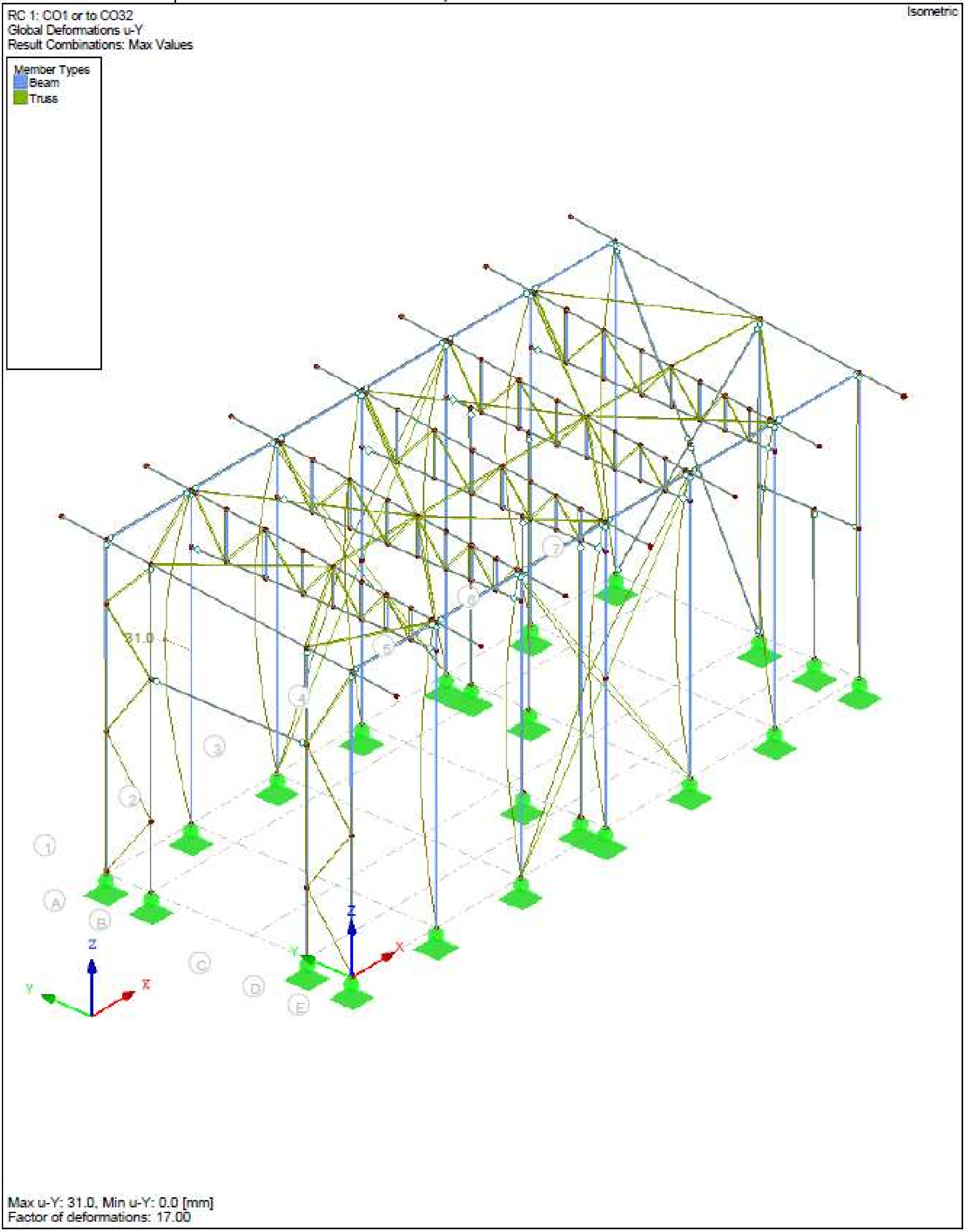
GLOBAL DEFORMATIONS u_X

RC 1: CO1 or to CO32
Global Deformations u-X
Result Combinations: Max Values

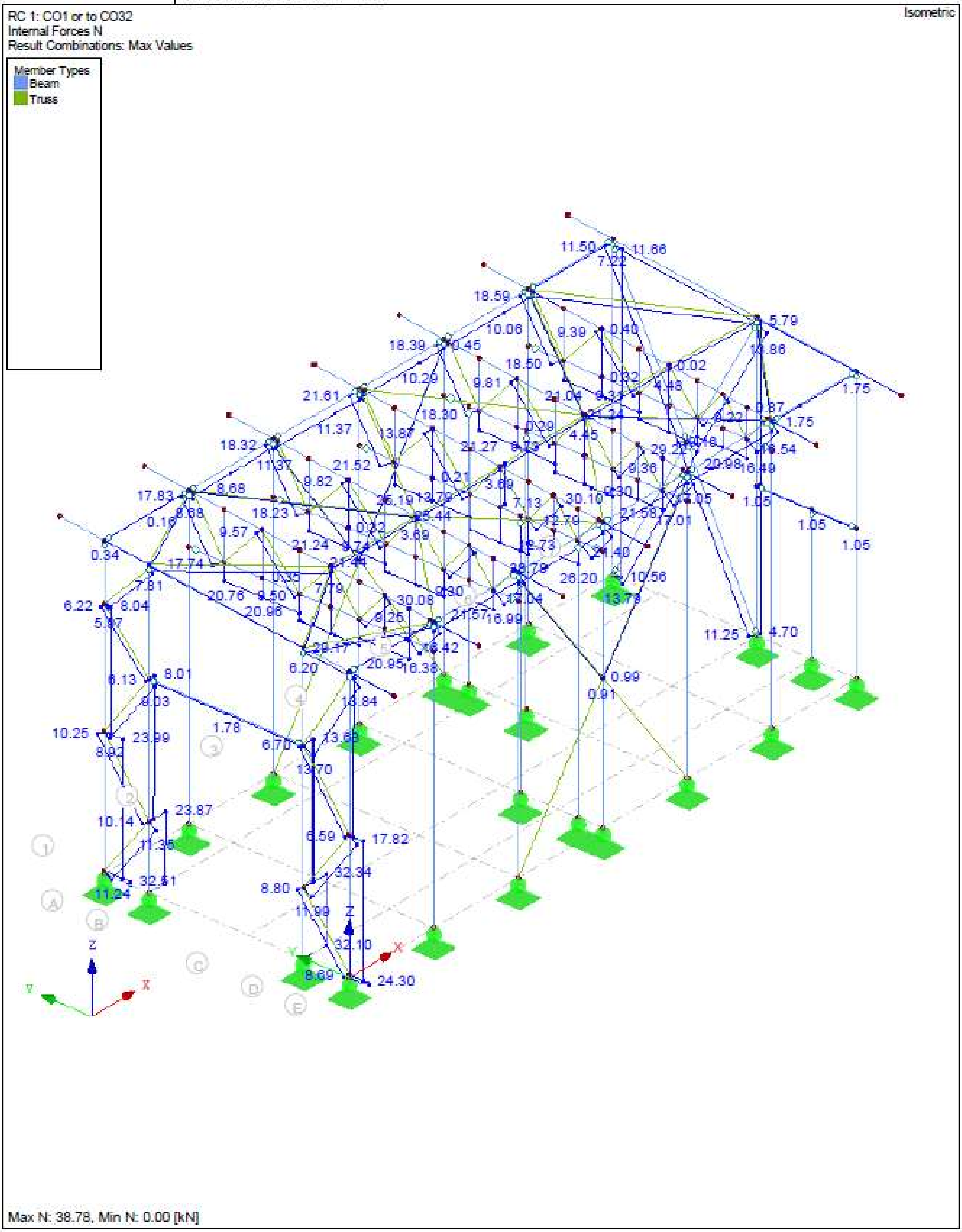


Max u-X: 26.4, Min u-X: 0.0 [mm]
Factor of deformations: 17.00

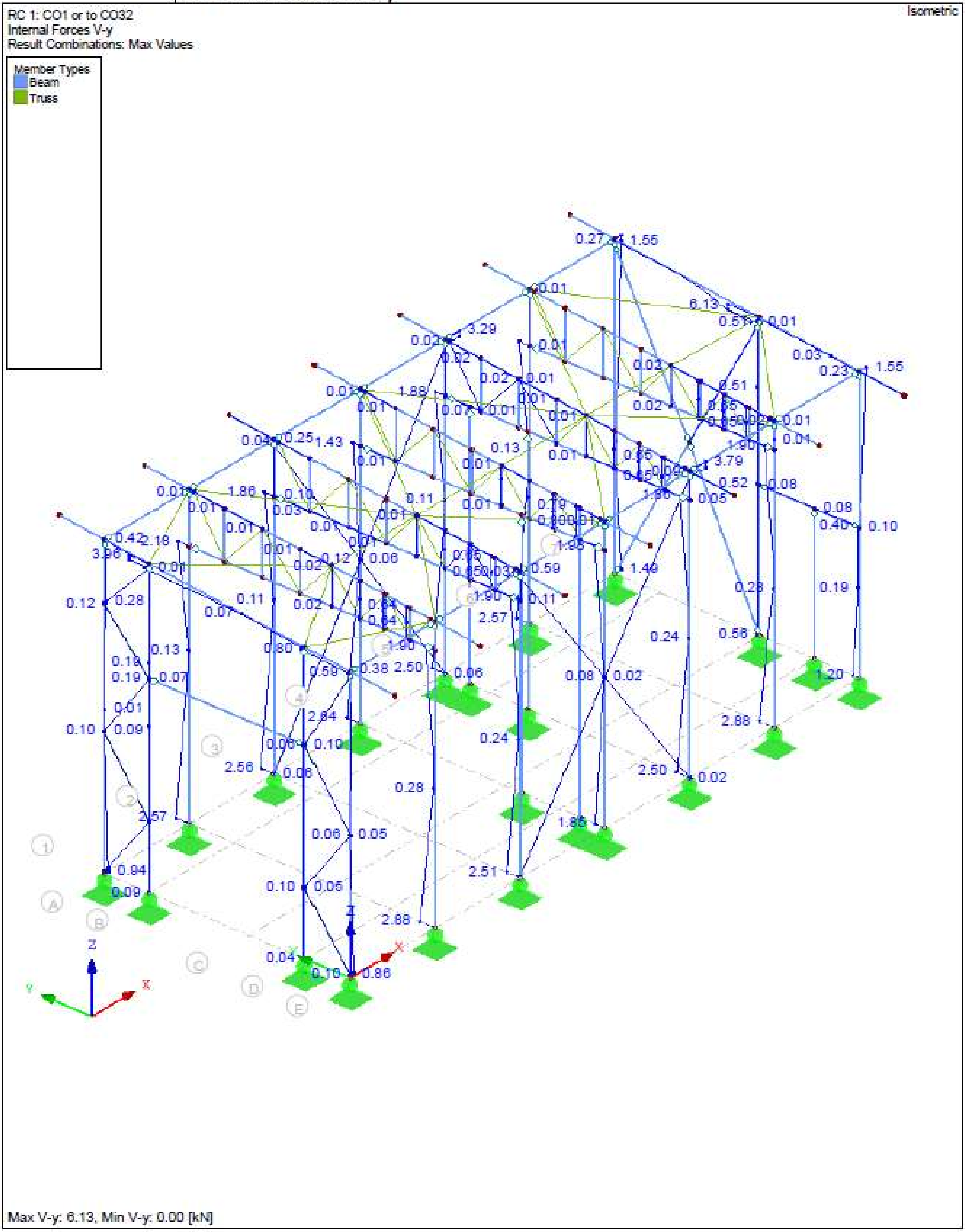
GLOBAL DEFORMATIONS u_Y



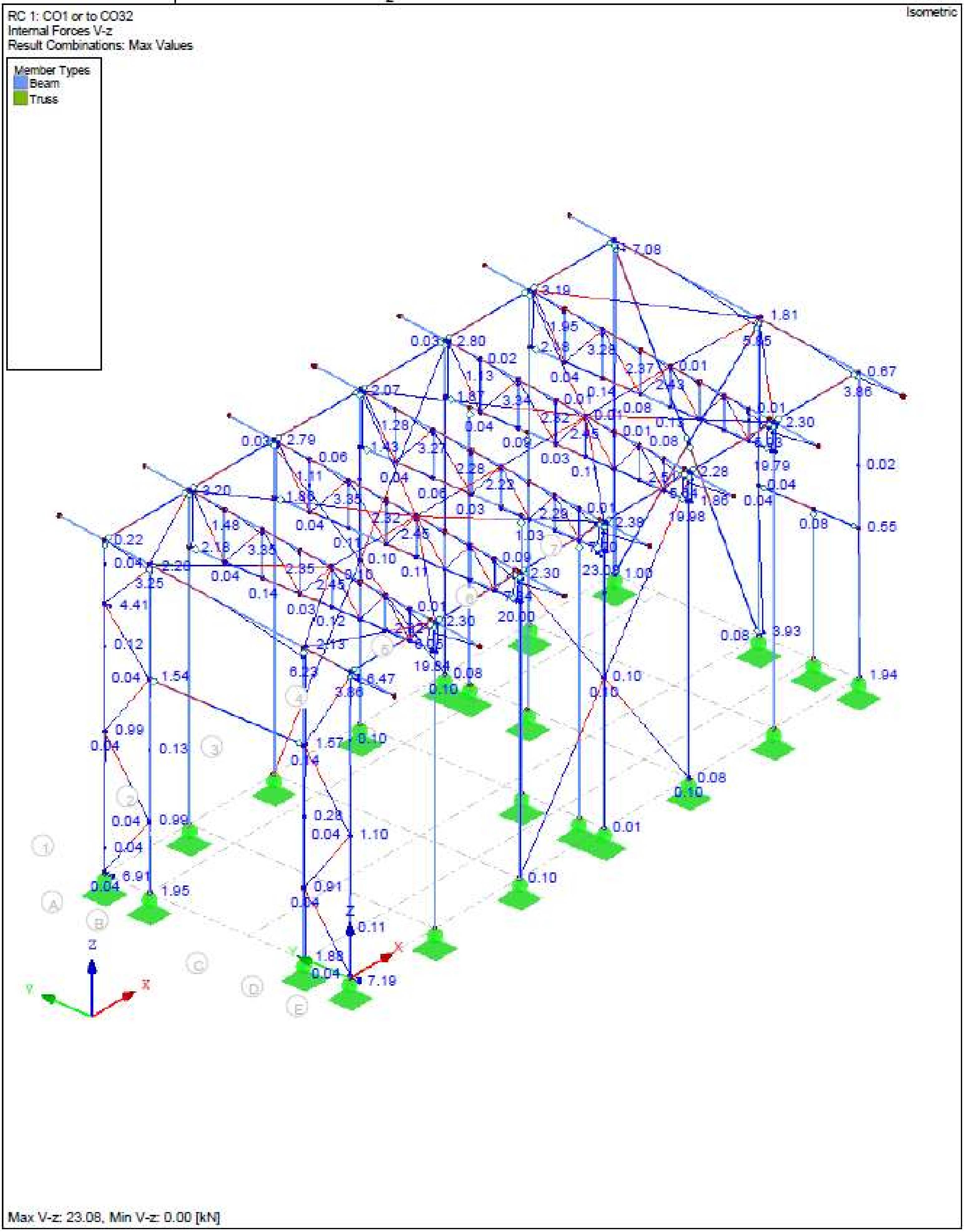
INTERNAL FORCES N



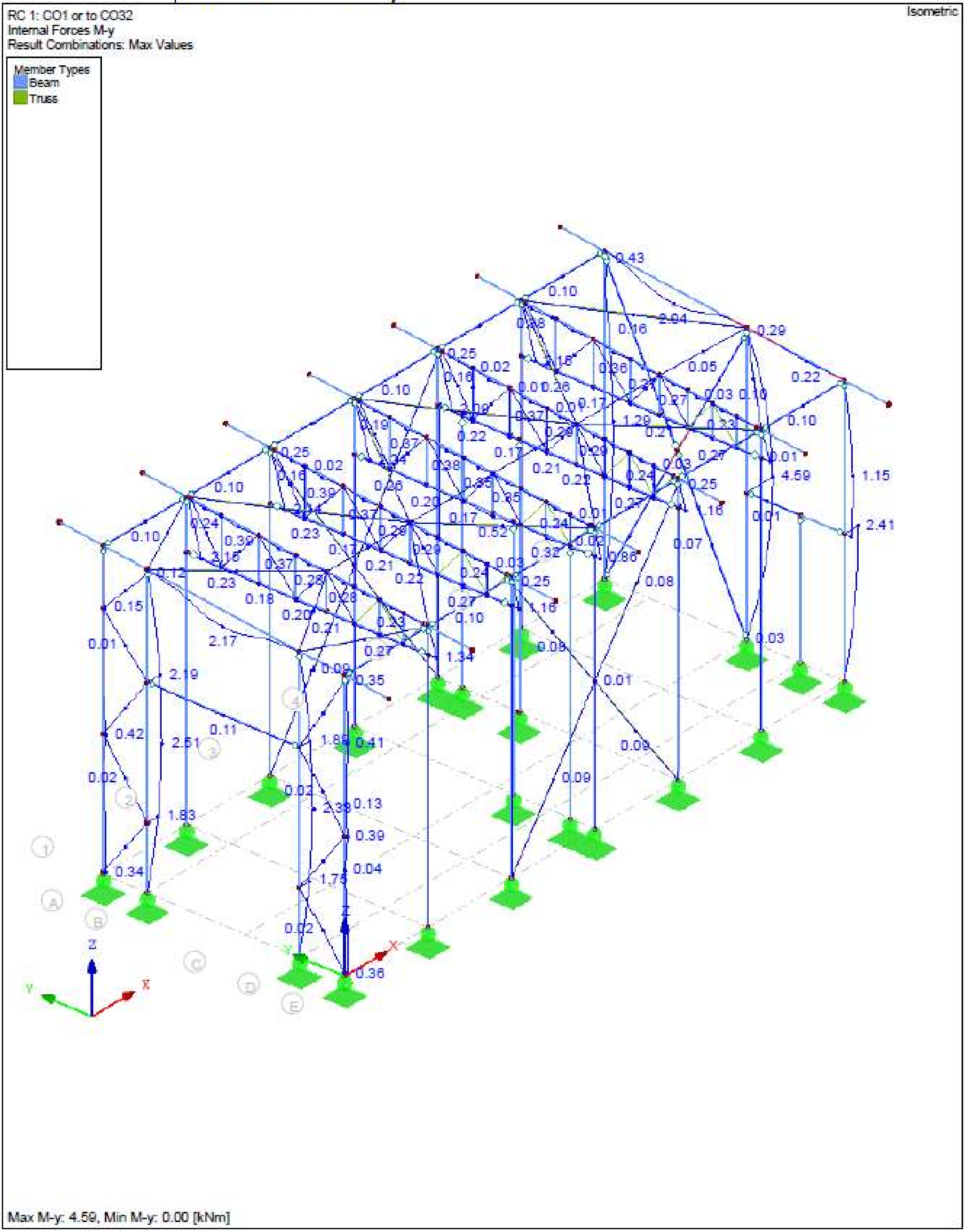
INTERNAL FORCES V_y



INTERNAL FORCES V_z



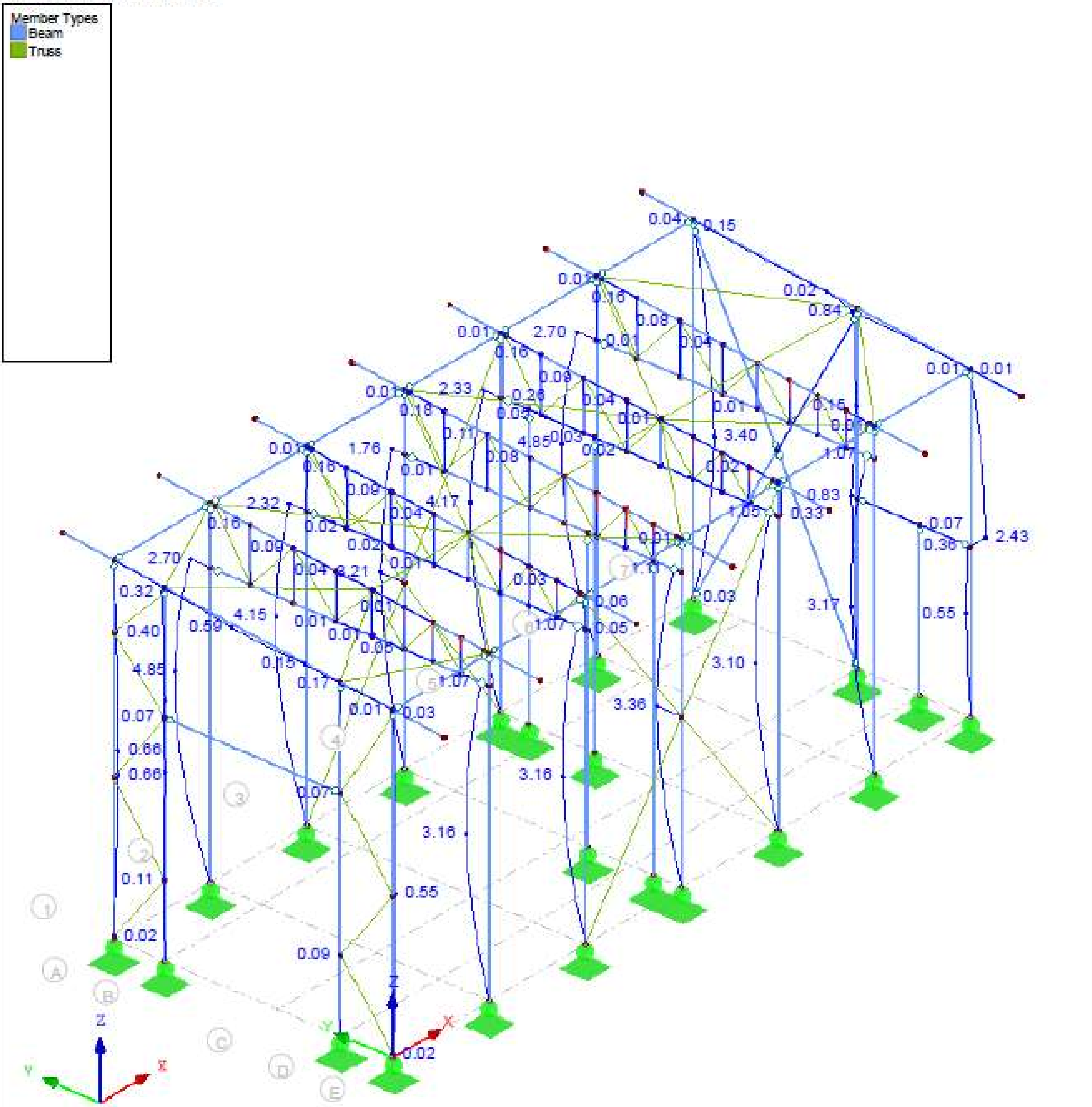
INTERNAL FORCES M_y



INTERNAL FORCES M_z

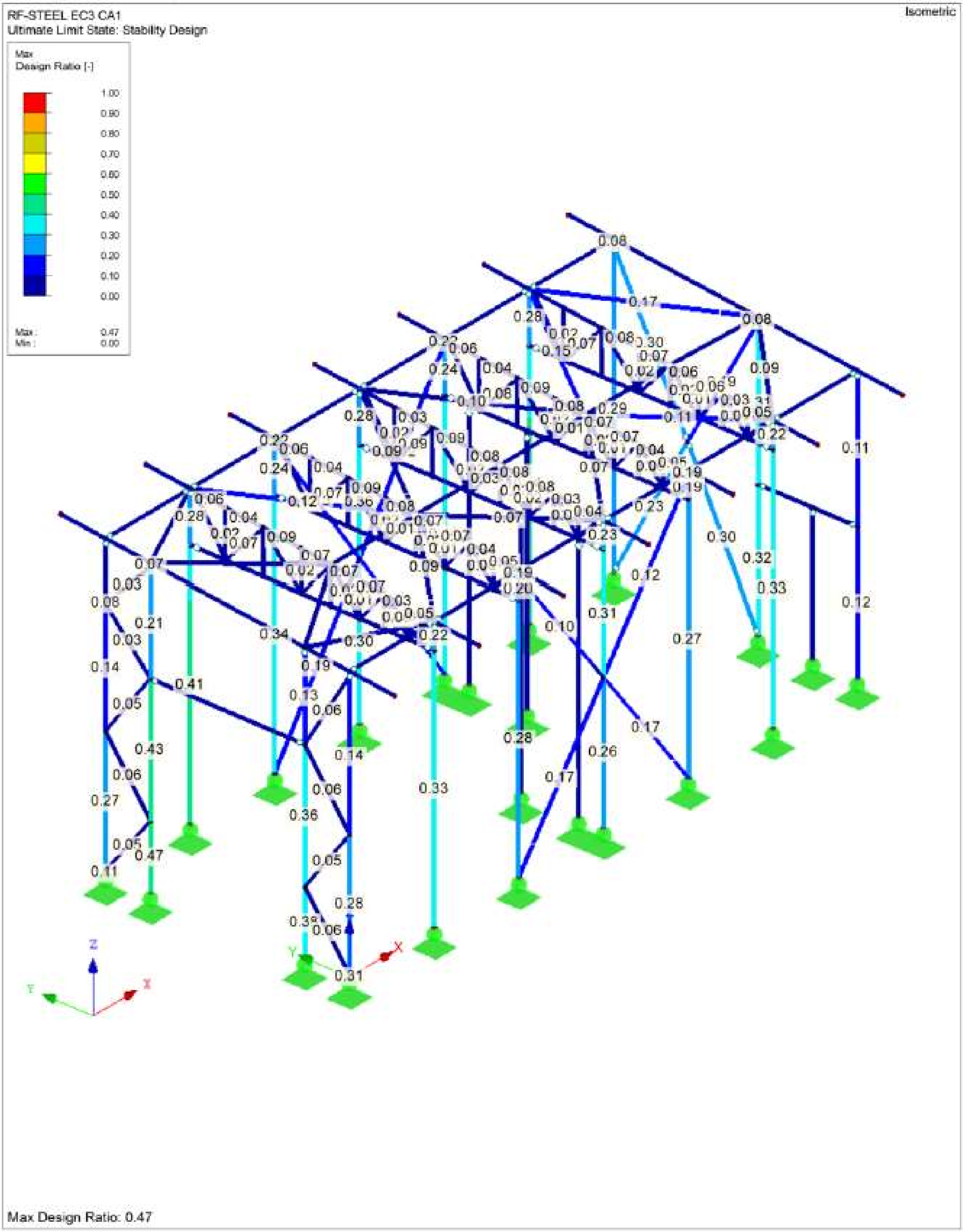
RC 1: CO1 or to CO32
Internal Forces M-z
Result Combinations: Max Values

Isometric



Max M-z: 4.85, Min M-z: 0.00 [kNm]

■ DESIGN: ULTIMATE LIMIT STATE - STABILITY DESIGN

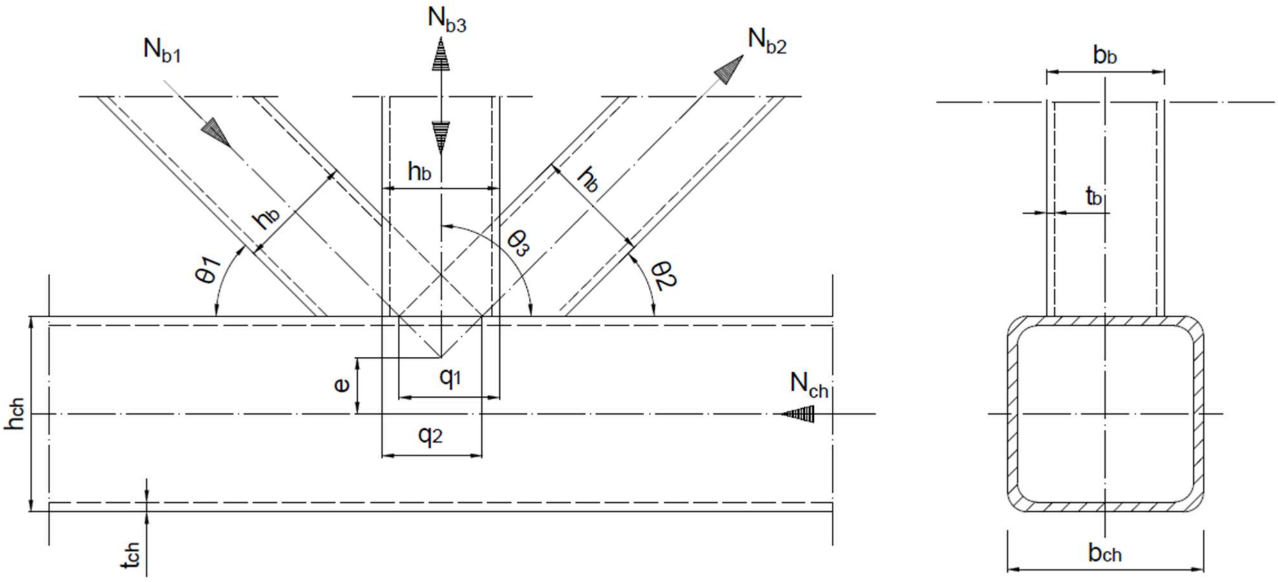


APPENDIX 3. RESISTANCE CALCULATIONS OF JOINTS

Symbols

N_{Ed}	is the design normal force in the cross-section
$N_{i.Ed}$	is the design normal force in the brace member
$N_{i.Rd}$	is the design value of the joint's normal force resistance
V_{Ed}	is the design shear force in the cross-section
$W_{el.i}$	is the elastic section modulus of the brace members
b_{eff}	is the effective width for a brace member to chord connection
$b_{e.ov}$	is the effective width for an overlapping brace (i.e. The brace located on top) to overlapping brace (i.e. the brace located underneath) connection
e	is the eccentricity of the joint
β_p	is the ratio of the brace member width to the width of the reinforcing plate
γ_{M5}	is the partial safety factor for the resistance of the lattice structure joint
η_p	is the ratio of the brace member depth to reinforcing plate width
λ_{ov}	is the overlap ratio of the overlap joint
θ_i	is the smaller angle between the brace member and the chord

1. KT-joint welded truss connection



Main joint data

Configuration	Welded bracing to truss lower chord
Truss lower chord	SHS 100x100x5 S355
Bracing	SHS 60x60x4 S355
Type of connection	Welded connection

Detailed characteristics

Truss lower chord SHS 100x100x5, S355

Depth	$h_{ch} = 100mm$
Width	$b_{ch} = 100mm$
Thickness	$t_{ch} = 5mm$
Area	$A_{ch} = 1836mm^2$
Yield strength	$f_{y,ch} = 355\frac{N}{mm^2}$

Truss webs SHS 60x60x4, S355

Depth	$h_b = 60mm$
Width	$b_b = 60mm$
Thickness of the profile	$t_b = 4mm$
Area	$A_b = 855mm^2$
Angle between the brace and the chord 1	$\theta_1 = 50.082°$
Angle between the brace and the chord 2	$\theta_2 = 51.435°$
Angle between the brace and the chord 3	$\theta_3 = 90°$
Eccentricity of the joint	$e = 0mm$
	$q_1 = 27.94mm$
	$q_2 = 28.51mm$

Yield strength $f_{y,b} = 355\frac{N}{mm^2}$

Safety factors

$\gamma_{m5} = 1$

Applied normal forces

Design normal force in chord $N_{ch,Ed} = 38.78kN$
Design normal force in web member 1 $N_{b1,Ed} = 21.61kN$
Design normal force in web member 2 $N_{b2,Ed} = 21.61kN$
Design normal force in web member 3 $N_{b3,Ed} = 7.22kN$

Validity area

$$\frac{b_b}{b_{ch}} = 0.6 \geq 0.25 \rightarrow \quad ok!$$

$$\frac{h_b}{b_{ch}} = 0.6 \geq 0.25 \rightarrow \quad ok!$$

t.b for all bracing the same so tb1=tb3

$$\frac{t_{b,1}}{t_{b,3}} = 1 \quad ok!$$

The relative value for the overlap

$$\lambda_{ov,1} = q_1 \frac{\sin(\theta_1)}{h_b} = 0.357$$

$$\lambda_{ov,2} = q_2 \frac{\sin(\theta_2)}{h_b} = 0.372$$

$$b_{eff} = 10\ b_b\ t_{ch}^2 \frac{f_{ych}}{\left(b_{ch}\ t_b\ f_{y,b}\right)} = 0.038m$$

$$b_{eff} \leq b_b \qquad ok$$

$$b_{e,ov} = \frac{\left(10\ b_b\ t_b^2\ f_{y,b}\right)}{b_b\ t_b\ f_{y,b}} = 0.04m$$

$b_{e,ov} \leq b_b \qquad ok!$

Case 1 web member failure

$$N_{1,Rd} = f_{y,b}\, t_b \frac{\left(b_{eff} + b_{e,ov} - 4t_b + \frac{\lambda_{ov,1}\ 100\ \ h_b\ 2}{50}\right)}{\gamma_{m5}} = 209.047kN$$

$N_{1Rd} \geq N_{b1Ed} \qquad ok!$

Utility ratio:

$$\frac{N_{b1Ed}}{N_{1Rd}} = 0.103$$

$$N_{2,Rd} = f_{y,b}\, t_b \frac{\left(b_{eff} + b_{e,ov} - 4\ t_b + \frac{\lambda_{ov,2}\ 100\ h_b\ 2}{50}\right)}{\gamma_{m5}} = 213.949kN$$

$N_{2,Rd} \geq N_{b2,Ed} \qquad ok!$

Utility ratio:

$$\frac{N_{b2,Ed}}{N_{2,Rd}} = 0.101$$

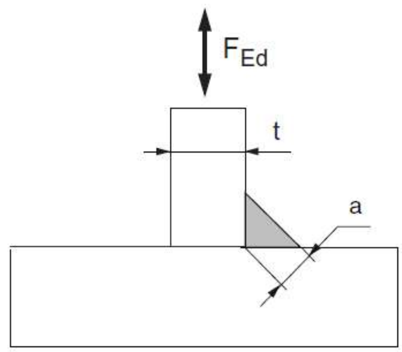
Weld design

Required throat thickness for the weld is:

$\alpha \geq 1.11\ t_b$

$1.11\ t_b = 4.44mm \quad \rightarrow \quad \alpha = 5mm$

Table 3.9 The required throat thickness for an equal strength fillet weld made around the perimeter of the hollow section which is subject to axial tension, compression and/or bending

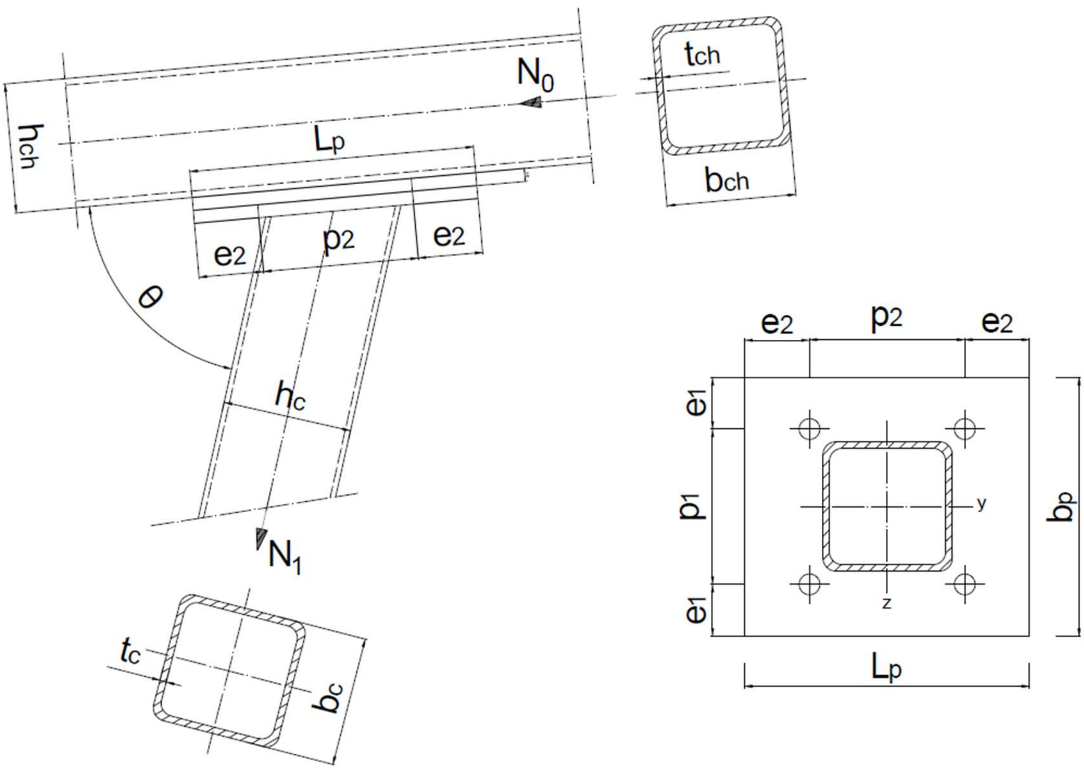


Steel grade	Yield strength a) f_y (N/mm^2)	Ultimate tensile strength a) f_u (N/mm^2)	Throat thickness of the weld b)
S235H	235	360	0,92·t
S275H	275	430	0,96·t
S355H	**355**	**510**	**1,11·t**
S275NH	275	370	1,12·t
S355NH	355	470	1,20·t
S460NH	460	550	1,48·t
S275MH	275	360	1,15·t
S355MH	355	470	1,20·t
S420MH	**420**	**500**	**1,48·t**
S460MH	460	530	1,53·t

a) Nominal strength values according to Table 1.7.
b) However at least a ≥ 3 mm.

- The values in this table have been calculated using the recommended values $\gamma_{M0} = 1,0$ and $\gamma_{M2} = 1,25$ as given in Parts EN 1993-1-1 and EN 1993-1-8 of Eurocode. **National requirements must be checked from the National Annex of the relevant country.**
- The throat thickness values of this table are valid:
  - when the hollow section to be joined is subject to axial tension or compression and/or bending
  - when the weld is made around the perimeter of the hollow section to be joined
  - when the hollow section to be joined is of the same grade or lower grade than the adjacent member
  - when using a welding consumable having its yield strength and ultimate tensile strength at least equivalent minimum values as those of the hollow section to be joined.

2. Column-to-truss connection



Main joint data

Configuration	Lower truss chord to column
Truss lower chord	SHS 100x100x5 S355
Column	SHS 100x100x5 S355
Type of connection	End plate at the support connection

Detailed characteristics

Truss lower chord SHS 100x100x5, S355

Depth	$h_{ch} = 100mm$
Width	$b_{ch} = 100mm$
Thickness	$t_{ch} = 5mm$
Area	$A_{ch} = 1836mm^2$
Yield strength	$f_{y,ch} = 355\frac{N}{mm^2}$
Ultimate strength	$f_{u,ch} = 490\frac{N}{mm^2}$
Elastic section modulus	$W_{el,ch} = 54220mm^3$

Column SHS 100x100x5, S355

Depth	$h_c = 100mm$
Width	$b_c = 100mm$
Thickness of the profile	$t_c = 5mm$
Area	$A_c = 855mm^2$
	$\theta = 84.29°$
Yield strength	$f_{y,c} = 355\frac{N}{mm^2}$
Ultimate strength	$f_{u,c} = 490\frac{N}{mm^2}$
Elastic section modulus	$W_{el,c} = 54220mm^3$

Plate 220x200x10, S355

Depth	$h_p = 220mm$
Width	$b_p = 200mm$
Thickness	$t_p = 10mm$
	$e_1 = 40mm$
	$e_2 = 50mm$
	$p_1 = 120mm$

$p_2 = 120mm$

Yield strength $f_{y,p} = 355 \frac{N}{mm^2}$

Ultimate strength $f_{u,p} = 490 \frac{N}{mm^2}$

Bolts M16, 8.8

Tensile stress area $A_s = 157mm^2$
Diameter of the shank $d = 16mm$
Diameter of the holes $d_0 = 18mm$
$d_m = 25.1mm$
Area of a bolt $A_{bolt} = 201mm^2$
Number of bolts $n_{bolts} = 4$
Number of rows $n_{rows} = 2$

Yield strength $f_{y,bolt} = 640 \frac{N}{mm^2}$

Ultimate strength $f_{u,bolt} = 800 \frac{N}{mm^2}$

Welds

Throat thickness of the weld $\alpha = 6mm$
$\beta = 0.9$

Safety factors

$\gamma_{M0} = 1$
$\gamma_{M2} = 1.25$
$\gamma_{m5} = 1$

Applied forces

Design normal force in the cross-section $N_{Ed} = 31.04kN$
Design shear force $V_{y,Ed} = -3.79kN$
Design shear force $V_{z,Ed} = -23.08kN$
Design bending moment in column $M_{y,Ed} = 3.48kNm$
Design bending moment in column $M_{z,Ed} = 0.37kNm$

$$N_{Ed,f} = \left[4\left(\frac{M_{y,Ed}}{2\,p_1} + \frac{M_{z,Ed}}{2\,p_2} + \frac{N_{Ed}}{4}\right)\right] = 95.207kN$$

Validity range

$e_1 \geq 1.2d_0 \qquad ok!$

$e_2 \geq 1.2d_0 \qquad ok!$

$2.2d_0 \leq p_1 \qquad ok!$

$2.4d_0 \leq p_2 \qquad ok!$

$h_p = p_2 + 2e_2 \qquad ok!$

$b_p = p_1 + 2e_1 \qquad ok!$

__**Chord resistance**__

Chord face failure by yielding

$$\beta_p = \frac{b_{ch}}{b_p} = 0.5$$

$$\eta_p = \frac{h_{ch}}{b_p} = 0.5$$

$$N_{1,Rd} = \frac{f_{y,p}}{\left(1-\beta_p\right)\sin(\theta)}\ t_p^2\ \frac{\left(2\frac{\eta_p}{\sin(\theta)} + 4\sqrt{1-\beta_p}\right)}{\gamma_{M5}} = 273.53kN$$

$N_{1,Rd} \geq N_{Ed,f} \qquad ok!$

Chord side wall buckling or yielding

$$\lambda = 3.46\left(\frac{h_{ch}}{t_{ch}} - 2\right)\frac{\sqrt{\frac{1}{\sin(\theta)}}}{\pi\sqrt{\frac{E}{f_{y,ch}}}} = 0.817$$

$$\chi = \frac{1}{\varphi + \sqrt{\varphi^2 - \lambda^2}} \leq 1 \ \ for\ \lambda > 0.2$$

$$\varphi = 0.5\left(1 + \alpha(\lambda - 0.2) + \lambda^2\right)$$

$$\varphi = 0.5\left(1 + 0.49(\lambda - 0.2) + \lambda^2\right) = 0.985$$

$$\chi = \frac{1}{\varphi + \sqrt{\varphi^2 - \lambda^2}} = 0.651$$

$$f_{buckling} = \chi\, f_{y,ch} = 2.313\ \times 10^8 Pa \quad Compression\ in\ chord$$

$$N_{2,Rd} = \frac{f_{buckling}}{\sin(\theta)}\ t_{ch}\ \frac{\left(2\ \frac{h_{ch}}{\sin(\theta)} + 10\ t_{ch}\right)}{\gamma_{M5}} = 291.663kN$$

$$N_{2,Rd} \geq N_{Ed,f} \quad ok!$$

Chord face punching shear

$$b_{e,p} = \frac{10}{\frac{b_p}{t_p}}\ b_{ch} = 0.05m$$

$$N_{3,Rd} = \frac{f_{y,p}}{\sqrt{3}\sin(\theta)}\ t_p\ \frac{\left(2\ b_{e,p} + 2\ \frac{h_{ch}}{\sin(\theta)}\right)}{\gamma_{M5}} = 619.998kN$$

$$N_{3,Rd} \geq N_{Ed,f} \quad ok!$$

Column resistance

Column failure:

$$b_{eff} = \frac{10}{\frac{b_p}{t_p}} \cdot \frac{f_{y,p}}{f_{y,c}\ t_c}\ t_p\ b_c = 0.1m$$

$$N_{4,Rd} = f_{y,c}\ t_c\ \frac{\left(2\ h_c - 4\ t_c + 2\ b_{eff}\right)}{\gamma_{M5}} = 674.5kN$$

$$N_{4,Rd} \geq N_{Ed,f} \quad ok!$$

Design of bolts

Tension resistance:

$$F_{t,Rd} = 0.9\ f_{u,bolt}\ \frac{A_s}{\gamma_{M2}} = 90.432kN$$

$$n_{bolt}\ F_{t,Rd} = 361.728kN$$

$$n_{bolt}\ F_{t,Rd} \geq N_{Ed,f} \quad ok!$$

Utility ratio:

$$\frac{N_{Ed,f}}{n_{bolt}\ F_{t,Rd}} = 0.219$$

$$\frac{N_{Ed,f}}{n_{bolt}\ F_{t,Rd}} < 1 \quad ok!$$

Punching shear resistance:

$$N_{p,Rd} = 0.6\,\pi\, d_m\, t_p\ \frac{f_{u,p}}{\gamma_{M2}} = 1.855\ \times 10^5 N$$

$$n_{bolt}\ N_{p,Rd} > N_{Ed,f} \quad ok!$$

$$V_d = \sqrt{V_{y,Ed}^2 + V_{z,Ed}^2} = 2.339\ \times 10^4 N$$

$$F_{v,Rd} = 0.6\, f_{u,bolt}\ \frac{A_{bolt}}{\gamma_{M2}} = 7.718\ \times 10^4 N$$

Utility ratio:

$$\frac{N_{Ed,f}}{n_{bolt}\ N_{p,Rd}} = 0.107\ < 1 \quad ok!$$

Shear resistance:

$$n_{bolt}\ F_{v,Rd} > V_d \quad ok!$$

Design of plates

$$\delta = 1 - \frac{d_0}{p_2} = 0.85$$

$$b_{prim} = \max\left(t_c + \frac{(p_2 - b_c)}{2}, t_c + \frac{(p_1 - h_c)}{2}\right) = 0.015m$$

$$K = 4\ \frac{b_{prim}}{\left(0.9\ \frac{f_{y,p}}{\gamma_{M0}}\ p_2\right)} = 1.565\ \times 10^{-3} \frac{mm^2}{N}$$

$$\mathrm{K}\left[\frac{F_{t,Rd}}{(1+\delta)}\right]^{0.5} = 8.746\ \times 10^{-3}m$$

$$\left(\mathrm{K}\,F_{t,Rd}\right)^{0.5} = 0.012m$$

$$\mathrm{K}\left[\frac{F_{t,Rd}}{(1+\delta)}\right]^{0.5} \le t_p \le \left(\mathrm{K}\,F_{t,Rd}\right)^{0.5}\quad ok!$$

$$\mathrm{a} = \left(\mathrm{K}\ \frac{F_{t,Rd}}{t_p^2} - 1\right)\left[\frac{\left(e_2 + \frac{d}{2}\right)}{\delta\ (e_2 + e_1 + t_c)}\right] = 0.298$$

$$N_{t,Rd} = t_p^2\ (1 + \delta\ a)\ \frac{n_{bolt}}{K\ \gamma_{M2}} = 2.563\ \times 10^5 N$$

$$N_{t,Rd} > N_{Ed,f}\quad ok!$$

Weld design

Chord to plate:

$$\beta_w = 0.9$$

$$\alpha_{ch} = 6mm$$

$$W_{el,p} = b_p\ \frac{h_p^2}{6} = 1.613\ \times 10^6 mm^3$$

$$\sigma_1 = \left(\frac{M_{y,Ed}}{W_{el,p}} + \frac{M_{z,Ed}}{W_{el,p}}\right)\frac{t_p}{2\ \alpha_{ch}\ \sqrt{2}} = 1.406\frac{N}{mm^2}$$

$$\tau_1 = \sigma_1$$

σ_1 is calculated without compression Axial force because it is reducing the moment. The maximum moment is taken in this case.

$$\tau_2 = \frac{V_{z,Ed}}{2\ \alpha_{ch}\ h_p} = -8.742\frac{N}{mm^2}$$

$$\sigma_f = \sqrt{\sigma_1^2 + 3\ (\tau_1^2 + \tau_2^2)} = 15.401\frac{N}{mm^2}$$

$$\frac{f_{u,ch}}{\gamma_{M2}\ \beta_w} = 435.556\frac{N}{mm^2}$$

$$\sigma_f \leq \frac{f_{u,ch}}{\gamma_{M2}\,\beta_w} \qquad ok!$$

$$\alpha_{ch} \geq 5.75mm \quad ok!$$

Column to plate:

Moment and axial and shear force Z-Direction

$$\alpha_c = 6mm$$

$$\sigma_1 = \left(\frac{M_{y,Ed}}{W_{el,p}} + \frac{M_{z,Ed}}{W_{el,p}}\right) \frac{t_p}{2\,\alpha_c\,\sqrt{2}} = 1.406\frac{N}{mm^2}$$

$$\tau_1 = \sigma_1$$

σ_1 is calculated without compression Axial force because it is reducing the moment. The maximum moment is taken in this case.

$$\tau_2 = \frac{V_{z,Ed}}{2\,\alpha_c\,b_c} = -19.233\frac{N}{mm^2}$$

$$\sigma_{f,z} = \sqrt{\sigma_1^2 + 3(\tau_1^2 + \tau_2^2)} = 33.432\frac{N}{mm^2}$$

$$\frac{f_{u,c}}{\gamma_{M2}\,\beta_w} = 435.556\frac{N}{mm^2}$$

$$\sigma_{f,z} \leq \frac{f_{u,c}}{\gamma_{M2}\,\beta_w} \qquad ok!$$

Moment and axial and shear force Y-Direction

$$\sigma_1 = \left(\frac{M_{y,Ed}}{W_{el,p}} + \frac{M_{z,Ed}}{W_{el,p}}\right) \frac{t_p}{2\,\alpha_c\,\sqrt{2}} = 1.406\frac{N}{mm^2}$$

$$\tau_1 = \sigma_1$$

σ_1 is calculated without compression Axial force because it is reducing the moment. The maximum moment is taken in this case.

$$\tau_2 = \frac{V_{y,Ed}}{2\,\alpha_c\,b_c} = -3.158\frac{N}{mm^2}$$

$$\sigma_{f,y} = \sqrt{\sigma_1^2 + 3(\tau_1^2 + \tau_2^2)} = 6.151\frac{N}{mm^2}$$

$$\frac{f_{u,c}}{\gamma_{M2}\,\beta_w} = 435.556\frac{N}{mm^2}$$

$$\sigma_{f,y} \leq \frac{f_{u,c}}{\gamma_{M2}\,\beta_w} \qquad ok!$$

$$0.7\,\beta_w\,\frac{t_c}{\sqrt{2}} = 2.227mm$$

$$\alpha_c \geq 0.7\,\beta_w\,\frac{t_c}{\sqrt{2}} \quad ok!$$

Required throat thickness for the weld is

$$\alpha_c \geq 5.75mm \quad ok!$$

Table 3.12 Welds between different steel grades. The required throat thickness for an equal strength fillet weld made around the perimeter of the hollow section which is subject to axial tension, compression and/or bending

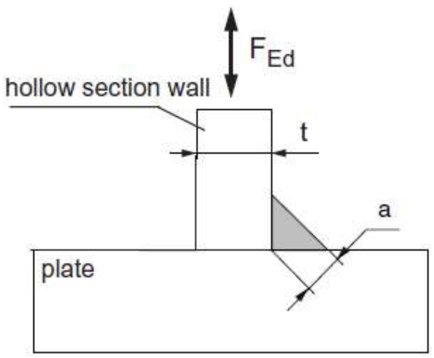
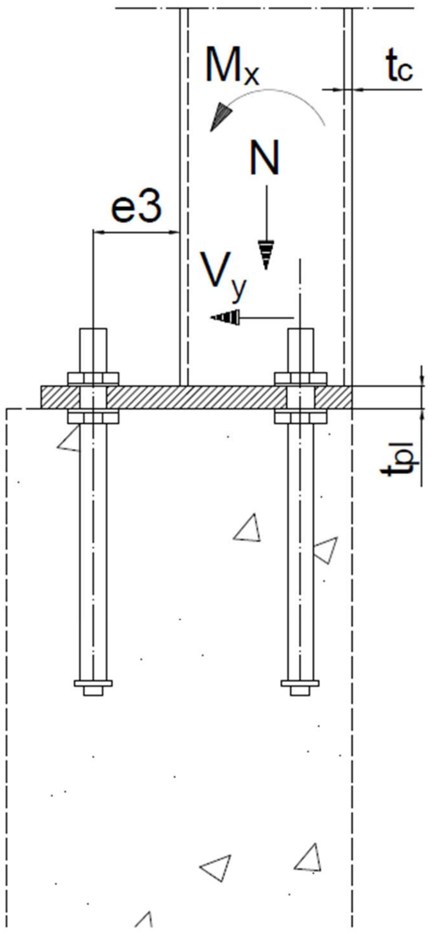
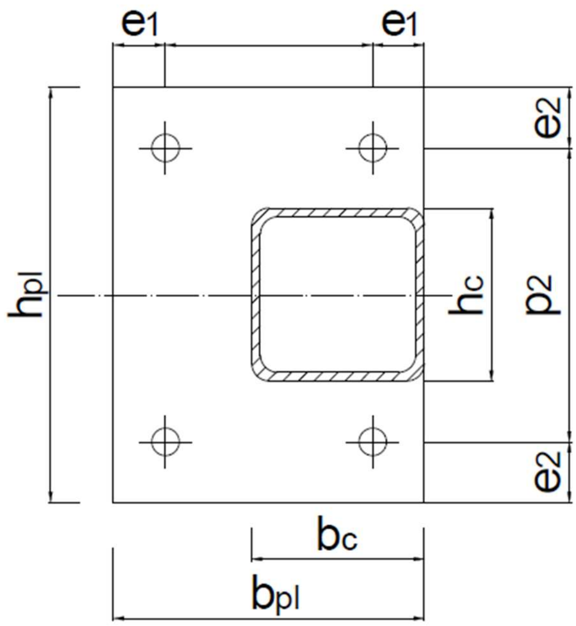


Plate		Throat thickness of the weld b)	
Steel grade	Ultimate tensile strength a) f_u (N/mm^2)	Structural hollow section S355H	Structural hollow section S420MH
S235	360	1,39·t	1,65·t
S275	430	1,24·t	1,48·t
S355	**490**	**1,15·t**	**1,36·t**
S275N	390	1,37·t	1,62·t
S355N	490	1,15·t	1,36·t
S420N	520	1,11·t	1,48·t
S460N	540	1,11·t	1,48·t
S275M	370	1,44·t	1,71·t
S355M	470	1,20·t	1,42·t
S420M	520	1,11·t	1,48·t
S460M	540	1,11·t	1,48·t

a) Nominal values of the ultimate tensile strength presented in EN 1993-1-1 for flat steels in grades S235-S460 conforming to EN 10025, when $t \leq 40$ mm.
b) However at least $a \geq 3$ mm.

- The values in this table have been calculated using the recommended values $\gamma_{M0} = 1{,}0$ and $\gamma_{M2} = 1{,}25$ as given in Parts EN 1993-1-1 and EN 1993-1-8 of Eurocode. **National requirements must be checked from the National Annex of the relevant country.**
- The throat thickness values of this table are valid:
  - when the hollow section to be joined is subject to axial tension or compression and/or bending
  - when the weld is made around the perimeter of the hollow section to be joined
  - when using a welding consumable having its yield strength and ultimate tensile strength at least equivalent minimum values as those of the weaker part to be joined.





Main joint data

Configuration	Column to foundation connection
Truss lower chord	SHS 100x100x5 S355
Type of connection	Base plate connection

Detailed characteristics

Column SHS 100x100x5, S355

Depth	$h_c = 100mm$
Width	$b_c = 100mm$
Thickness	$t_c = 5mm$
Area	$A_c = 1836mm^2$
Yield strength	$f_{y,c} = 355\frac{N}{mm^2}$
Elastic section modulus	$W_{el,c} = 54220mm^3$

Plate 240x180x13, S355

Depth	$h_p = 240mm$
Width	$b_p = 180mm$
Thickness	$t_p = 20mm$
	$e_1 = 30mm$
	$e_2 = 35mm$
	$e_3 = 50mm$
	$p_1 = 120mm$
	$p_2 = 170mm$
Yield strength	$f_{y,p} = 355\frac{N}{mm^2}$
Ultimate strength	$f_{u,p} = 490\frac{N}{mm^2}$

Bolts HPM/L 16

Tensile stress area	$A_s = 157mm^2$
Diameter of the shank	$d = 16mm$
Diameter of the holes	$d_0 = 18mm$
	$d_m = 25.1mm$
Area of a bolt	$A_{bolt} = 201mm^2$
Number of bolts	$n_{bolts} = 4$
Number of rows	$n_{rows} = 2$
Ultimate tensile strength of a bolt	$F_{u,bolt} = 65.4kN$
Ultimate shear strength of a bolt	$V_{u,bolt} = 14.7kN$

Welds

Throat thickness of the weld	$\alpha = 6mm$
	$\beta_w = 0.9$

Safety factors

$\gamma_{M0} = 1$
$\gamma_{M2} = 1.25$

Applied forces

Design normal force in the cross-section	$N_{Ed} = 42.20kN$
Design shear force	$V_{y,Ed} = -4.85kN$
	$V_{x,Ed} = 3.6kN$
	$M_{x,Ed} = 4.40kNm$

$$M_{y,Ed} = 4.87kNm$$

Combined stress:

$$F_{Ed} = \frac{M_{x,Ed}}{2\,p_1} + \frac{M_{y,Ed}}{2\,p_2} - \frac{N_{Ed}}{3} = 22.107kN$$

$$F_{u,bolt} \geq F_{Ed} \qquad ok!$$

$$V_{Ed} = \frac{\sqrt{V_{y,Ed}^2 + V_{x,Ed}^2}}{4} = 1.51kN$$

$$V_{u,bolt} \geq V_{Ed} \qquad ok!$$

$$Ratio = \frac{V_{Ed}^{\frac{4}{3}}}{V_{u,bolt}^{\frac{4}{3}}} + \frac{F_{Ed}^{\frac{4}{3}}}{F_{u,bolt}^{\frac{4}{3}}} = 0.284 \;\leq 1 \quad ok!$$

Plate:

$$e_4 = \sqrt{\left[\frac{(h_p - h_c - 2e_2)}{2}\right]^2 + e_3^2} = 61mm$$

$$M_{1,Ed} = F_{Ed}\, e_4 = 1.349kNm$$

E4 is the distance between the axial force point of application (centre of the column's profile) to the middle of the distance between the bolts
B is the length of the line on which the plate will bend. In W_{pl} it is divided by 5 because of plastic deformation.

$$B = \left(\frac{\left[\frac{(h_p - 2e_2 - h_c)}{2}\right] + e_2}{\sin(45°)}\right) + \frac{(b_p - b_c)}{\sin(45°)} = 0.212m$$

$$W_{el,p} = B\,\frac{t_p^2}{5} = 1.697\; \times 10^4 mm^3$$

$$f_d = \frac{M_{1,Ed}}{W_{el,p}} = 79.505\,\frac{N}{mm^2}$$

$$f_{y,p} = 355\,\frac{N}{mm^2}$$

$$f_{y,p} \geq f_d \qquad ok!$$

Weld design:

Moment and axial and shear force X direction

$$f_u = 490 \frac{N}{mm^2}$$

$$\sigma_1 = \left(\frac{M_{x,Ed}}{W_{el,c}} + \frac{M_{y,Ed}}{W_{el,c}}\right) \frac{t_p}{2\ \alpha\ \sqrt{2}} = 201.49 \frac{N}{mm^2}$$

$$\tau_1 = \sigma_1$$

σ_1 is calculated without compression Axial force because it is reducing the moment. The maximum moment is taken in this case.

$$\tau_2 = \frac{V_{x,Ed}}{2\ \alpha\ b_c} = 3 \frac{N}{mm^2}$$

$$\sigma_{f,x} = \sqrt{\sigma_1^2 + 3\ (\tau_1^2 + \tau_2^2)} = 403.014 \frac{N}{mm^2}$$

$$\frac{f_u}{\gamma_{m2}\ \beta_w} = 435.556 \frac{N}{mm^2}$$

$$\sigma_{f,x} \leq \frac{f_u}{\gamma_{m2}\ \beta_w} \qquad ok!$$

Moment and axial and shear force Y direction

$$\sigma_1 = \left(\frac{M_{x,Ed}}{W_{el,c}} + \frac{M_{y,Ed}}{W_{el,c}}\right) \frac{t_p}{2\ \alpha\ \sqrt{2}} = 201.49 \frac{N}{mm^2}$$

$$\tau_1 = \sigma_1$$

σ_1 is calculated without compression Axial force because it is reducing the moment. The maximum moment is taken in this case.

$$\tau_2 = \frac{V_{y,Ed}}{2\,\alpha\,b_c} = -4.042\,\frac{N}{mm^2}$$

$$\sigma_{f,y} = \sqrt{\sigma_1^2 + 3\,(\tau_1^2 + \tau_2^2)} = 403.041\,\frac{N}{mm^2}$$

$$\frac{f_u}{\gamma_{m2}\,\beta_w} = 435.556\,\frac{N}{mm^2}$$

$$\sigma_{f,y} \le \frac{f_u}{\gamma_{m2}\,\beta_w} \qquad ok!$$

$$0.7\,\beta_w\,\frac{t_c}{\sqrt{2}} = 2.227mm$$

$$\alpha \ge 0.7\,\beta_w\,\frac{t_c}{\sqrt{2}} \qquad ok!$$

Required throat thickness for the weld is:

$$\alpha \ge 1.15\,t_c \qquad ok!$$

Table 3.12 Welds between different steel grades. The required throat thickness for an equal strength fillet weld made around the perimeter of the hollow section which is subject to axial tension, compression and/or bending

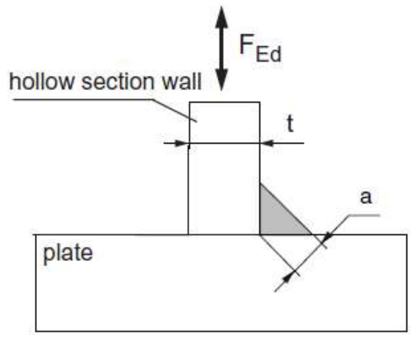


Plate		Throat thickness of the weld b)	
Steel grade	Ultimate tensile strength a) f_u (N/mm^2)	Structural hollow section S355H	Structural hollow section S420MH
S235	360	1,39·t	1,65·t
S275	430	1,24·t	1,48·t
S355	**490**	**1,15·t**	**1,36·t**
S275N	390	1,37·t	1,62·t
S355N	490	1,15·t	1,36·t
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S355M	470	1,20·t	1,42·t
S420M	520	1,11·t	1,48·t
S460M	540	1,11·t	1,48·t

a) Nominal values of the ultimate tensile strength presented in EN 1993-1-1 for flat steels in grades S235-S460 conforming to EN 10025, when t ≤ 40 mm.
b) However at least a ≥ 3 mm.

- The values in this table have been calculated using the recommended values $\gamma_{M0} = 1{,}0$ and $\gamma_{M2} = 1{,}25$ as given in Parts EN 1993-1-1 and EN 1993-1-8 of Eurocode. **National requirements must be checked from the National Annex of the relevant country.**
- The throat thickness values of this table are valid:
  - when the hollow section to be joined is subject to axial tension or compression and/or bending
  - when the weld is made around the perimeter of the hollow section to be joined
  - when using a welding consumable having its yield strength and ultimate tensile strength at least equivalent minimum values as those of the weaker part to be joined.

APPENDIX 4. GENERAL ARRANGEMENT AND PRODUCTION DRAWINGS

I.	**GENERAL ARRANGEMENT DRAWING**	
II.	**ASSEMBLY DRAWINGS**	
	Contents	*Drawing number*
1	Column assembly	C/38
2	Beam assembly	P/1
3	End beam assembly	B/68
4	Wall truss assembly	C/1
5	Roof truss assembly	TR/1
6	Diagonal bracing assembly	WB/15
III.	**PART DRAWINGS**	
	Contents	*Drawing number*
1	End plate	PL/160
2	Gusset plate	PL/129
3	End plate	BP/5
4	Truss web tube	D/41
5	Beam hollow section tube	P/40
6	Diagonal bracing tube	P/18

ELEMENTTILUETTELO				
PIIRUSTUS	NIMI	MITAT [HxBxL]	AP. KORKO	PAINO [t]

YLEISTIEDOT

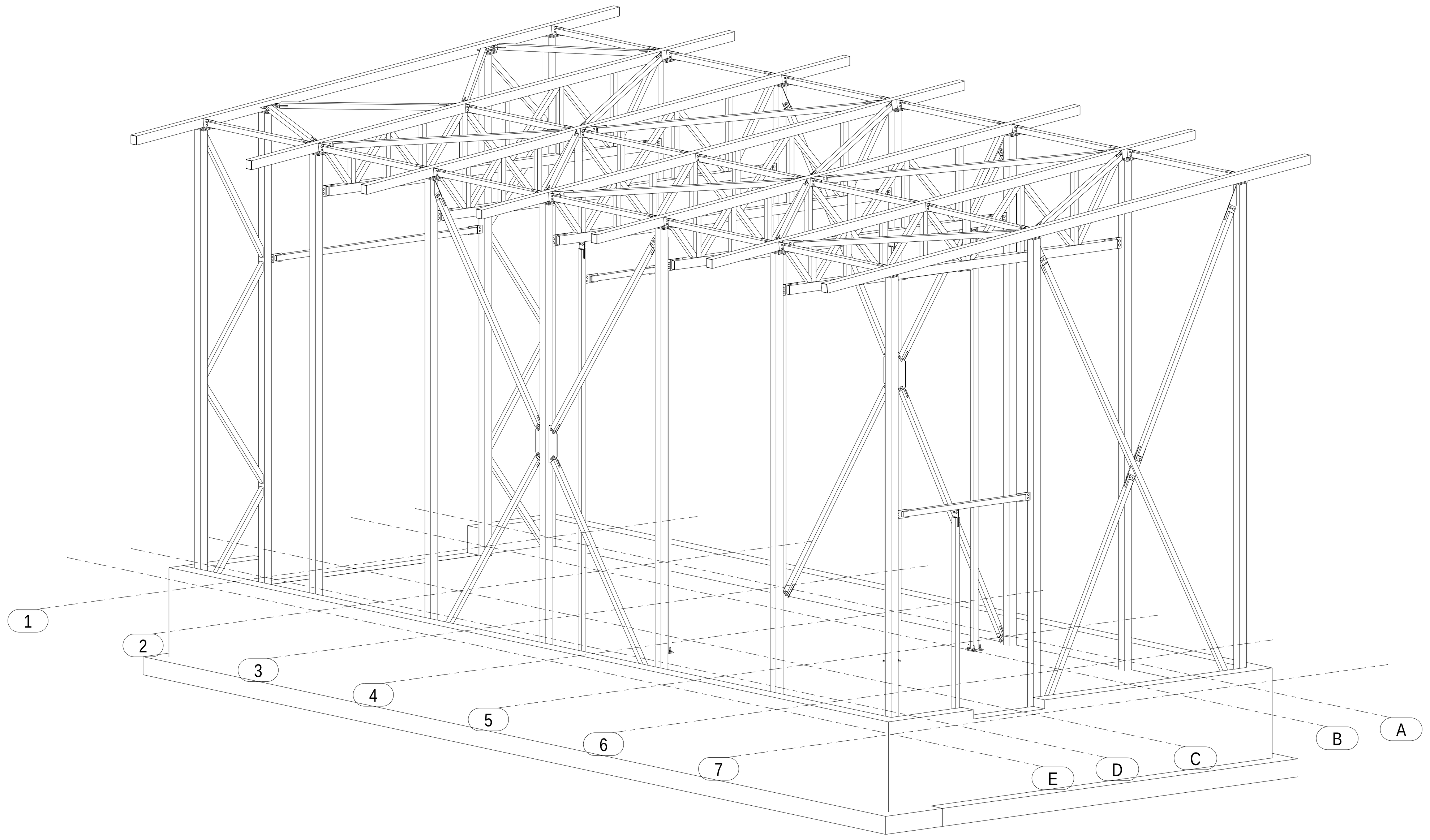
TERÄSTIEDOT

BETONITIEDOT

KUORMATIEDOT

LIITTYVÄT PIIRUSTUKSET

3d



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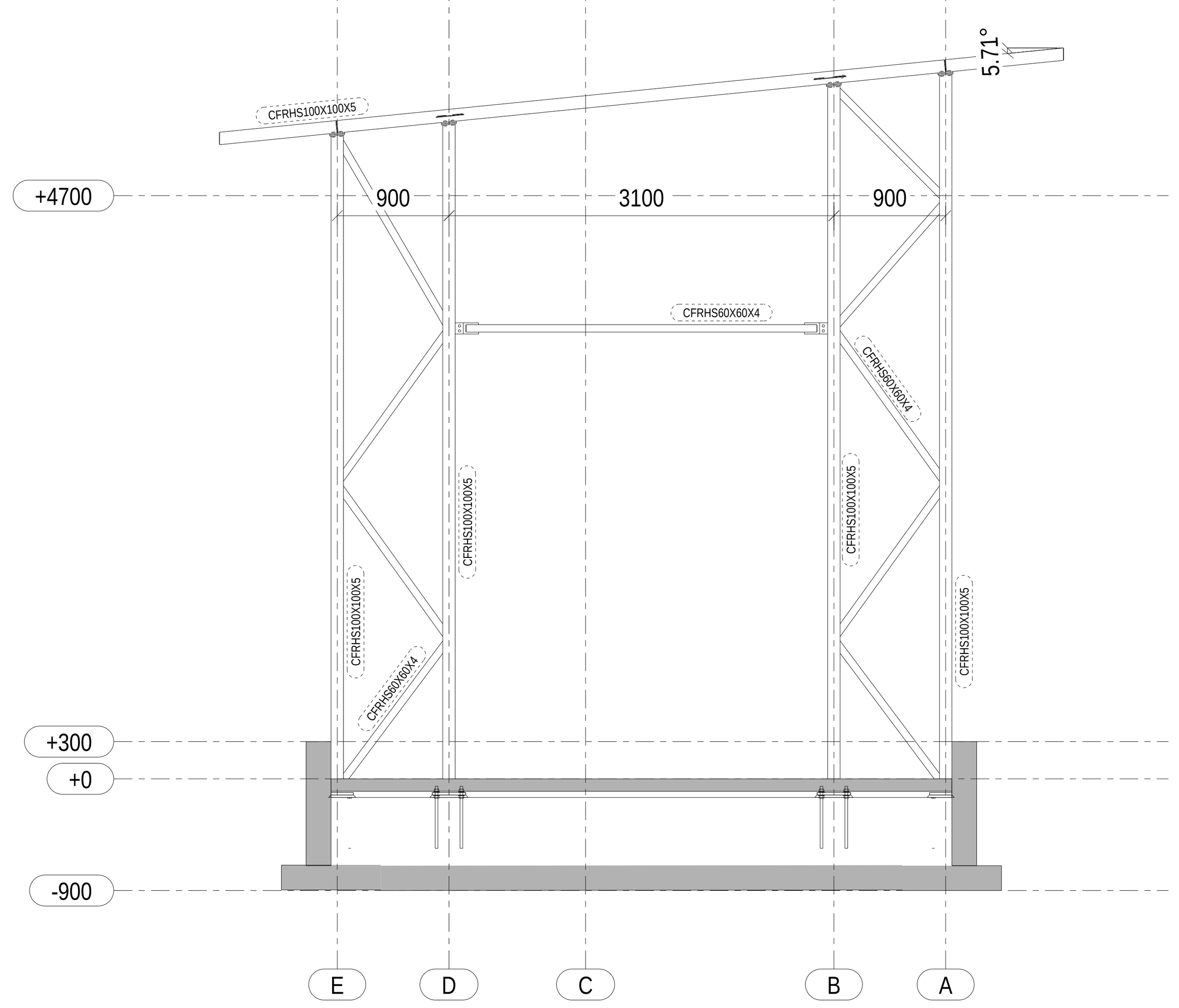

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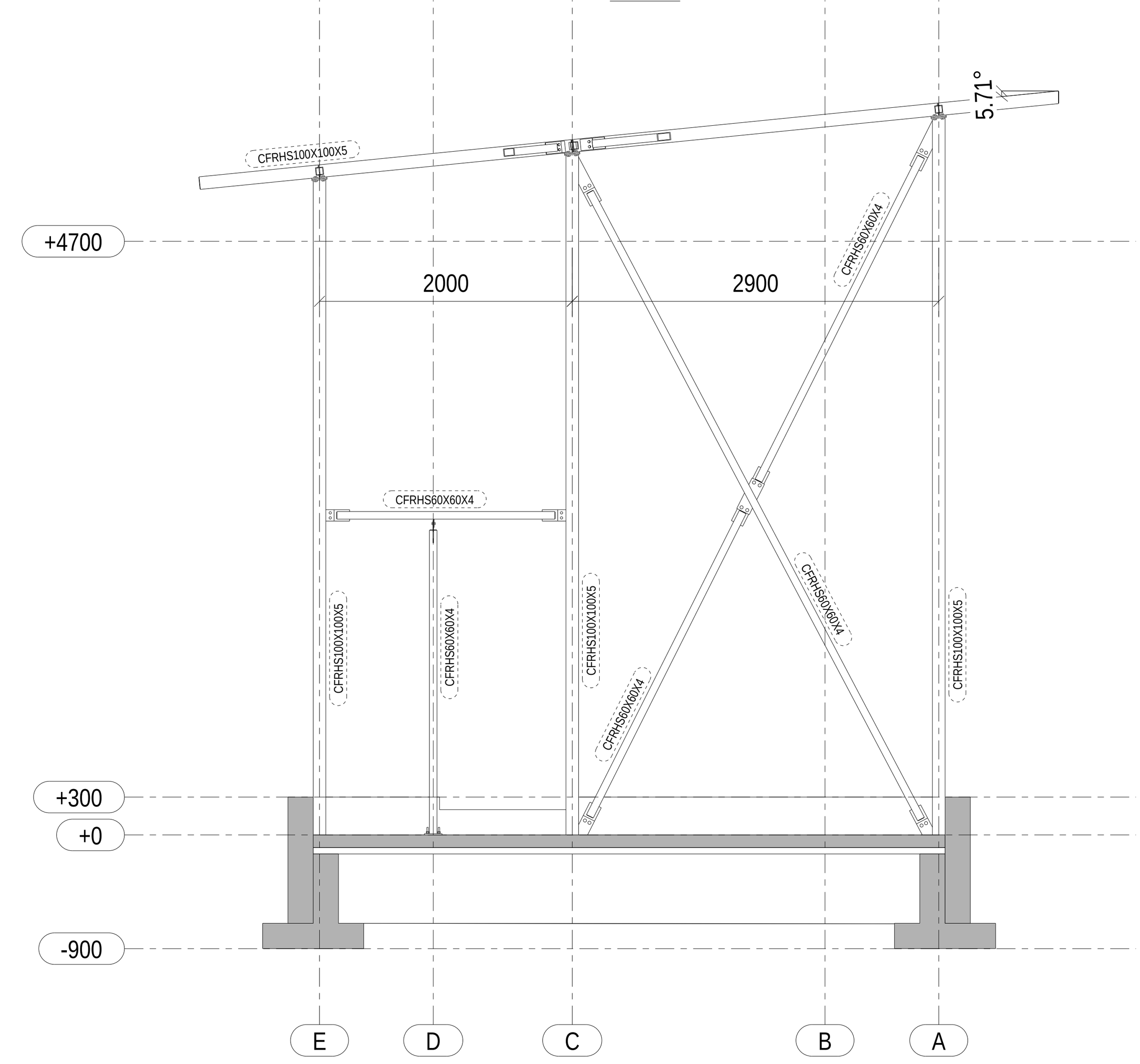
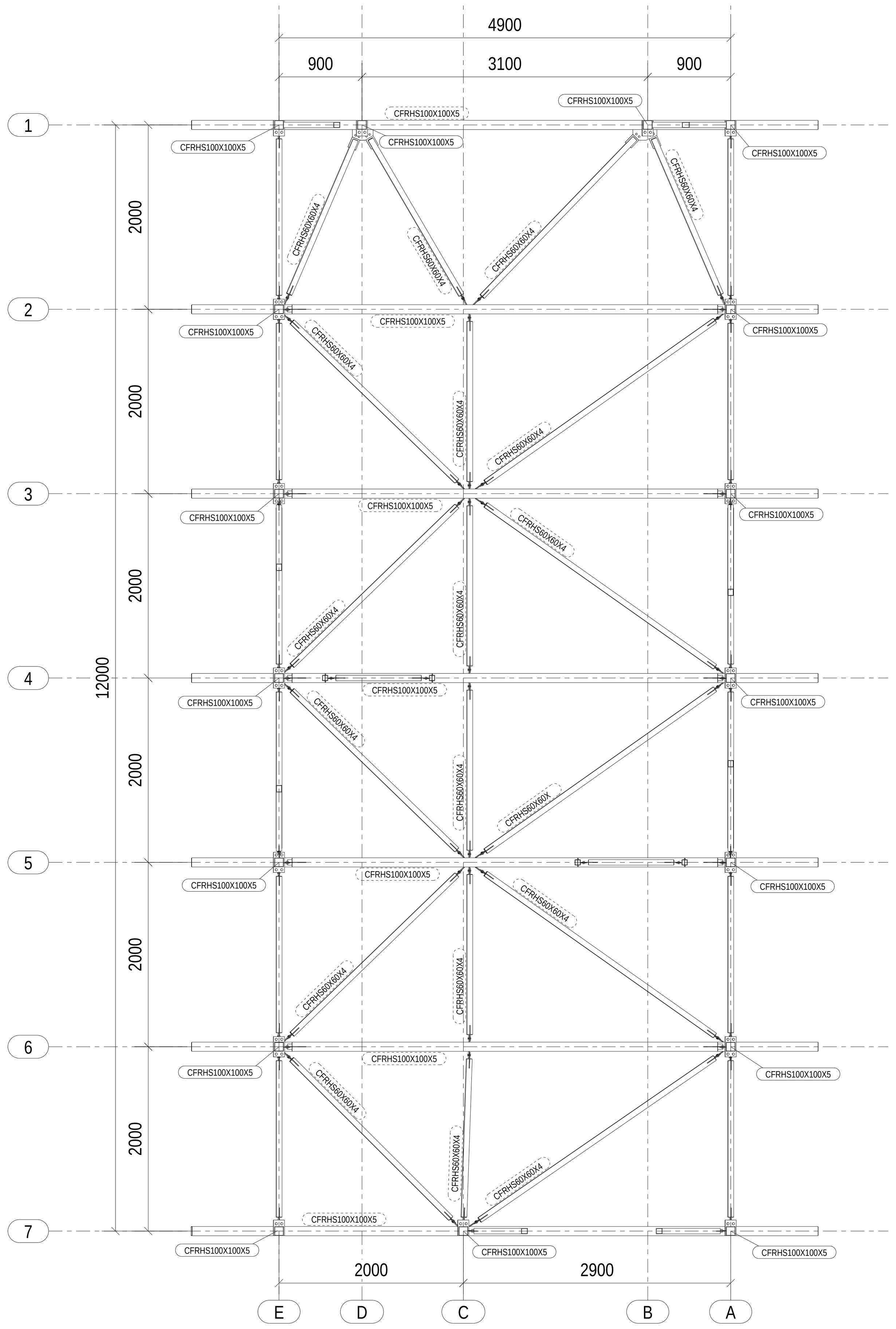


PLAN +0



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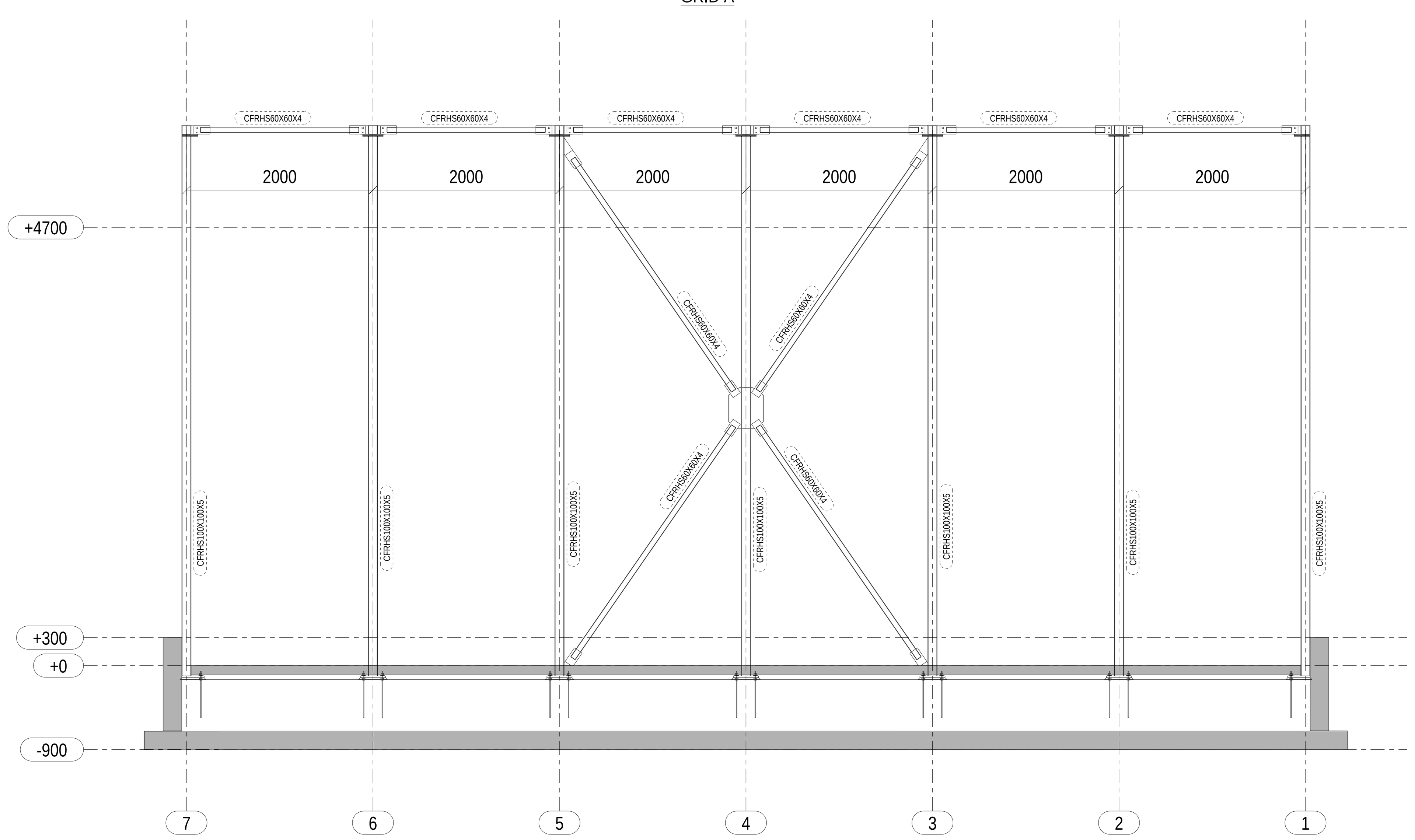

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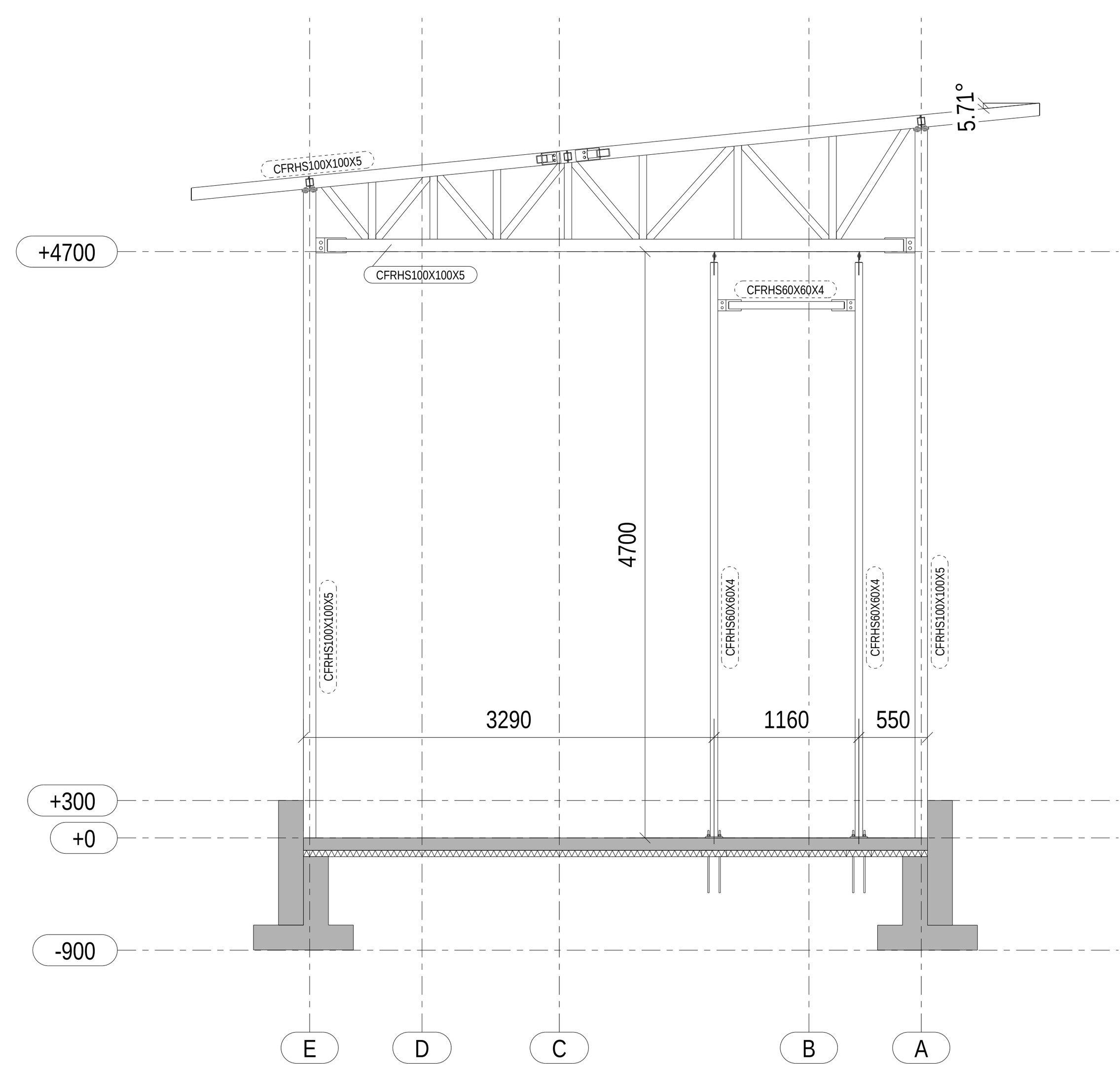

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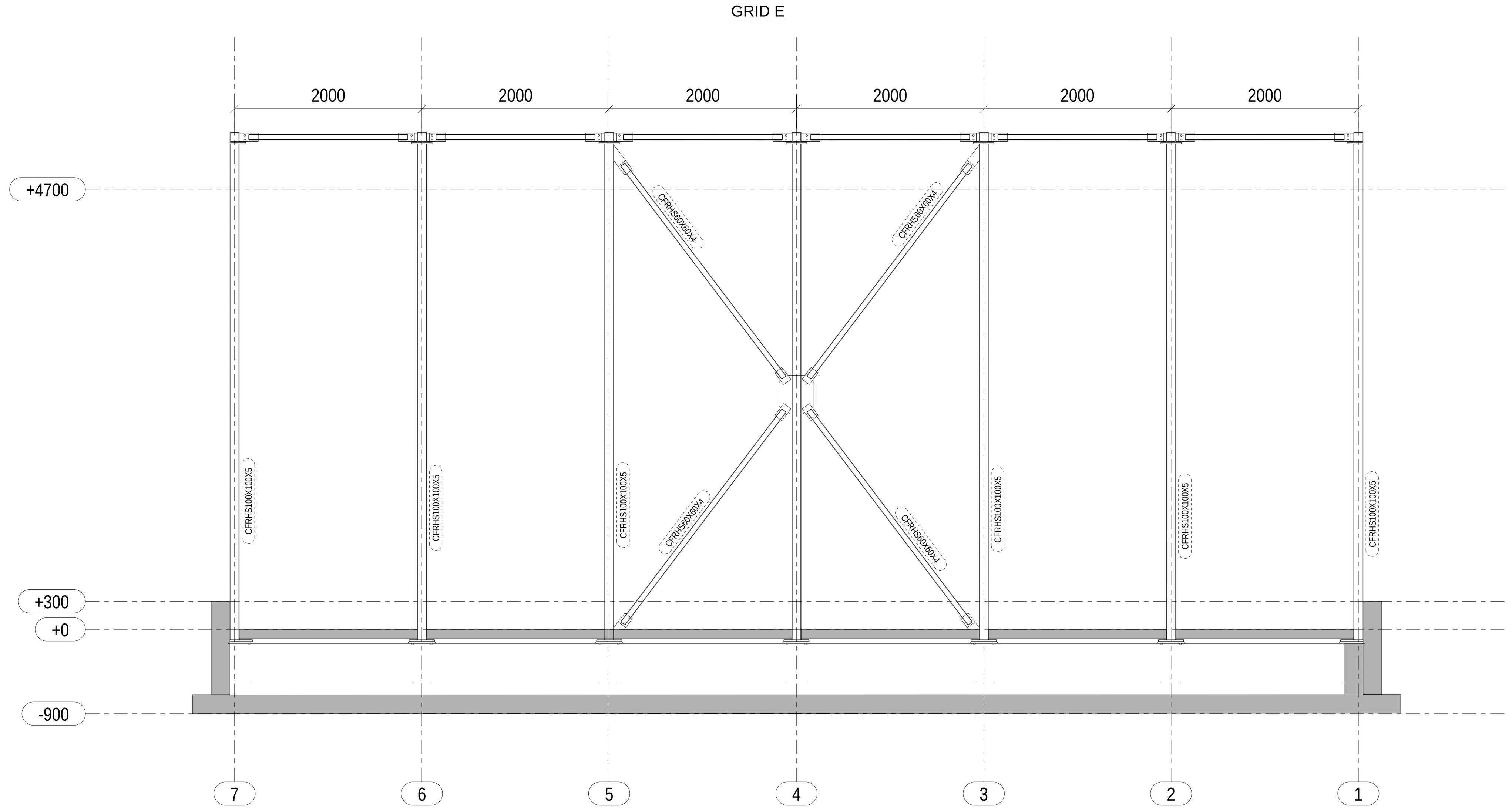


Muutos	Päiväys	Suunnittelija	Selitys

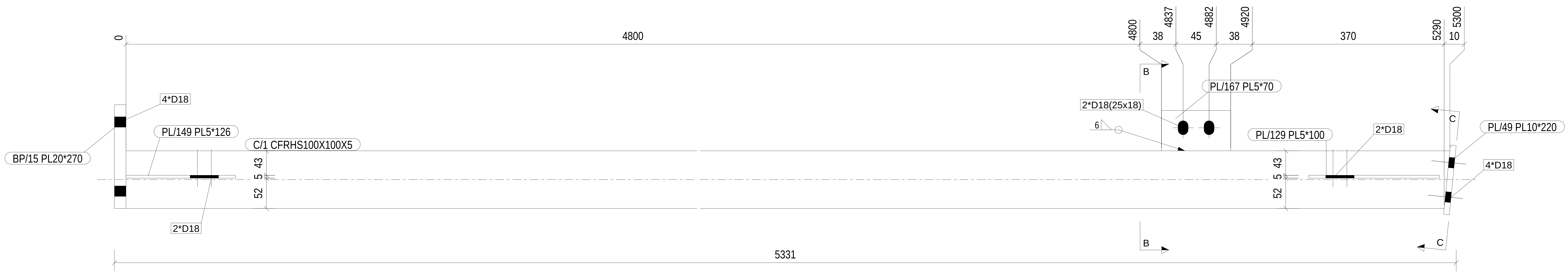
K.osa/Kylä	Kortteli/Tila	Tontti/Rn:o	Viranomaisten merkintöjä varten
Villinnotko	872	12	
Rakennustoimenpide Käsittelytila			Piirustuslaji Pääpiirustus
Suunnittelija Hämeen ammattikorkeakoulu Cristina Tirteu			Asiakkaan nimi ja osoite Kuittilan Erämiehet ry Antti Markkula puh. +358 40 8687 534 antti.markkula@ssab.com

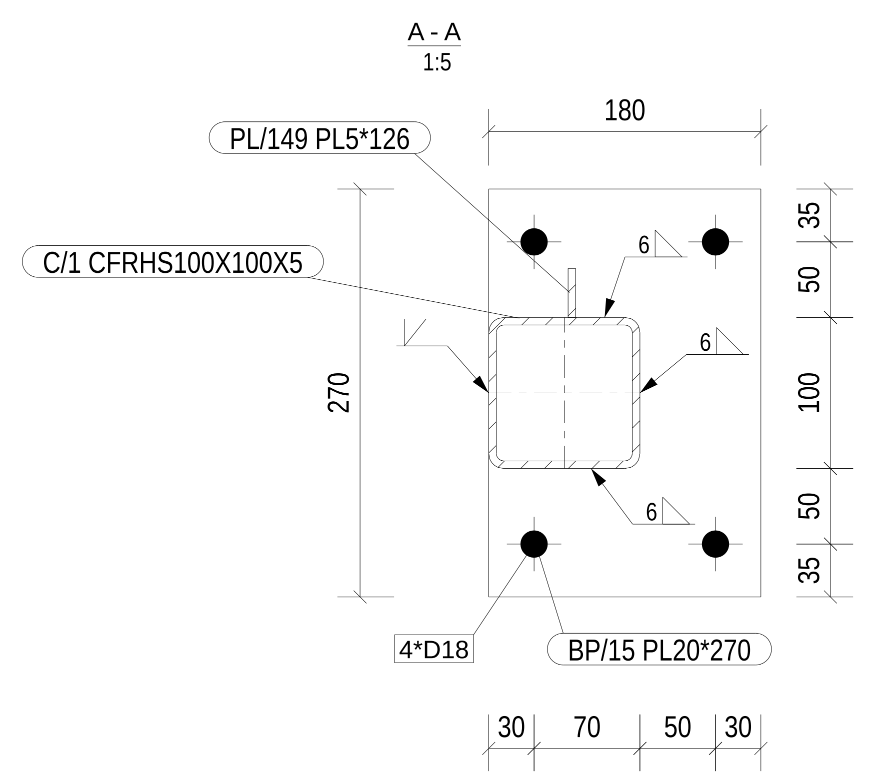
	Nimi	Mittakaava: 1:50	[2]
Piirtäjä	Nikita Drochkov	Päiväys: 26.04.2019	
Tarkistanut			

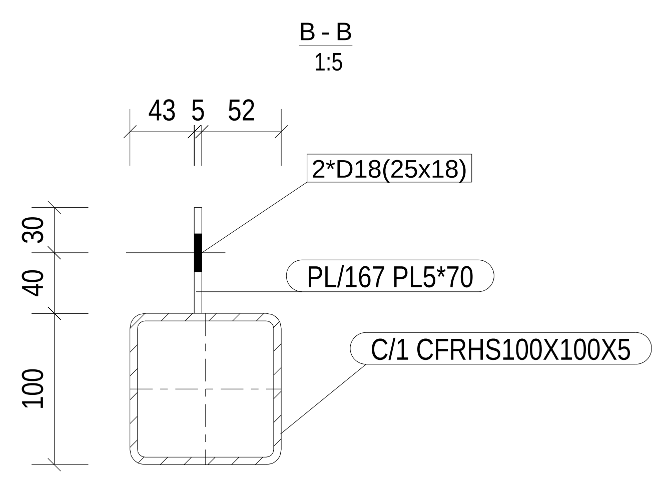
Tekla structures

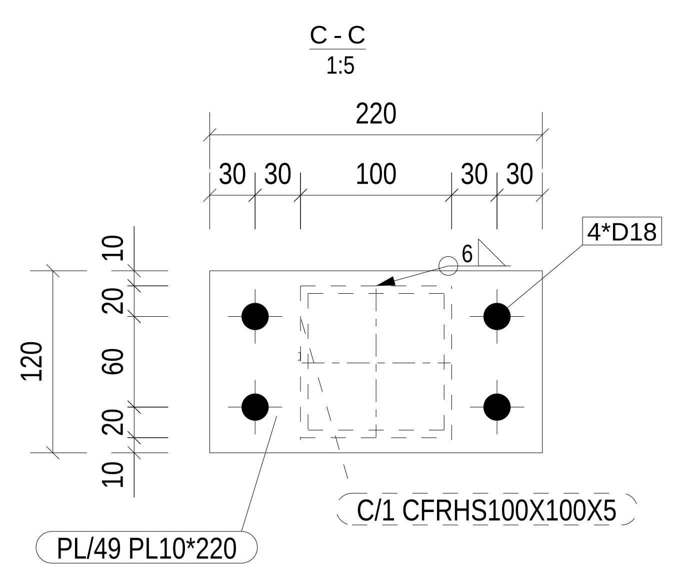
OSALUETTELO KOKOONPANOLLE C/38, JOTA VALMISTETAAN 1 KAPPALETTA						
OSA	PROFIILI	MATERIAALI	PITUUS [mm]	ALA [m2]	PAINO [kg]	LKM
BP/15	PL20*270	S355J2	180	0.1	7.6	1
C/1	CFRHS100X100X5	S355J2H	5300	2.0	79.0	1
PL/49	PL10*220	S355J2	120	0.1	2.1	1
PL/129	PL5*100	S355J2	209	0.0	0.6	1
PL/149	PL5*126	S355J2	190	0.0	0.5	1
PL/167	PL5*70	S355J2	120	0.0	0.3	1
			YHTEENSÄ:	2.3	90.1	

KOKOONPANON KIINNIKELUETTELO								
NIMIKE	HALK	KOKO	STANDARDI	LUJUUS	MATER./PINTA	VÄRI	kg/YHT.	LKM
					YHTEENSÄ:		0.0	









Muutos	Päiväys	Suunnittelija	Selitys

K.osa/Kylä	Kortteli/Tila	Tontti/Rn:o	Viranomaisten merkintöjä varten
Villinnotko	872	12	
Rakennustoimenpide Käsittelytila			Piirustuslaji Pääpiirustus
Suunnittelija Hämeen ammattikorkeakoulu Cristina Tirteu			Asiakkaan nimi ja osoite Kuittilan Erämiehet ry Antti Markkula puh. +358 40 8687 534 antti.markkula@ssab.com

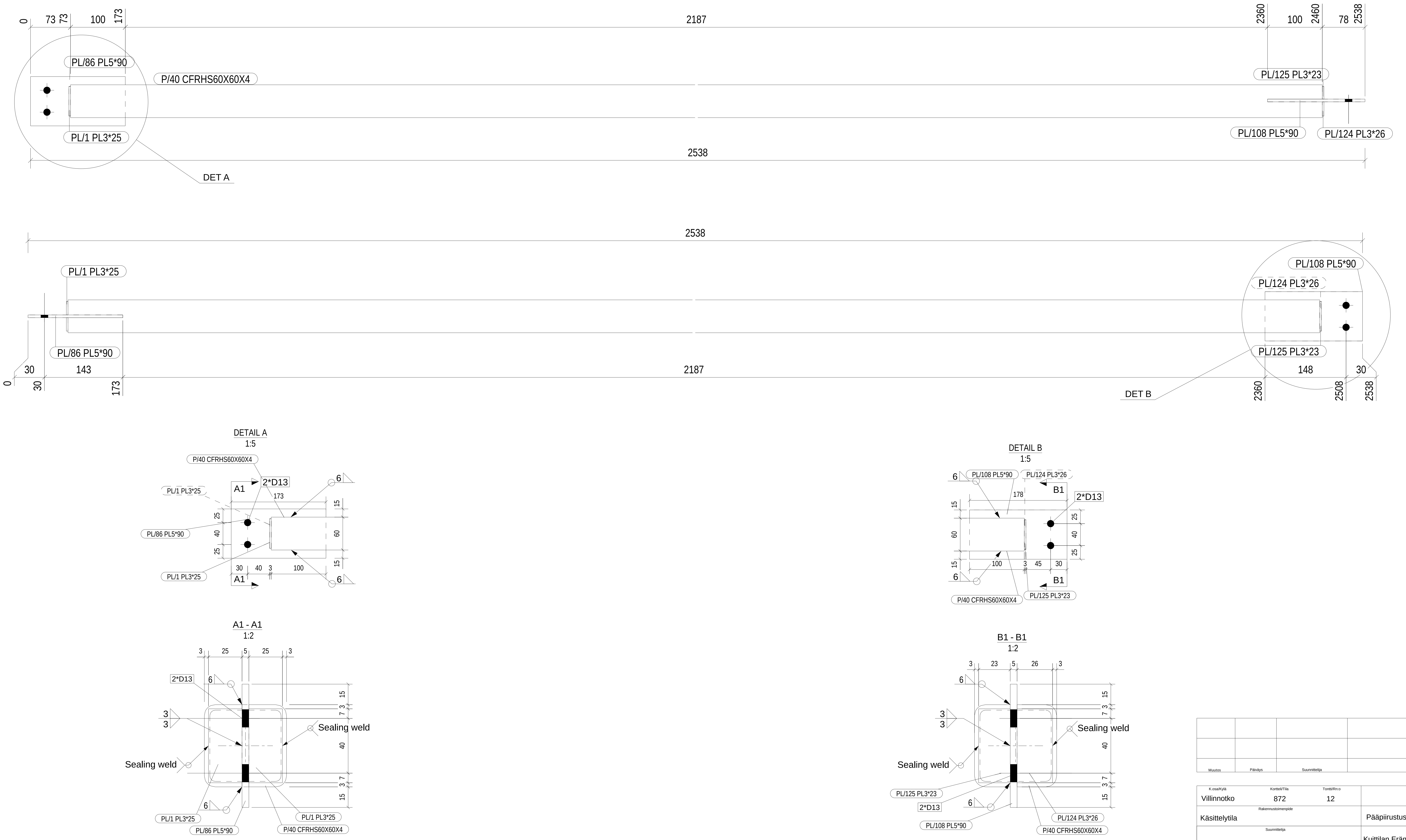
	Nimi	Mittakaava: 1:5	[C.38]	RAK
Piirtäjä	M. Chistyakov	Päiväys:		
Tarkistanut	J. Havula	10.07.2018		

VALMISTUS	HITSIT	PINTAKÄSITTELY

Tekla structures

OSALUETTELO KOKOONPANOLLE P/1, JOTA VALMISTETAAN 1 KAPPALETTA						
OSA	PROFIILI	MATERIAALI	PITUUS [mm]	ALA [m2]	PAINO [kg]	LKM
P/40	CFRHS60X60X4	S355J2H	2387	0.5	16.7	1
PL/1	PL3*25	S355J2	54	0.0	0.0	2
PL/86	PL5*90	S355J2	173	0.0	0.6	1
PL/108	PL5*90	S355J2	178	0.0	0.6	1
PL/124	PL3*26	S355J2	54	0.0	0.0	1
PL/125	PL3*23	S355J2	54	0.0	0.0	1
			YHTEENSÄ:	0.6	18.1	

KOKOONPANON KIINNIKELUETTELO								
NIMIKE	HALK	KOKO	STANDARDI	LUJUUS	MATER./PINTA	VÄRI	kg/YHT.	LKM
					YHTEENSÄ:		0.0	



Muutos	Päiväys	Suunnittelija	Selitys

K.osa/Kylä	Kortteli/Tila	Tontti/Rn:o	Viranomaisten merkintöjä varten
Villinnotko	872	12	

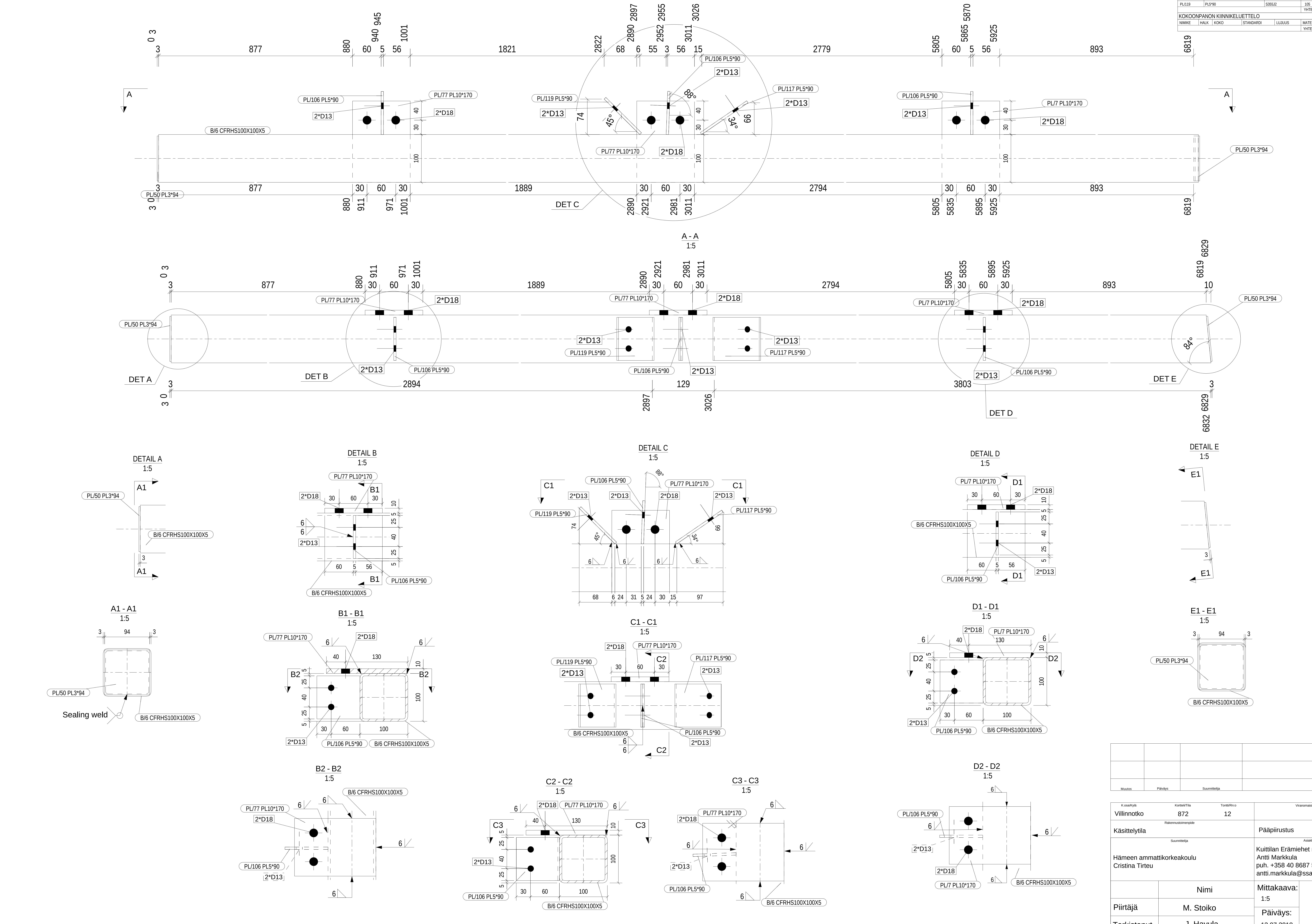
Rakennustoimenpide	Piirustuslaji
Käsittelytila	Pääpiirustus

Suunnittelija	Asiakkaan nimi ja osoite
Hämeen ammattikorkeakoulu Cristina Tirteu	Kuittilan Erämiehet ry Antti Markkula puh. +358 40 8687 534 antti.markkula@ssab.com

	Nimi	Mittakaava: 1:2 1:5	[P.1]	RAK
Piirtäjä	M. Stoiko	Päiväys: 13.07.2018		
Tarkistanut	J. Havula			

Tekla structures

VALMISTUS	HITSIT	PINTAKÄSITTELY



OSALUETTELO KOKOONPANOLLE B/68, JOTA VALMISTETAAN 1 KAPPALETTA

OSA	PROFIILI	MATERIAALI	PITUUS [mm]	ALA [m2]	PAINO [kg]	LKM
B/6	CFRHS100X100X5	S355J2H	6826	2.6	101.7	1
PL/7	PL10*170	S355J2	121	0.0	1.6	1
PL/50	PL3*94	S355J2	94	0.0	0.2	2
PL/77	PL10*170	S355J2	121	0.1	1.6	2
PL/106	PL5*90	S355J2	90	0.1	0.3	3
PL/117	PL5*90	S355J2	117	0.0	0.4	1
PL/119	PL5*90	S355J2	105	0.0	0.4	1
			YHTEENSÄ:	2.9	108.7	

KOKOONPANON KIINNIKELUETTELO

NIMIKE	HALK	KOKO	STANDARDI	LUJUUS	MATER./PINTA	VÄRI	kg/YHT.	LKM
					YHTEENSÄ:		0.0	

Muutos	Päiväys	Suunnittelija	Selitys

K.osa/Kylä	Kortteli/Tila	Tontti/Rn:o	Viranomaisten merkintöjä varten
Villinnotko	872	12	

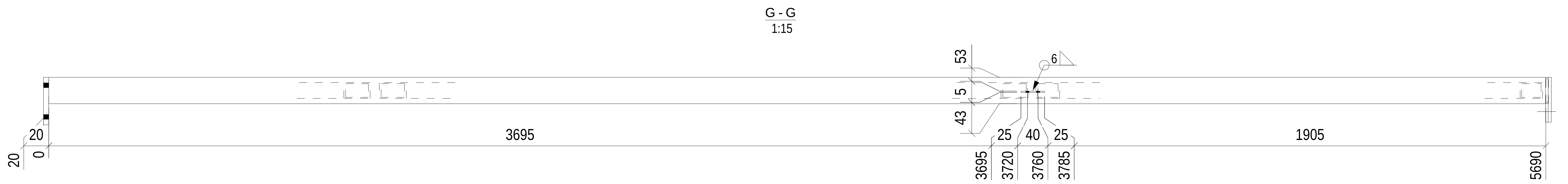
Rakennustoimenpide	Piirustuslaji
Käsittelytila	Pääpiirustus

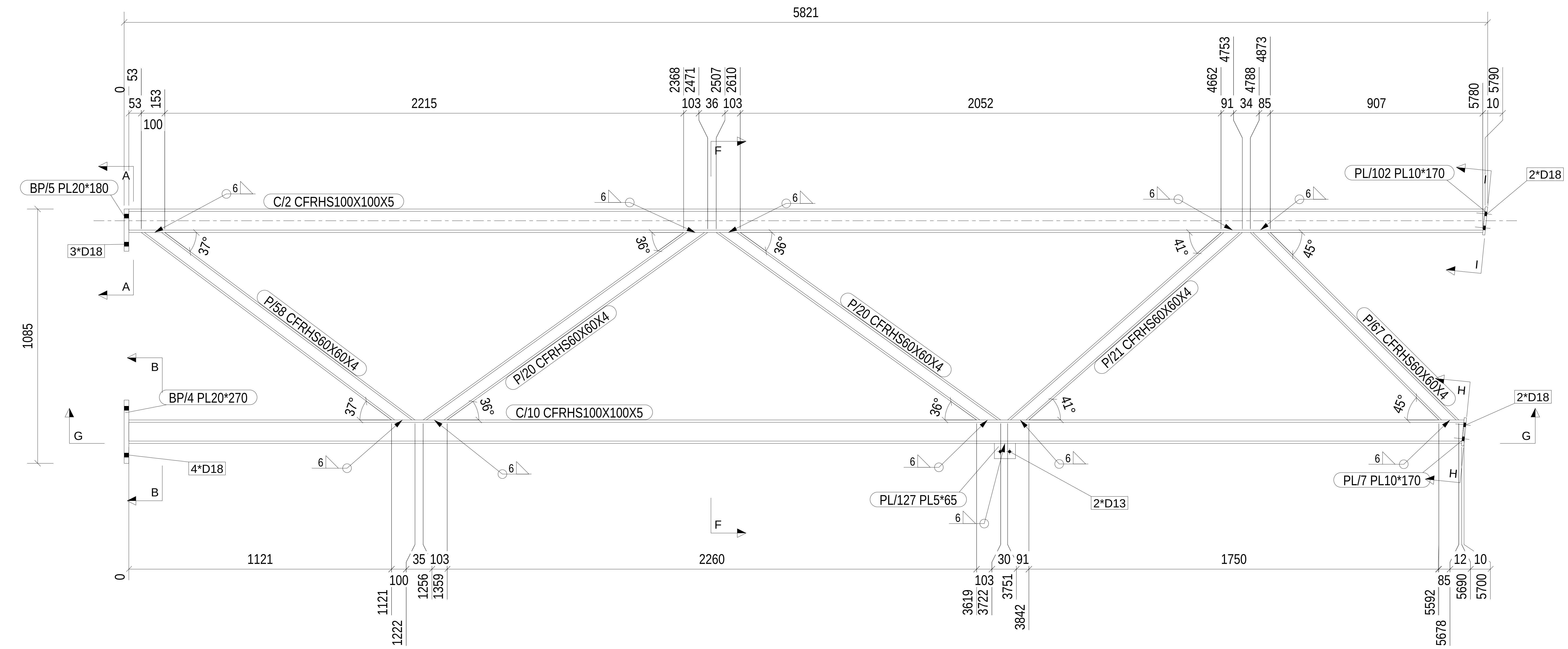
Suunnittelija	Asiakkaan nimi ja osoite
Hämeen ammattikorkeakoulu Cristina Tirteu	Kuittilan Erämiehet ry Antti Markkula puh. +358 40 8687 534 antti.markkula@ssab.com

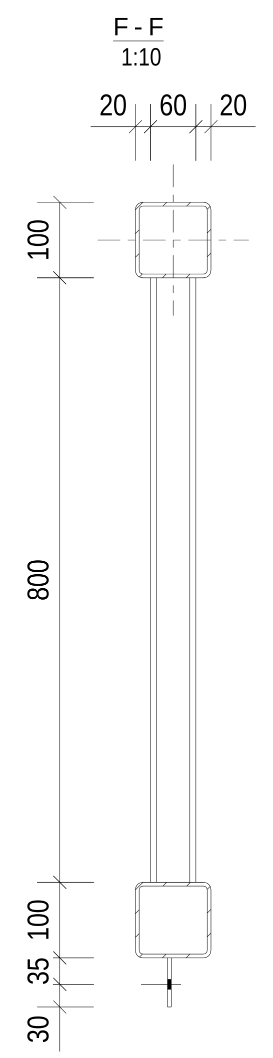
	Nimi			
Piirtäjä	M. Stoiko	Mittakaava: 1:5	[B.68]	RAK
Tarkistanut	J. Havula	Päiväys: 13.07.2018		

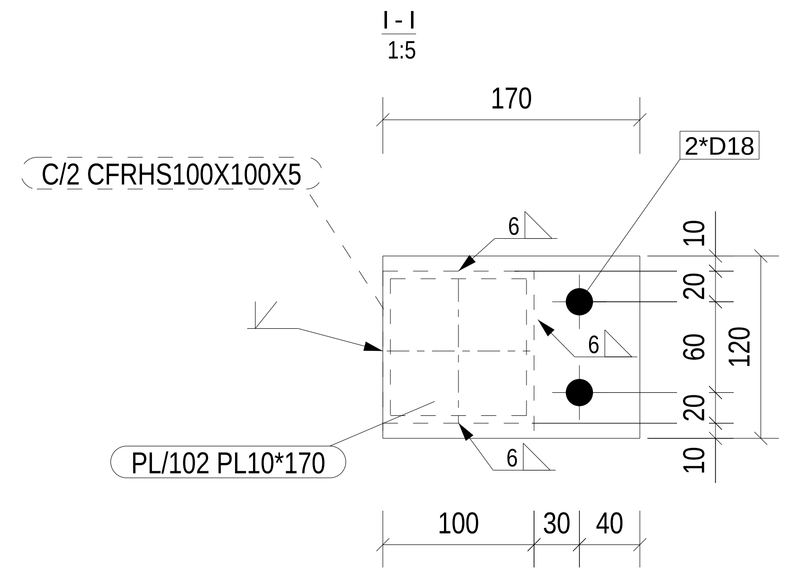
VALMISTUS	HITSIT	PINTAKÄSITTELY

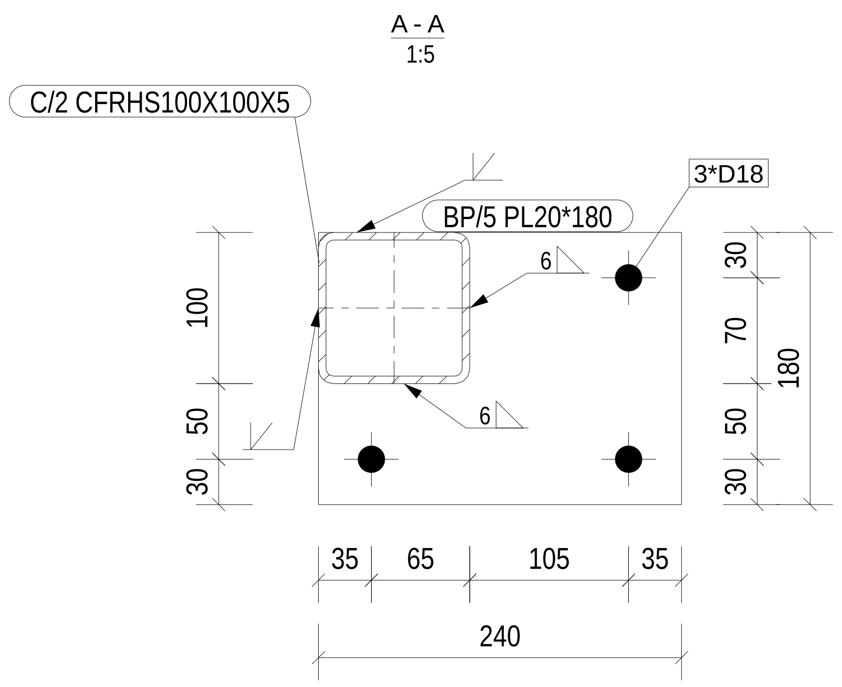
Tekla structures

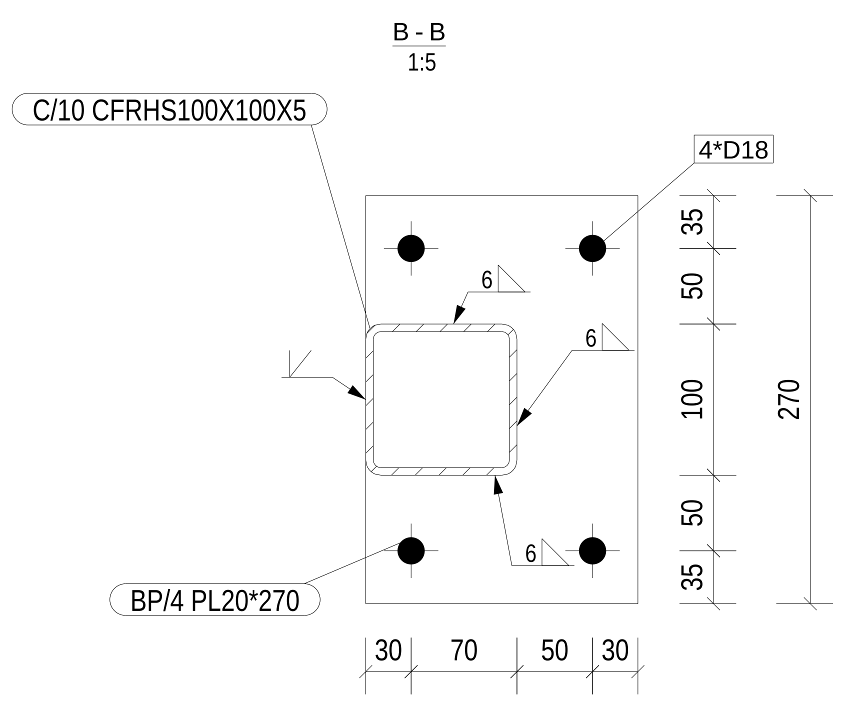


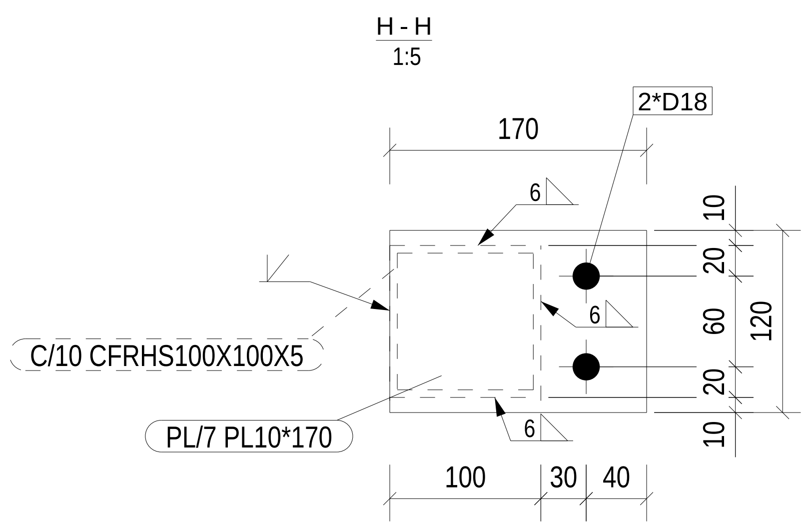












OSALUETTELO KOKOONPANOLLE C/1, JOTA VALMISTETAAN 1 KAPPALETTA						
OSA	PROFIILI	MATERIAALI	PITUUS [mm]	ALA [m2]	PAINO [kg]	LKM
BP/4	PL20*270	S235JR	180	0.1	7.6	1
BP/5	PL20*180	S235JR	240	0.1	6.8	1
C/2	CFRHS100X100X5	S355J2H	5790	2.2	86.3	1
C/10	CFRHS100X100X5	S355J2H	5700	2.2	84.9	1
P/20	CFRHS60X60X4	S355J2H	1453	0.7	9.6	2
P/21	CFRHS60X60X4	S355J2H	1281	0.3	8.5	1
P/58	CFRHS60X60X4	S355J2H	1415	0.3	9.4	1
P/67	CFRHS60X60X4	S355J2H	1195	0.3	8.0	1
PL/7	PL10*170	S355J2	120	0.0	1.6	1
PL/102	PL10*170	S355J2	120	0.0	1.6	1
PL/127	PL5*65	S355J2	90	0.0	0.2	1
			YHTEENSÄ:	6.3	234.2	

KOKOONPANON KIINNIKELUETTELO								
NIMIKE	HALK	KOKO	STANDARDI	LUJUUS	MATER./PINTA	VÄRI	kg/YHT.	LKM
					YHTEENSÄ:		0.0	

Muutos	Päiväys	Suunnittelija	Selitys

K.osa/Kylä	Kortteli/Tila	Tontti/Rn:o	Viranomaisten merkintöjä varten
Villinnotko	872	12	

Rakennustoimenpide	Piirustuslaji
Käsittelytila	Pääpiirustus

Suunnittelija	Asiakkaan nimi ja osoite
Hämeen ammattikorkeakoulu Cristina Tirteu	Kuittilan Erämiehet ry Antti Markkula puh. +358 40 8687 534 antti.markkula@ssab.com

	Nimi	Mittakaava: 1:5 1:10	[C.1]	RAK
Piirtäjä	M. Chistyakov	Päiväys:		
Tarkistanut	J. Havula	10.07.2018		

VALMISTUS	HITSIT	PINTAKÄSITTELY

OSALUETTELO KOKOONPANOLLE TR/1, JOTA VALMISTETAAN 1 KAPPALETTA						
OSA	PROFIILI	MATERIAALI	PITUUS [mm]	ALA [m2]	PAINO [kg]	LKM
D/29	CFRHS60X60X4	S355J2H	818	0.2	5.5	1
D/30	CFRHS60X60X4	S355J2H	1026	0.2	6.9	1
D/31	CFRHS60X60X4	S355J2H	1067	0.2	7.2	1
D/32	CFRHS60X60X4	S355J2H	675	0.2	4.7	1
D/33	CFRHS60X60X4	S355J2H	560	0.1	3.7	1
D/34	CFRHS60X60X4	S355J2H	458	0.1	3.2	1
D/35	CFRHS60X60X4	S355J2H	507	0.1	3.5	1
D/36	CFRHS60X60X4	S355J2H	558	0.1	3.9	1
D/37	CFRHS60X60X4	S355J2H	615	0.1	4.3	1
D/38	CFRHS60X60X4	S355J2H	751	0.2	5.3	1
D/39	CFRHS60X60X4	S355J2H	827	0.2	5.8	1
D/40	CFRHS60X60X4	S355J2H	662	0.1	4.4	1
D/41	CFRHS60X60X4	S355J2H	676	0.2	4.5	1
D/42	CFRHS60X60X4	S355J2H	794	0.2	5.3	1
D/45	CFRHS60X60X4	S355J2H	1022	0.2	6.9	1
LC/1	CFRHS100X100X5	S355J2H	4614	1.8	68.7	1
PL/49	PL10*220	S355J2	120	0.1	2.1	1
PL/50	PL3*94	S355J2	94	0.0	0.2	2
PL/82	PL10*220	S355J2	120	0.1	2.1	1
PL/106	PL5*90	S355J2	90	0.1	0.3	4
PL/113	PL5*90	S355J2	112	0.0	0.4	1
PL/114	PL5*90	S355J2	107	0.0	0.4	2
PL/115	PL5*90	S355J2	115	0.0	0.4	2
PL/116	PL5*70	S355J2	90	0.0	0.2	1
PL/118	PL5*90	S355J2	103	0.0	0.4	1
PL/159	PL5*120	S355J2	233	0.1	1.1	2
PL/160	PL3*48	S355J2	100	0.0	0.1	4
UC/2	CFRHS100X100X5	S355J2H	6838	2.6	101.8	1
			YHTEENSÄ:	7.4	256.6	

KOKOONPANON KIINNIKELUETTELO								
NIMIKE	HALK	KOKO	STANDARDI	LUJUUS	MATER./PINTA	VÄRI	kg/YHT.	LKM
					YHTEENSÄ:		0.0	

Muutos	Päiväys	Suunnittelija	Selitys

K.osa/Kylä	Kortteli/Tila	Tontti/Rn:o	Viranomaisten merkintöjä varten
Villinnotko	872	12	

Rakennustoimenpide	Piirustuslaji
Käsittelytila	Pääpiirustus

Suunnittelija	Asiakkaan nimi ja osoite
Hämeen ammattikorkeakoulu Cristina Tirteu	Kuittilan Erämiehet ry Antti Markkula puh. +358 40 8687 534 antti.markkula@ssab.com

	Nimi	Mittakaava: 1:5 1:10 1:8	[TR.1]	RAK
Piirtäjä	M. Stoiko	Päiväys:		
Tarkistanut	J. Havula	19.07.2018		

Tekla structures

VALMISTUS	HITSIT	PINTAKÄSITTELY

OSALUETTELO KOKOONPANOLLE WB/15, JOTA VALMISTETAAN 1 KAPPALETTA						
OSA	PROFIILI	MATERIAALI	PITUUS [mm]	ALA [m2]	PAINO [kg]	LKM
P/18	CFRHS60X60X4	S355J2H	5611	1.3	39.3	1
PL/2	PL3*25	S355J2	54	0.0	0.0	1
PL/3	PL3*25	S355J2	54	0.0	0.0	1
PL/92	PL5*100	S355J2	178	0.1	0.7	2
PL/97	PL3*25	S355J2	54	0.0	0.0	2
PL/143	PL10*110	S355J2	270	0.1	2.3	1
			YHTEENSÄ:	1.4	43.1	

KOKOONPANON KIINNIKELUETTELO								
NIMIKE	HALK	KOKO	STANDARDI	LUJUUS	MATER./PINTA	VÄRI	kg/YHT.	LKM
					YHTEENSÄ:		0.0	

Muutos	Päiväys	Suunnittelija	Selitys

K.osa/Kylä	Kortteli/Tila	Tontti/Rn:o	Viranomaisten merkintöjä varten
Villinnotko	872	12	
Rakennustoimenpide: Käsittelytila			Piirustuslaji: Pääpiirustus
Suunnittelija: Hämeen ammattikorkeakoulu Cristina Tirteu			Asiakkaan nimi ja osoite: Kuittilan Erämiehet ry Antti Markkula puh. +358 40 8687 534 antti.markkula@ssab.com

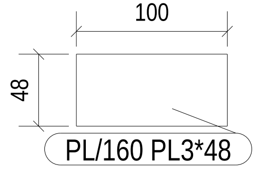
	Nimi	Mittakaava: 1:5 1:2 1:3	[WB.15]	RAK
Piirtäjä	P.Zhukov			
Tarkistanut	J. Havula	Päiväys: 19.07.2018		

Tekla structures

VALMISTUS	HITSIT	PINTAKÄSITTELY

OSA	PROFIILI	MATERIAALI	LKM	PITUUS	PAINO
PL/160	PL3*48	S355J2	4	100	0.1
				YHTEENSÄ:	0.4

FRONT



Muutos	Päiväys	Suunnittelija	Selitys

K.osa/Kylä Villinnotko	Kortteli/Tila 872	Tontti/Rn:o 12	Viranomaisten merkintöjä varten
Rakennustoimenpide Käsittelytila			Piirustuslaji Pääpiirustus
Suunnittelija Hämeen ammattikorkeakoulu Cristina Tirteu			Asiakkaan nimi ja osoite Kuittilan Erämiehet ry Antti Markkula puh. +358 40 8687 534 antti.markkula@ssab.com

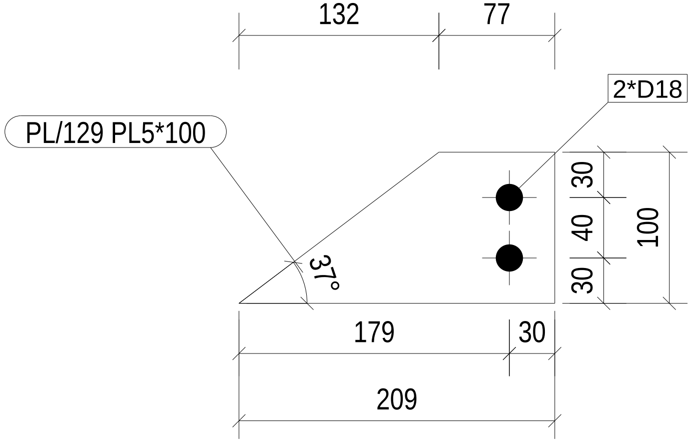
	Nimi	Mittakaava: 1:5	[PL.160]	RAK
Piirtäjä	M. Stoiko	Päiväys: 13.07.2018		
Tarkistanut	J. Havula			

KOKOONPANOSSA	LKM
TR/1	4

Tekla structures

OSA	PROFIILI	MATERIAALI	LKM	PITUUS	PAINO
PL/129	PL5*100	S355J2	1	209	0.6
				YHTEENSÄ:	0.6

FRONT



Muutos	Päiväys	Suunnittelija	Selitys

K.osa/Kylä	Kortteli/Tila	Tontti/Rn:o	Viranomaisten merkintöjä varten
Villinnotko	872	12	

Rakennustoimenpide	Piirustuslaji
Käsittelytila	Pääpiirustus

Suunnittelija	Asiakkaan nimi ja osoite
Hämeen ammattikorkeakoulu Cristina Tirteu	Kuittilan Erämiehet ry Antti Markkula puh. +358 40 8687 534 antti.markkula@ssab.com

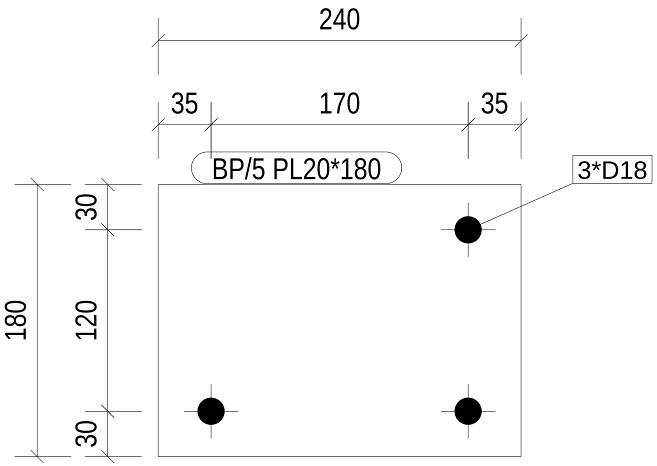
	Nimi	Mittakaava: 1:5	[PL.129]	RAK
Piirtäjä	M. Chistyakov	Päiväys:		
Tarkistanut	J. Havula	13.07.2018		

KOKOONPANOSSA	LKM
C/38	1

Tekla structures

OSA	PROFIILI	MATERIAALI	LKM	PITUUS	PAINO
BP/5	PL20*180	S235JR	1	240	6.8
				YHTEENSÄ:	6.8

FRONT



Muutos	Päiväys	Suunnittelija	Selitys

K.osa/Kylä	Kortteli/Tila	Tontti/Rn:o	Viranomaisten merkintöjä varten
Villinnotko	872	12	

Rakennustoimenpide	Piirustuslaji
Käsittelytila	Pääpiirustus

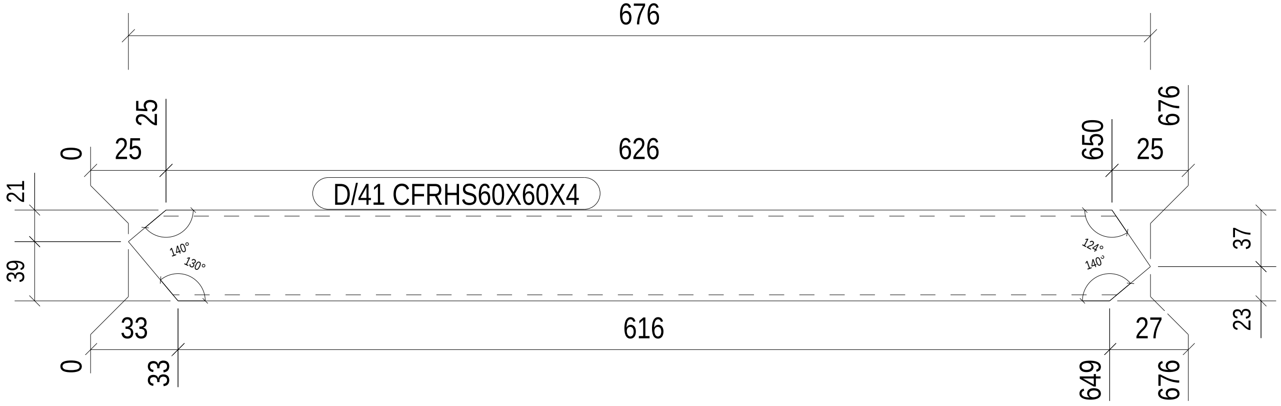
Suunnittelija	Asiakkaan nimi ja osoite
Hämeen ammattikorkeakoulu Cristina Tirteu	Kuittilan Erämiehet ry Antti Markkula puh. +358 40 8687 534 antti.markkula@ssab.com

	Nimi	Mittakaava: 1:5	[BP.5]	RAK
Piirtäjä	M. Chistyakov	Päiväys: 13.07.2018		
Tarkistanut	J. Havula			

KOKOONPANOSSA	LKM
C/1	1

Tekla structures

OSA	PROFIILI	MATERIAALI	LKM	PITUUS	PAINO
D/41	CFRHS60X60X4	S355J2H	5	676	4.5
				YHTEENSÄ:	22.4



Muutos	Päiväys	Suunnittelija	Selitys

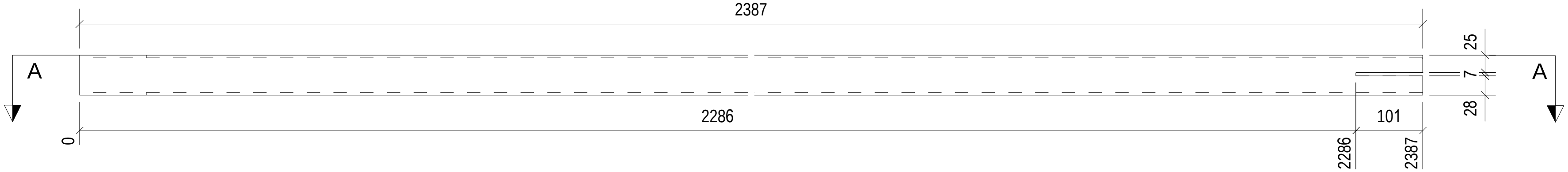
K.osa/Kylä	Kortteli/Tila	Tontti/Rn:o	Viranomaisten merkintöjä varten
Villinnotko	872	12	
Rakennustoimenpide Käsittelytila			Piirustuslaji Pääpiirustus
Suunnittelija Hämeen ammattikorkeakoulu Cristina Tirteu			Asiakkaan nimi ja osoite Kuittilan Erämiehet ry Antti Markkula puh. +358 40 8687 534 antti.markkula@ssab.com

	Nimi	Mittakaava: 1:5	[D.41]	RAK
Piirtäjä	M. Stoiko	Päiväys: 13.07.2018		
Tarkistanut	J. Havula			

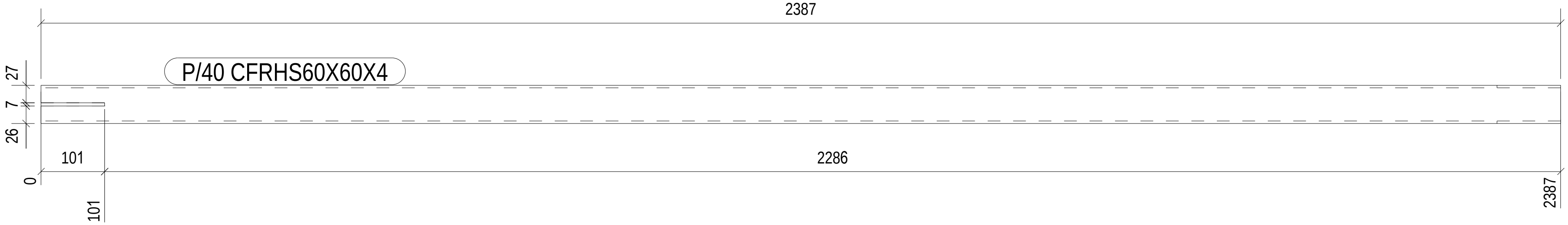
KOKOONPANOSSA	LKM
TR/1	1
TR/2	1
TR/3	1
TR/4	1
TR/5	1

Tekla structures

OSA	PROFIILI	MATERIAALI	LKM	PITUUS	PAINO
P/40	CFRHS60X60X4	S355J2H	1	2387	16.7
				YHTEENSÄ:	16.7



A - A
1:10



Muutos	Päiväys	Suunnittelija	Selitys

K.osa/Kylä	Kortteli/Tila	Tontti/Rn:o	Viranomaisten merkintöjä varten
Villinnotko	872	12	

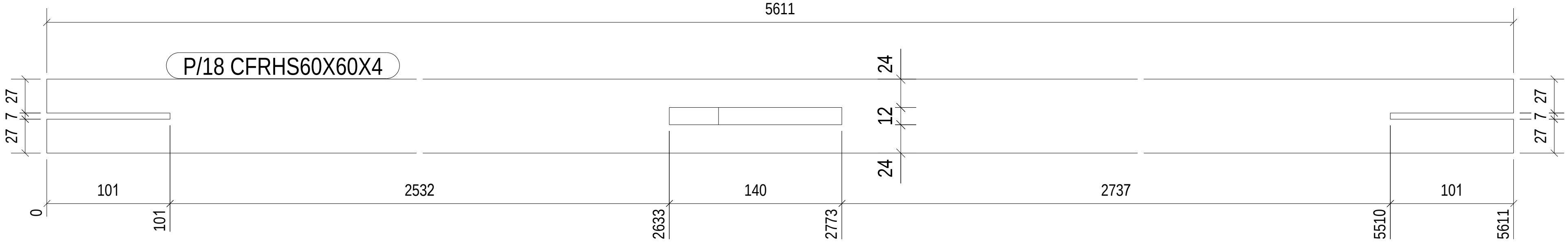
Rakennustoimenpide	Piirustuslaji
Käsittelytila	Pääpiirustus
Suunnittelija Hämeen ammattikorkeakoulu Cristina Tirteu	Asiakkaan nimi ja osoite Kuittilan Erämiehet ry Antti Markkula puh. +358 40 8687 534 antti.markkula@ssab.com

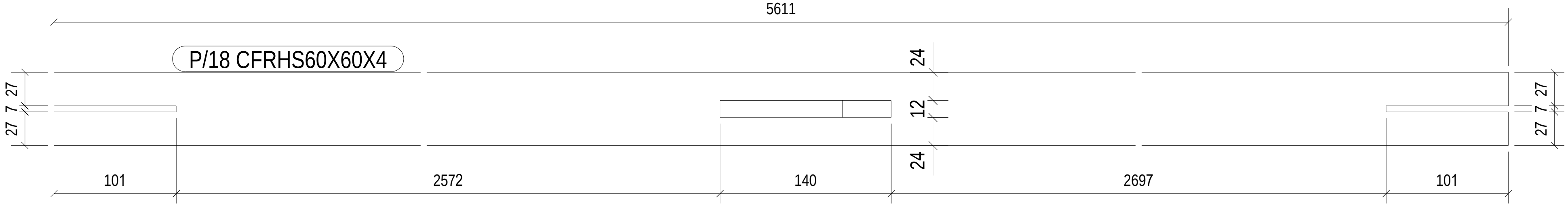
	Nimi	Mittakaava: 1:10	[P.40]	RAK
Piirtäjä	M. Stoiko	Päiväys: 13.07.2018		
Tarkistanut	J. Havula			

KOKOONPANOSSA	LKM
P/1	1

Tekla structures

OSA	PROFIILI	MATERIAALI	LKM	PITUUS	PAINO
P/18	CFRHS60X60X4	S355J2H	1	5611	39.3
				YHTEENSÄ:	39.3





Muutos	Päiväys	Suunnittelija	Selitys

K.osa/Kylä	Kortteli/Tila	Tontti/Rn:o	Viranomaisten merkintöjä varten
Villinnotko	872	12	

Rakennustoimenpide: Käsittelytila

Piirustuslaji: Pääpiirustus

Suunnittelija: Hämeen ammattikorkeakoulu, Cristina Tirteu

Asiakkaan nimi ja osoite:
Kuittilan Erämiehet ry
Antti Markkula
puh. +358 40 8687 534
antti.markkula@ssab.com

	Nimi	Mittakaava: 1:5		
Piirtäjä	P.Zhukov		[P.18]	RAK
Tarkistanut	J. Havula	Päiväys: 31.07.2018		

KOKOONPANOSSA	LKM
WB/15	1

Tekla structures