DESIGN OF A TWO-APARTMENT HOUSE

Design and knowledge assessment process

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This document shows the design of a two-apartment house with the address Kolari, Lapland. In addition to that, it emphasizes the learning process for specific design tasks. Also, this thesis shows the identification of knowledge missing during the studies and what should be the right way to follow in case the information is missing to perform a particular design task.

For this project, architectural drawings, as well as structural types, have been realized. Based on these drawings, the central part of this document has been built, which consists of the design of structural elements starting from the foundation, timber walls, and floors. The design of stud elements is done based on the guidelines provided by the relevant Eurocode. Also, the general design of stiffening walls will be included in the design of timber structures. This document shows that walls have enough resistance to withstand wind load based on general calculations. For the design of the concrete floor between the first floor and the basement is used one of the innovative products of the company Ruukki consists of the use of load-bearing sheets, which will have the role of bars, avoiding the use of the traditional method consisting of the use of formwork, which would require time and even more expense. For the foundation design, the procedures defined in the relevant Eurocodes, based on which the dimensions of the strip foundation have been determined and the reinforcement needed has been determined.

The methodology of performing these drawings and designing structures is based on Eurocodes and the standards set out in the National Annex of Finland. Engineering programs such as AutoCAD and Mathcad have been used to relate them.

Keywords Two-apartment house, design, timber, concrete.
Pages 117 and appendices 91 pages

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1 Introduction

Over time, humanity seeks new opportunities that offer better conditions for the design of buildings based on the characteristics of places. Therefore, people in the engineering field have developed different methods, products, and solutions for a structure designed for different uses. To reach the design stage, it has become possible for the necessary drawings related to the architecture and structural types to be created following Finnish standards for this project. The drawings are performed in constant consultation with the client to achieve the required result. With the help of engineering programs like AutoCAD, it has become possible to generate these drawings.

Once all the necessary drawings have been done, this document shows the design of wooden wall elements, the design of the stiffening wall on an extensive scale, and the design of the floors by implementing solutions offered by the company Ruukki and the design of the strip foundation. The determination of all loads going to the structure is done following the relevant Eurocodes and using the National Annex of Finland where needed. These elements are designed for the Ultimate Limit State, and the objective of this document is to show that the above structures mentioned for the design manage to perform correctly. This document will show the design procedures of various wood and concrete elements.

Wooden structures have been used for many years, and over time with the evolution of technology, more and more are being used. Like any other construction material, wood has its specifics provided in the relevant Eurocode. On the other hand, the same applies to concrete structures for wooden houses that use the foundation and floor design.

In conclusion, the main objective of this document is practical-based, which enables the acquisition of knowledge regarding the design of structures that will be honored in reality for this project. This provides the opportunity to understand the design process better, starting from architectural drawings and structural types, and following the essential part of this document: the design of structures. The following chapter will provide a better understanding of the methodology implemented in this thesis to achieve the objectives.

2 Methodology

The methods included in this thesis constitute the design of a two-apartment house in Finland. The structures that will be designed in this project will consist of timber structures and concrete structures, for which the design of the walls and the foundation will be done, respectively. The basic methodology for designing these structures is based on the rules set out in the Eurocodes for the various structures. Although the design of the structures will be done based on the relevant Eurocodes, the National Annex of Finland must be used when required. This is important as some factors vary from country to country, and for this reason, the structures must be designed according to the conditions of the respective countries. The way the structures or the building behaves like one for projects of this scale is known. Therefore, approximate calculations can be used to design projects of this scale, making it possible to bypass precise measures widely used in complex projects. The different loads that come to the structures are calculated following the Eurocodes.

Projects are classified into different scales ranging from the simplest to the most complicated. This would allow for a particular assumption, making the calculation method easier for the designer. Contrary to this, the exact calculation method considers all needed details for a complex project.

In this project, the design of structures is based on the given dimensions of a structure, such as a stud element, which will be checked for stability phenomena. The structure will be built if these dimensions of the wall stud pass the necessary verifications according to the Eurocode and the National Annex of Finland. This phase, in which the selection of the dimensions of the structural elements was made, was realized through meetings between the client and the engineer.

The other way this thesis finds application is to design structures based on prior assumptions. In this type of design, the engineer selects the preliminary dimensions of the structure to be designed and checks based on this data. Suppose the dimensions of the structure do not exceed the appropriate controls specified in Eurocode. The engineer provides the necessary solutions, such as changing the sizes, material, and others, to design a structure that will exceed the specified controls.

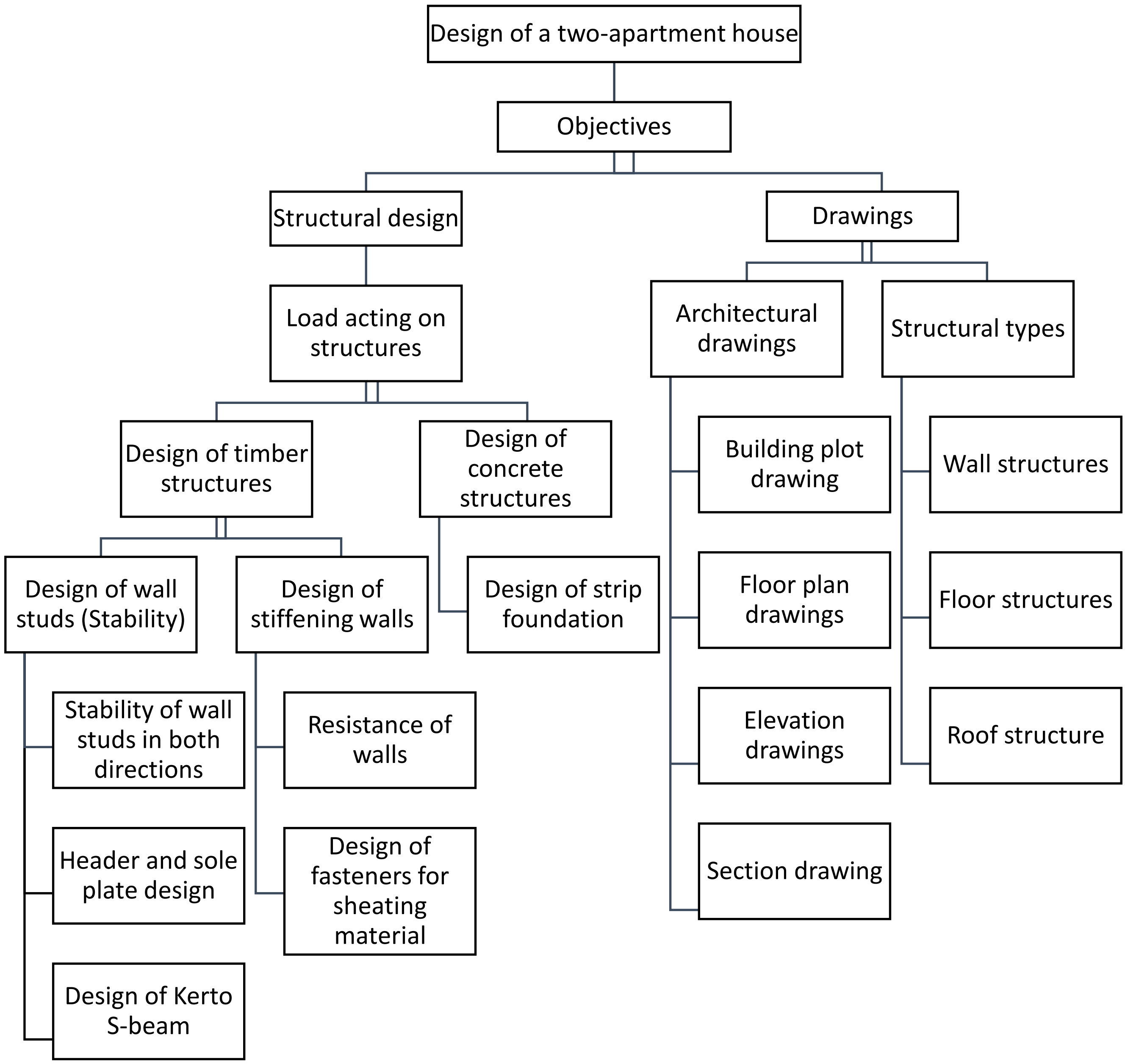
The dimensions suggested by the engineer and the client are used for architectural drawings and structural types. For structures, such as walls or floor slabs, various resources have been used by local companies in Finland, which offer the possibility of creating a structure according to the client's wish. This document finds practical use that can be used to understand the process of designing different structures for a two-apartment house according to Finnish standards.

The parts that were performed in collaboration with Igert Loka include the calculation of snow, wind, and the strip foundation's design because he will have another design that consists of the use of columns, but the calculation of these elements is somehow the same, although the design is different.

2.1 Aims and objectives

This project will be realized, which increases the value of the objectives and goals of this project. These objectives have made it possible to make the right decisions about the project during the various phases. Also, the objectives have a significant value as they manage to clarify and assist in the relationship between the client and the engineer. The main aim of this document is to design a two-apartment house, including all the necessary drawings and the design of structures for a safer environment for the client. In order to achieve the main goal in different phases of the project, specific objectives have been set where through their progress, the final goal will be achieved. Different phases have been followed for this project to achieve its objectives (Figure 1).

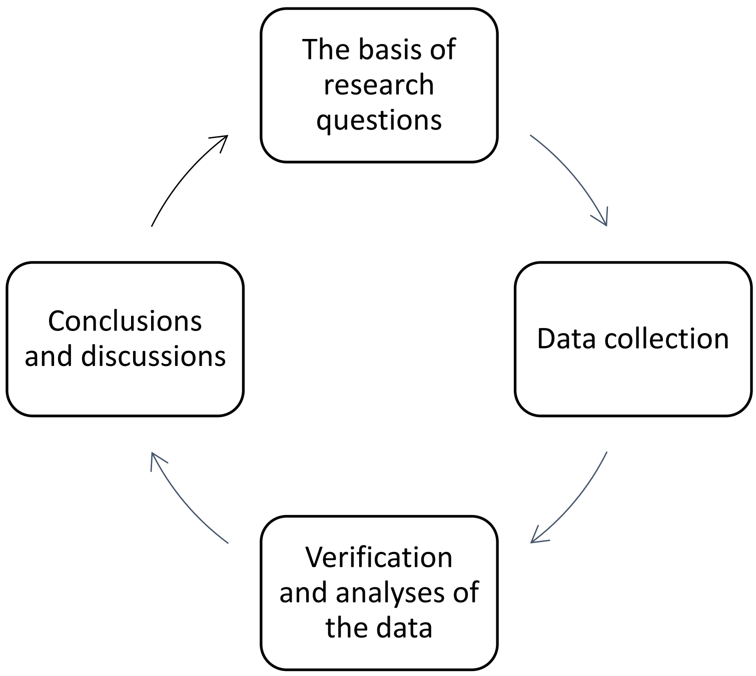
Figure 1. Aims and objectives of the project.



2.2 Research view of the thesis

Research offers the possibility of analyzing various problems based on scientific methods. Although the document is practice-oriented, a research structure will raise problems and various questions. The research workflow chart is shown below (Figure 2).

Figure 2. Research plan.



2.2.1 Basis of research questions

Even though this document is more practice-oriented, this document highlights various problems that require specific solutions based on the knowledge gained during the study years. Research questions are fundamental as they form the basis of a particular problem on which the solution to a particular problem will be based. The research questions that find implementation and answers in this document are presented below (Figure 3).

Figure 3. Research orientation.

Research question	How should the knowledge gained during academic studies be used to design a project?
	What are the phases through which the project is developed and certain result is achieved?
	In case of lack of knowledge for designing a structure what should be the paths to be followed?

2.2.2 Data collection

Gathering information is very important as it is the basis for any research. The collection of data for this thesis considers practical information to verify various elements and theoretical information, a detailed explanation of phenomena in different structures.

The information gathered comes from study materials that have been obtained during the academic years, but for some elements explained in the thesis, individual research was needed to design a particular structure. This required an individual learning process to achieve the objectives set at the beginning of the project.

2.2.3 Verification and analyses of the data

Once the objectives have been defined, and a suitable basis has been found to obtain the information, this information must be processed in the right way to find application in practical solutions. Verification of structures or drawings should be based on guidelines defined in Eurocode and RATU cards. In terms of drawings, the rules are straightforward and must be followed meticulously to achieve the desired product. While for the calculation of structures, the verifications are defined in Eurocode, one should think about solutions that offer a sustainable solution when difficulties are encountered. If the verifications are not fulfilled, various analyses will be made to achieve the objective.

2.3 Limitations

This document shows the design of a two-apartment house for its main elements. Although the structural part and the necessary drawings for the building permit have been made, in this document, the limitations include:

- Drawings of heating, ventilation, and air conditioning plans, electrical plans, and sewerage system. As no knowledge was gained during the studies regarding these drawings, the design process was affected to a certain level, and these drawings were not provided.
- Information about the truss element. There was no exact information about the type of the truss element when calculating the structural element. Therefore, an assumption was made for the thickness of the truss member to continue the design procedure. It is worth mentioning that the thickness was taken conservatively.
- Lack of English material to create the drawings. The language has slightly affected the drawing delivery as the codes used for drawings in Finland were available only in the Finnish language, restricting a more precise result.
- Lack of available data. During the academic years, no information was given about the design of the strip foundation and the design of the screw for the stiffening walls, and at the beginning phase of the project, this created a delay as research work needed to be done to find the correct information. This has been achieved through research and various sources based on books and the internet to gain enough knowledge and provide solutions.

Note! The project has a column for which the design is covered in the same project, but the design includes columns and beams. The critical column was designed from the model carried out by Igert Loka. The column dimension is set to be 200x200, and the timber material is C24 (Loka, 2022).

2.4 Implementation of engineering software for the design

Various programs are used to design different projects, create the necessary drawings, or check the integrity of structures from the point of view of stability. This project will include architectural drawings, drawings of structural types, and floor plans, which will be created based on AutoCAD software. Furthermore, this software can demonstrate different figures related to the elements designed or show how the loads go into the structure. All the necessary drawings are performed through this software.

The necessary formulas for the design of the structures will be obtained from the respective Eurocodes. The Mathcad program will be used to implement these formulas, which offers the possibility of defining variables and calculating based on the formulas provided by the Eurocode. It is convenient as, in many cases, small mistakes might happen during the design of different structures, and Mathcad offers the possibility to change these variables, and all below calculations would be updated automatically.

3 Interpretation of building's design

There are different ways how a building should be designed. For the design of this two-apartment house, the following steps have been followed:

- Identification of the position of the building and the description of the place where this project will be built.
- The preliminary analysis makes it possible to create a clear design idea, starting from foundations, walls, and roof structures.
- Design terms, which include the requirements submitted by the client.
- The general analysis which identifies different parameters and additional actions that affect the structure.

3.1 Preliminary phase

Projects vary in size as well as complexity. Nevertheless, it should be possible for the designed project to meet all the conditions set out in the relevant Eurocodes and the National Annex of Finland. During the initial phase of the project, it is possible to meet with the client who expresses his idea for the project, and then the engineer will be able to create a better picture of what the project will look like.

The preliminary stage is crucial after setting the first objectives related to the project according to the client's wishes. Based on what the client wanted, it is possible to understand what should be suitable materials to carry out this project. At this stage, it is possible to get all the possible information from the client to achieve the correct result.

The design conditions must be clear to achieve good building performance, safety, and lifetime. The regular meeting with the client offered the opportunity to create different designs for this building.

For learning purposes, it is informed that in the preliminary phase, the model of the building was introduced. This will be a two apartment house, wherein the first floor and basement space will be used as living space in one of the apartments. The other apartment will be used for people who want to rent it for holidays. This is because the building is located in the north of Finland, a tourist area during the winter season. For the structural side of the building, part of the basement will be composed of a lightweight concrete block, while the upper part will consist of timber elements. In this design, the roof will consist of truss elements, information which is very important in the design of stud elements.

3.2 Evaluation phase

Once the primary selection of structures has been made, it is imperative to start the phase during which the drawings should be made and the control of the various elements from a structural point of view. In this document, the central part will consist of structural calculations, verifying stud walls, floor design, and foundation design. In order to carry out the necessary structural verifications, all the necessary drawings have been carried out in terms of the architectural point of view and structural types.

The design of different elements in this project includes different concepts where these concepts will be evaluated based on the performance of the analysis. Risk analysis is based on the principles shown on Finland's relevant Eurocodes and National Annex. The controls are made for different elements in terms of risk analysis, starting from the most critical structural phenomena and calculating and controlling less critical phenomena. So the elements, whereas a consequence of failure that can occur, the whole structure can collapse have priority in design.

3.3 Outcome phase

It has become possible for all project objectives to be met. The points that have been realized for this project are:

- Design structural elements following the relevant Eurocodes and the National Annex of Finland.
- Architectural drawings.

– Drawings for structural types.

The most important part of this document consists of structural calculators, where verifications are performed for the Ultimate Limit State following the National Annex of Finland. When performing various verifications, conservative methods are used for projects of this scale, which are not very complex.

4 Architectural drawings of the building

Architectural drawings are technical presentations, and based on them, it is possible to offer the client the idea of designing the project. Through these drawings, the client can see what the project will look like. Also, the client can suggest things to be included in the project at the stage during which the architectural drawings are presented. During the design process, there may be changes that need to be updated until the final stage of the project. This document will present all the necessary building plots, elevation, and floor plan drawings.

Note! To see the drawings in digital and high-quality form, one must follow the appendices mentioned in different chapters.

4.1 Data collection for drawings

In order to achieve the purpose of submitting the drawings to the client, it is necessary to gather information about the rules of these drawings that find application in Finland. It is worth mentioning that these documents are offered in the Finnish language, for which time and effort have been spent to make the relevant drawings. These rules have been followed to realize these drawings, defined according to the necessary Finnish standards. Below are the documents from which the necessary information was collected.

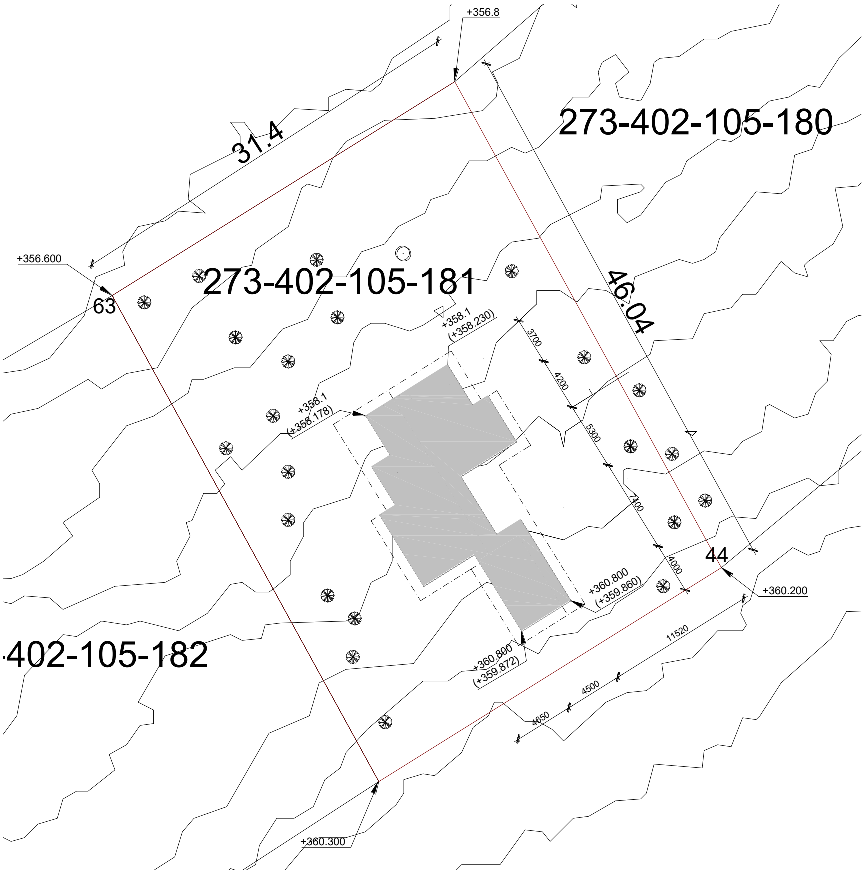
- Main drawings, specific plans, and studies, RATU 15-10824.
- Presentation instructions construction drawings, RATU 15-10635.

In the following chapters, as they relate to drawings, this information will be included in the description of the drawings, where the scale of the drawing, the spaces of the building, and others can be mentioned. These drawings respect the limits explained in these RATU cards, as otherwise when the application for the building permit is undertaken, the authorities may return the project. In this case, the respective clauses should be reviewed to ensure that everything complies with the rules.

4.2 Building plot drawing

This building is located in the northern part of Finland, with the address in Kolari, Lapland. The building plot drawing shows that everything presented in this plan follows Finland's land-use plan and building order (Figure 4). The rules for drawing an entire building must comply with the regulations set out in the Ratu cards. This document shows the rules that must be followed to meet the appropriate criteria. A scale of 1: 200 below is used for building plot drawing. The digital format of the building plot is attached in appendix 16.

Figure 4. Building plot drawing.

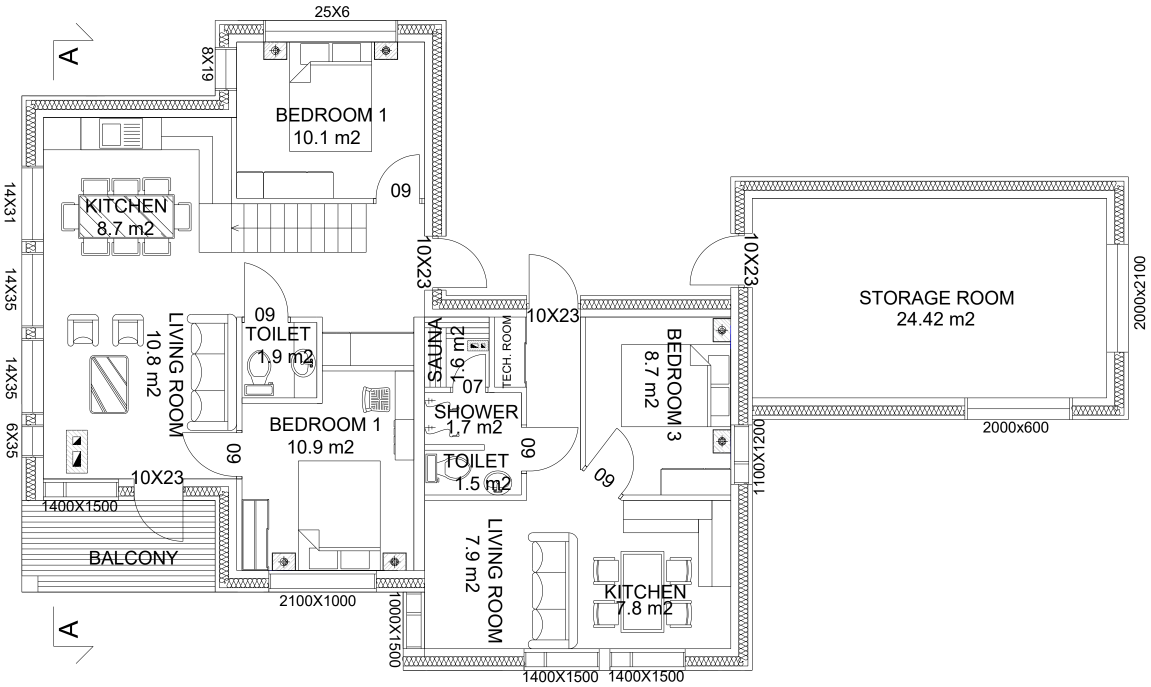


4.3 Floor plan drawings

The floor plan shows all the necessary information about the dimensions, the position of the essential equipment, the room's location, the building's entrance, and others (Figure 5). The drawings will be presented for the foundation, basement, and the first floor (Appendix 17). These plans are created based on the dimensions of the building, in accordance with the structural types of the building. The client determines the rooms' location, which plays a significant role in the design, as it becomes possible to make maximum use of the living space in this building. When the client selects the location of the rooms and the sizes of the windows, from the architectural point of view, the building is very presentable, but also this creates different challenges in terms of designing the structures. This document will show the calculation of timber panel resistances compared to the force coming from wind pressure. The first floor of the building will consist of two apartments. The first apartment will also include the basement as a living space. On the first floor, the kitchen and living room will share the same area in relation to their position. Also, the apartment will have two bedrooms which the client can use according to his needs.

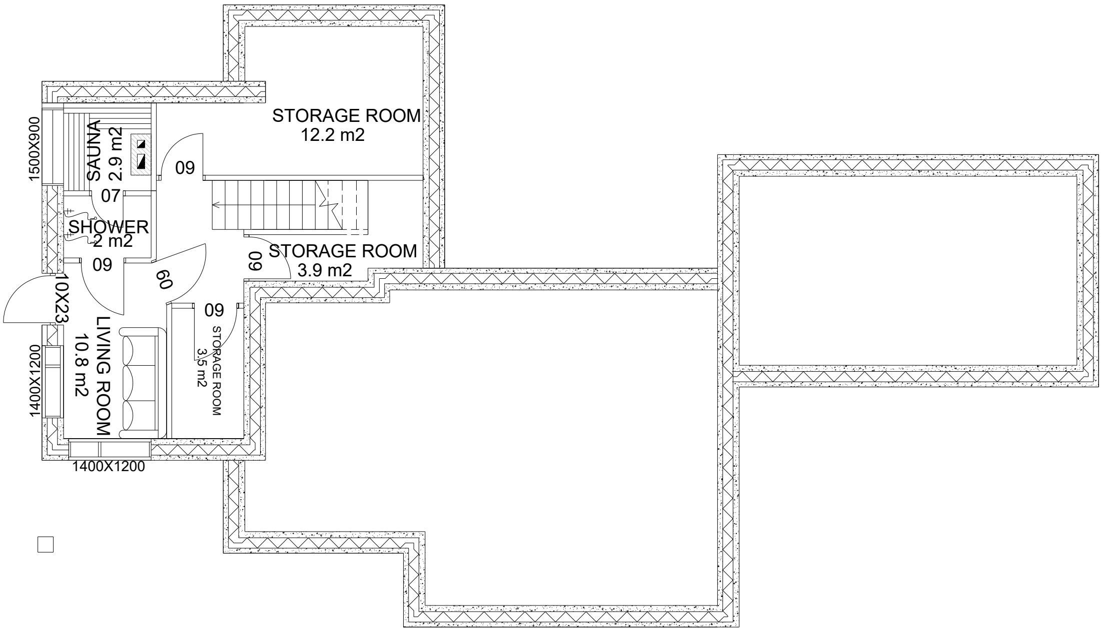
On the contrary, the other apartment will be smaller and consist of a kitchen and living room. Also, this apartment will have a bedroom. Both flats have utility spaces like showers and bathrooms, as shown on the floor plans. Also, there will be a storage room in this building that will store various items according to customer needs.

Figure 5. First floor plan.



As mentioned above, the basement will be the living space, which will consist of a living room and a sauna (Figure 6).

Figure 6. Basement floor plan.



Note! Providing a better quality, digital drawings have been attached at the end of the document (Appendix 17).

4.4 Elevation drawings

Presentation of the building at different angles is necessary as it manages to present how the building will look after its construction is completed.

These drawings illustrate the building from the outside. So, the facade of the building is presented in a parallel view from all four sides. Elevation drawings represent the floor levels and the materials of which the windows, the roof, and the walls of the building are made. The instructions which must be followed to create elevation drawings RT 15-10635 are followed, which presents all the necessary guidelines that must be followed for buildings designed in Finland. Below is one of the elevation drawings for this project (Figure 7).

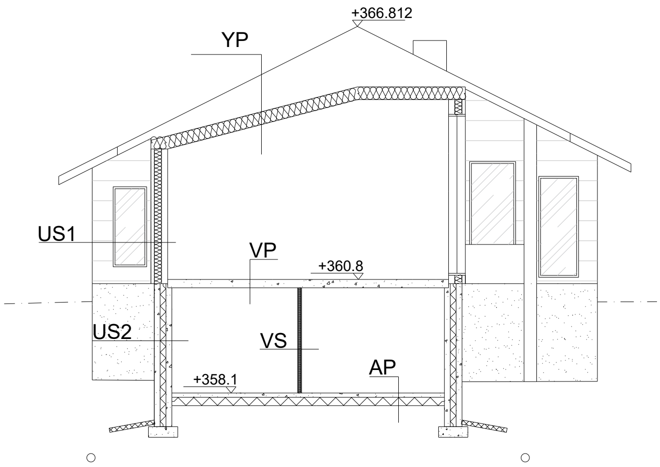
Figure 7. Back view of the building. .

Note! Images of the building from the other side are presented in digital form at the end of the document (Appendix 18).

4.5 Section drawing

Section drawings offer the opportunity to see or show a part of the building in a particular location. This is achieved by making an imaginary cut in a specific position, and based on this, all the elements are drawn, which are seen from that cut (Figure 8). The cutting position is shown in the respective plans. This type of drawing provides more detailed information regarding the walls and roof types that the building has. Like all architectural drawings provided in this document, section drawing follows the rules set out in RT 15-10635 following the National Building Code of Finland. Section drawings are presented in appendix 19.

Figure 8. Section drawing of the building.



5 Structural types drawings

This document also shows the structural types that the building will have. Since Finland has a cold climate, sufficient insulation must be provided for the building envelope to be comfortable during the customers’ stay inside the space. This chapter will explain the structures that the building will have, starting with the walls, floors, and roof. These drawings are essential because, through them, information is gathered about the structure, the strength of the material, and the dimensions. This data will be used in calculations and structural verifications of the building. Once the building structures have been determined, the necessary drawings for connecting these structures must be given to building a specific structure. Detail drawings are attached at the end of this document (Appendix 14).

Note! The thermal behavior of the structures is not within the scope of this document.

5.1 Wall structures

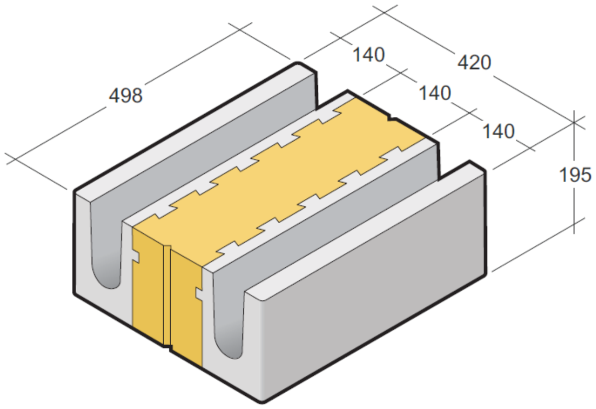
This building has a basement and first floor; therefore, the client determines the wall structure for each floor, and the drawings are done based on the information received. The basement wall will consist of blocks based on lightweight concrete, while the first-floor wall will be of the timber structure.

5.1.1 Basement wall

The basement wall will be constructed of Leca blocks that offer flexibility in the construction phase and thermal behavior following Finnish standards (Figure 9). The design and work manual of the blocks is based on dimension under SFS-EN 1996-1-1 (Leca Finland Oy, 2018). Leca ingots create a durable and robust wall structure despite their lightness and workability. It is easy to install horizontal reinforcement in the groove so that the mortar surrounds the steel, protecting them from corrosion and ensuring the cooperation of the steel and the ingot.

These blocks will be a load-bearing structure that will take the load from the upper floor and transfer it safely to the ground. The purpose of the foundations is to transfer the loads caused by the building to the ground. The most significant factor to consider when designing the foundations of a detached house is the subsidence of the foundations. In addition to the permissible loads determined based on the stresses, it must be checked that there is sufficient safety in some cases.

Figure 9. LTP-420-6 Leca (Leca Finland Oy, 2018).



Since the wooden structure will be placed at the top of the block, special care must be taken to protect the wood material from moisture. The joint between the base wood and the foundations is sealed against air leaks. For example, rubber bitumen strips, closed-cell plastic tape, or polyurethane foam can be used as insulation and sealant.

The purpose of the basement determines the required thermal insulation. They are usually designed according to the requirements of the living quarters. In this project, the basement will be a living space; therefore, the wall insulation will be done up to the bottom to provide better thermal comfort for occupants. Below ground level, the thermal resistance of the ground can be taken into account.

The boundary state method's boundary states are the ultimate and operating limit states. The fracture at the limit state shows that the strength of the structure or crosssection calculated by the calculation strengths is equal to the stress determined from the calculation loads. In the service limit state, it is checked that the structural crack calculated according to the specific strengths and specific loads and deformations are within acceptable limits (Leca Finland Oy, 2018).

Note! The structural type of the basement wall is shown at the end of this document (Appendix 14).

5.1.2 First-floor wall

The first-floor wall will be a ventilated facade, including a uniform ventilation space between the cladding and insulation layers. This offers a better performance in terms of protection from moisture, to which the wood is very delicate. The selection of the wall is from the company Paroc, and the same company will produce the insulation materials. When designing a wooden frame wall structure, several aspects that impact the wall's fire, thermal, and moisture performance must be considered. The timber frame structure is dimensioned according to load and energy efficiency requirements. The soft sheet insulation made of Paroc stone wool is elastic and rigid, making it easy to fit the surrounding structures without fasteners (Paroc, 2022).

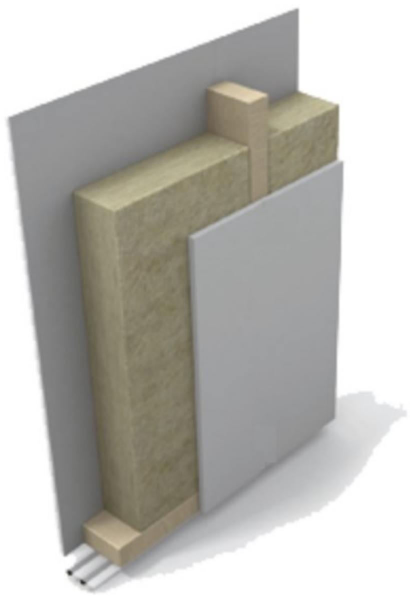
To protect the wall structure from fire, tests have been applied following SFS-EN 1995-1-1. Also, a tight air and vapor barrier layer are placed between the wall layers to control moisture. The critical point for the moisture barrier is providing complete continuity at the connection point, for example, between the wall and roof structure. Also, the moisture barrier must be protected from various penetrations during the installation of the cables; therefore, the cables are placed from the inside to avoid drilling the moisture barrier during the construction phase. The central part of the wall is in the middle of the structure, which consists of an insulation layer and the load-carrying stud with dimensions 48x173 millimeters. The controls set out in the relevant Eurocode will be followed to see if this type of wood manages to perform appropriately during the life of the building, where it will be loaded with different loads.

Note! The structural type of the first-floor wall is attached at the end of this document (Appendix 14).

5.1.3 Partition wall

In most cases, partition walls are non-load bearing and are used to separate the different spaces of the building. A partition wall must meet requirements mainly related to sound insulation, impact resistance, and fire compartmentation. For this project, the interior walls will have acoustic insulation, through which the room occupants would be able to have their privacy. In most cases, the outer covering of the wall consists of gypsum boards, but in this project, the covering will be made of wooden boards according to the client's wish. An interior wall model is presented below (Figure 10).

Figure 10. Partition wall (Paroc Group Oy, n.d.).



Note! The structural type of the partition wall is attached at the end of this document (Appendix 14).

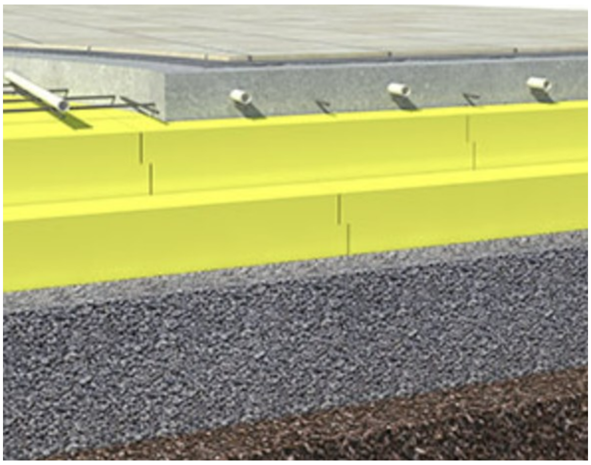
5.2 Floor structures

This project will have two different floor systems. One of them consists of the traditional way concrete will be poured on construction sites, while for the first floor, a solution will be made that offers economic and time benefits. Different companies in Finland do different tests and offer products that replace the old methods of creating floor slabs. One of these companies, called Ruukki, offers load-bearing sheets in which concrete will be poured straight. The use of these load-bearing sheets has the same effect as in the case of reinforcing bars in concrete, but in this type of solution, it is not necessary to use formwork, which reduces the cost and construction time.

5.2.1 Floor against the ground

Floor against the ground is essential because moisture problems will appear if insufficient insulation is provided. Also, the floor structure must be tight to prevent radon gas from coming inside the space as it is harmful to the occupants. For this project, the floor structure will be taken from Finnfoam, which offers a good performance of underground slabs (Figure 11).

Figure 11. Finnfoam floor structure presentation.



These concrete slabs have functionality as well as structural tolerance. The elements mentioned above are essential as they offer structure, functionality, and longevity. Below the concrete slab will be thermal insulation, which protects against water damage. Insulation provided by the Finnfoam company performs very well in terms of strength, ensuring that no failure will happen. Since this structure will be in direct contact with the soil, the relative humidity values in the soil are about 95-100%, and for this reason, the insulation provided can have high vapor resistance (Finnfoam Oy, n.d.). Despite this, using this type of insulation, one of the most common problems in residential houses, which is further related to ground heating, becomes possible to avoid. For this type of floor, the insulation from Finnfoam is poorly permeable to water vapor; therefore, this makes it possible for the concrete to dry from the inside, making the level of moisture content in the concrete slab low.

Note! The structural type of the floor against the ground is attached at the end of this document (Appendix 14).

5.3 First floor

Floor structures are important because they serve as support for the superstructure of the building. In addition, this floor structure is one of the main elements of the building as it receives the various loads applied to it and transfers them to the relevant elements such as load-bearing walls.

For the floor structure design, this chapter will show the traditional way of building the floor structure and an innovative method that includes composite sheets for the construction of the floor structure.

5.3.1 Innovative floor design

The traditional method can be used for the concrete slab design, which uses wood materials to create formwork for the concrete to be poured. The concrete is then left for a certain period until it gains enough strength; the formworks are removed accordingly. This method is used in different projects, but with technological development, other companies do testing and offer innovative ways that provide the same result in construction work. Because the metal sheet, in this case, acts as reinforcement, the use of concrete in the floor structure will be reduced. Also, the need to reinforce steel decreases as the metal sheet acts as reinforcement and takes the bending moments. Therefore, the cost of this product would be smaller compared to the traditional method of building floor structures.

In this project, the software provided by the company Ruukki is used to design the first-floor slab. This company uses the load-carrying metal sheet known as ComSlab, used as formwork during construction, and concrete will be poured on top (Figure 12). This minimizes construction costs as the metal sheet will be there. There is no need to remove any formwork of the slab compared to timber formwork, where the formwork must be removed after the strengthening period of concrete. The metal sheet can be used as a ceiling because of its aesthetic design, avoiding using a suspended ceiling (Ruuki, n.d.).

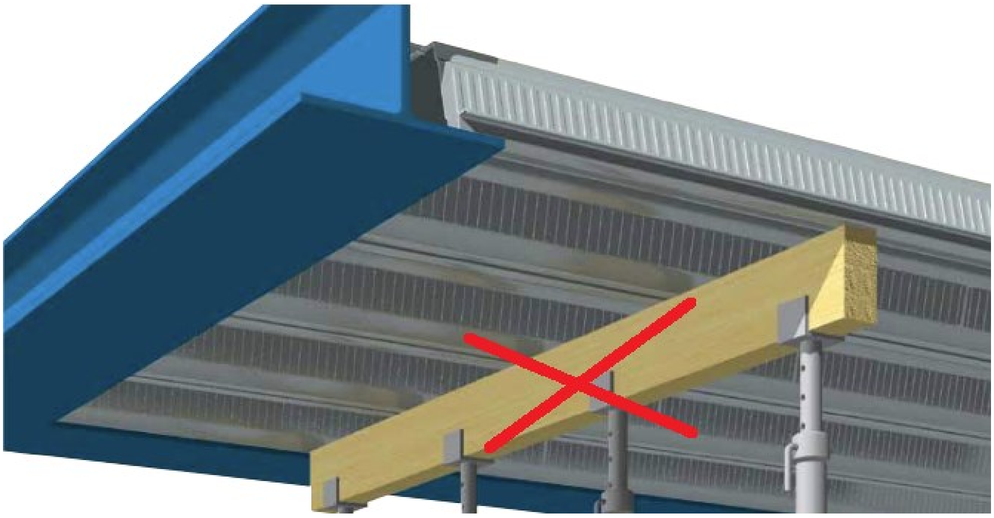
Figure 12. Composite sheet CS48-36-750 (Ruuki, n.d.).

Building with a composite floor system has many advantages, which are:

- Speed of construction.
- The deck is acting as a working platform.
- Reducing the weight of the floor.
- Fire resistance.

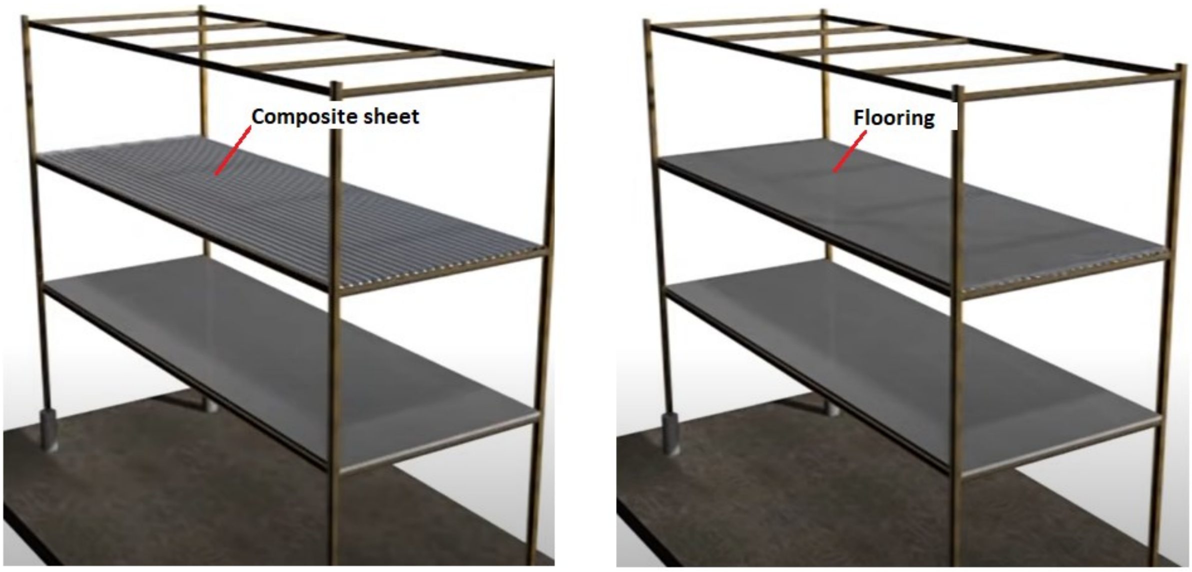
This product has been tested following Eurocode 3 for cold-formed steel and Eurocode 4 for composite slabs and beams. The innovation of this type of sheet replaces the timber formwork, which is time-consuming. Besides this, for projects of this scale, which are not very complex and the span of the slab is not going to be significant, the thickness of the composite slab can be decreased, making the cost even smaller. Moreover, these types of composite sheets in length from 4 meters to 4.5 meters do not need propping compared to timber formwork, where these proppings are necessary (Figure 13).

Figure 13. Composite sheet slab (Tata Steel UK Limited, 2017).



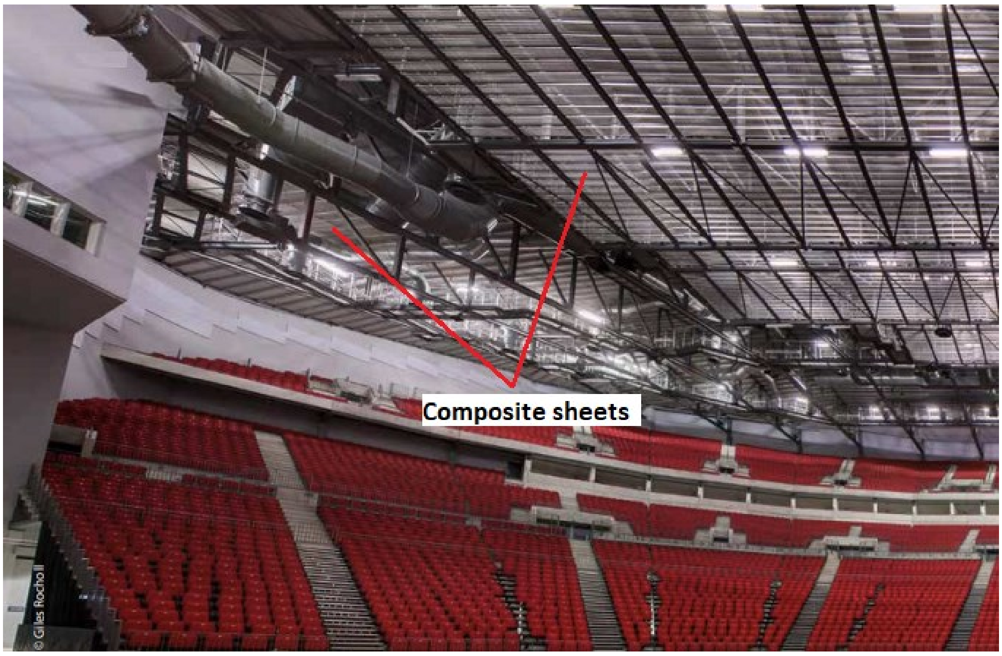
The composite sheet acts as formwork during the concrete casting phase, leading to faster occupancy because the construction steps are simple and do not require excessive time. The composite sheet placed on the support points for this project will be the load-carrying wall provided by the Leca company. A demonstrative erecting step of the composite slabs is shown below (Figure 14).

Figure 14. Erecting steps of composite sheet.



Apart from the fact that these floors offer a cost-effective range, from the outside, they are aesthetically pleasing, and underneath they will not need any surface material (Figure 15).

Figure 15. Composite slabs used in a large project (Tata Steel UK Limited, 2017).



Note! The structural type of the first floor is attached at the end of this document (Appendix 13).

5.3.2 Traditional floor design

Creating a concrete slab traditionally involves a process during which the following steps are performed:

- Preparation of formwork.
- Placement of reinforcement.
- Casting concrete.
- Removal of formwork.
- Curing the concrete slab.

For floor slabs, the initial stage involves creating the correct formwork. Enabling a process during which all formwork elements are placed in regular shape reduces the chances that the concrete slab will have damage during its lifetime. To perform the formwork, specific rules must be followed so that during the concrete casting phase, the structure can withstand the load but not only. Also, the formwork element must be placed correctly to avoid concrete leaking.

In case of any error during the erection phase, additional costs may be required. Furthermore, when using this method, it must be possible for the level of the sheathing board on which the concrete will be cast to be the same because the Serviceability Limit State problem may occur after the concrete is cured. For example, the concrete may crack, which for such structures is critical and must follow the limits set in the relevant Eurocode. The construction of the formwork must be done correctly, and this, in most cases, takes time, as all the supporting elements must be within the defined rules.

Figure 16. A typical formwork configuration for floor slabs (Uzodimma, 2020).

After this phase is done, the installation of bars stops the cracking of concrete in cases when there are different loads. At the same time, this makes the concrete floor more durable. At this stage, the irons must be placed according to the distances defined following the plans.

Before the concrete is poured, in most cases, the slabbed will be sprinkled with water to prevent moisture loss (Concrete Floor Slab Construction Process 2021, n.d.). Then the concrete will be poured, where during the drying phase, excessive control should be provided for the joints, adding control points; this for the fact that the concrete shrinks, and the goal is to reduce as much as possible the cracks in concrete. Finally, the concrete must be left to dry until it has sufficient capacity to remove the formwork. For a large-scale project, the following figure shows what the formwork would look like (Figure 17).

Figure 17. Formwork of a project (Uzodimma, 2020).

5.3.3 Conclusion

Since the floor slab design does not occupy a large area in this project, it was decided to use this innovative way for its advantage. Based on this benefit, the use of composite sheets by the company Ruukki will construct the floor structure for this project. A summary table showing the factors that influenced the selection of this solution is shown below (Table 1).

Table 1. Key markers that affected the design selection.

Traditional method (Timber formwork)	Innovative method (Composite sheets)
Higher costs, because of labour work and materials	The most extensive cost-effective
Slower build time as a result of creating the formwork	Faster build time as the process includes placing the sheet and pouring the concrete
Propping is needed to support the slab deck	For span from 4 to 4.5 meters propping is not needed
Not preferable for handling	Easy handling
More weight as more reinforcement will be needed	Less weight as the sheet is acting as reinforcement
In case of fire, timber is volatile to fire	In case of fire, the profile has high fire resistance
Limited use	Can be used once
Surface material is needed after removing the formwork	Aesthetically pleasing, even without surface material

5.4 Roof structure

In Finland, the roof must have sufficient capacity to support the snow load due to climatic conditions, especially during the snowy winter months. Therefore, the roof must be sloping so that the snow slides off; otherwise, if the roof were flat, much snow would accumulate, making the design difficult but also, in practice, would not be the right solution. For residential homes, in most cases, the roofs are duo-pitched. The same goes for this project, where the slope is 1: 2. It is essential to mention the fact that the basic structure of the roof will consist of truss elements located at a distance of 900 millimeters. As a result of the stack effect, the air tends to go bottom-up so that a thick insulation layer will minimize thermal energy loss.

Note! The structural type of the roof structure is attached at the end of this document (Appendix 14).

6 Design of timber structures

Before starting precise calculations, it is necessary to have a clear view of the resources from which the data for design will be obtained. Each of the following chapters indicates the theoretical source of information used as a basis for conducting practical activities.

As for the load-bearing structures of the building, it is first necessary to create a transparent model. This offers the possibility of being conservative, which enables ease in structural calculations but at the same time makes it possible for the design of the building to be on the safe side.

The modeled structures must be constructed according to a specific design, including limits. Since, in reality, it is impossible to avoid displacements and deformations during the phase when the structure is loaded with weight, these things must be checked respectively so that the design is safe for people.

Based on the Eurocodes and the National Annex of Finland, the requirements regarding structural safety are entirely related to the definition of the situation during which the structure fails to meet the performance requirements. Consequently, the system's design is based on various verifications through which it is shown that the conditions for the selected materials and dimensions are correct. There are different ways to achieve the functioning of the structure, including calculations according to Eurocode and the National Annex of Finland or testing the material in laboratory conditions. For the two-apartment house shown in this document, analyses have been used to verify that the selected material will perform adequately when the different loads come into the structure. The integrity of the design is guaranteed by taking into account the state of the ultimate limit state according to the relevant Eurocode.

Calculations based on the Ultimate Limit State control are related to collapses or other forms of structural failure. This phase includes loss of stability, failure of the structure through pronounced deformations, disruption of balance, and the transformation of structures into a mechanism.

On the other hand, through Serviceability Limit State, it is possible to control deformations that affect the structure's appearance, vibrations that cause discomfort to people, and damage to the structure through cracking, which has a significant effect on the stability of the design.

6.1 Designed and checked structures

The design of this project in its central part includes the design of wooden structures based on several aspects that include:

- Control of the wall structure in the Ultimate Limit State.
- Stability control for walls.
- Header sole plate connection control.
- Control of wall diaphragms.

Additionally, the design of concrete structures includes the design and control of the following structures:

- Design and control of the strip foundation.
- Design and control of the concrete slab for the first floor using Ruukki's solutions software, ComSlab.

6.2 Basic principles of working in limit state design

The model of this house is based on different limit states, which take into account:

- Characteristics of the material which will be used to build the project.
- The way these materials behave under specific conditions, such as the load duration, can be mentioned.
- Climatic conditions have a critical role in timber structures.
- Construction phases, during which some elements of the leading design can be changed, including the condition of the supports.

6.2.1 List of Eurocodes

Designing structures for this project considers different variables, including the properties of materials, geometric information, further actions acting in the structures to be designed, and coefficients obtained from relevant Eurocodes.

To ensure the control and design of structures, the data are taken from:

- SFS-EN 1990: Eurocode: Basis of structural design.
- SFS-EN 1991: Eurocode 1: Actions on structures.
- SFS-EN 1991-1-1 Densities, self-weight, and imposed load.
- SFS-EN 1991-1-3 Snow load.
- SFS-EN 1991-1-4 Wind actions.
- SFS-EN 1995-1-1 Design of timber structures.
- SFS-EN 1997-1 Geotechnical design.

6.2.2 National Annex of Finland

Each European country can create a National Annex (NA) for different parts that refer to the Eurocodes produced by the Technical Committee CEN / TC. The National Annex of Finland is used for this project as it provides coefficients or decisions related to national parameters that take into account climatic conditions and admissibility in deflection.

6.2.3 Eurocodes used to design structures

To design the structure, specific Eurocodes will be used so that the structure will be able to withstand the loads that will come into the structure. Eurocodes are based on applied 'Principles' and 'Application rules.'

The Principles are exact, and no other alternatives are allowed. They must be followed exactly as described in the relevant Eurocodes. As a Principle, for instance, the control of the steel plate where the defined clauses must be followed precisely to make sure that the plate will withstand all the loads (IStructE/TRADA, 2007).

The Application rules are rules which comply with the Principles. Other types of rules can be used in reality to be more conservative. Still, they align with the Principles and give approximately the same strength, safety, and serviceability level.

As for the Application rules, an example can be noted in the wind pressure calculation on the structure. In SFS-EN 1991-1-4, chapter 7.2.2, the wind pressure is precisely defined in different building zones with rectangular walls. Still, other more conservative methods are used to calculate the wind pressure in a facile way. It is worth mentioning that it finds wide use for structures for which the behavior is known. For instance, small residential buildings fall into categories where conservative methods can determine the loads applied to the structures. These conservative applications find no application when the complex project involves a specific connection check.

6.2.4 Notation of variables used in calculations

Greek and Latin characters are applied following the relevant Eurocodes, where their description is made to the left of the formula, in the appendices where the precise control of structures is done.

7 The essential design of the building

Based on SFS-EN 1990: Eurocode: Basis of structural design, it is possible to meet the conditions related to safety, serviceability, robustness, and durability. In addition, from the relevant Eurocodes, the design can meet all acceptable requirements regarding the Building Regulations of Finland.

7.1 Structural design

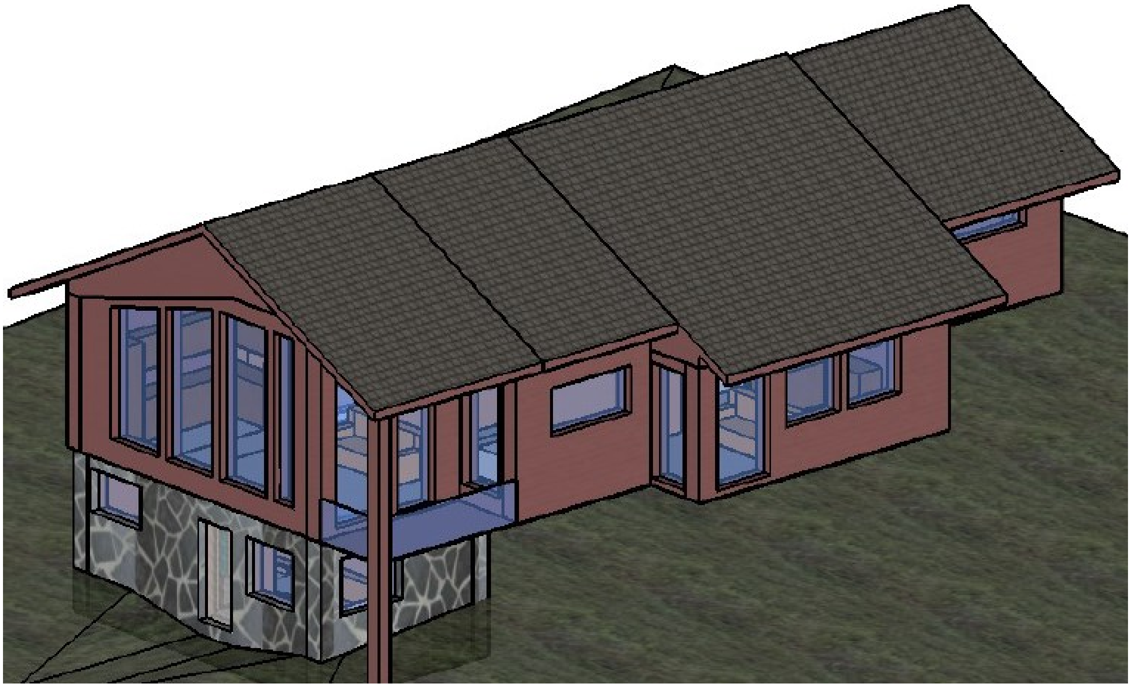
All calculations of structures are made following the SFS-EN 1995-1-1 design of wooden structures based on the principles presented in SFS-EN 1990: Eurocode: Basis of structural design. If different values of the coefficients are more accurate concerning the calculations, they can be used to check if the structure will be able to withstand the loads; otherwise, SFS-EN 1991-1-1 Densities, self-weight, and imposed load must be followed by the National Annex of Finland.

It is not easy to check the structures based on formulas in different projects, so the design method is based on tests. By performing other tests, accurate conclusions about the same number of components on a large scale are reached. The project described in this document is a two-apartment house designed following the relevant Eurocodes and National Annex of Finland; therefore, it is not necessary to design structures based on testing for a project of this scale.

7.1.1 Initial dimensions

In the initial phase of the project, the client provided a model of the building to create an idea about the dimensions of the various structures (Figure 18). This applies to structures that will serve as load-carrying members, following the relevant Eurocodes. It is possible to follow conservative methods such as considering loads with approximate values for buildings that are not very complex and structurally simpler than elaborate buildings. Furthermore, tables can be used during this phase with ready information about structures commonly used in these projects.

Figure 18. Preliminary sketch up model of the project.



7.2 Structural building materials

Products used for structural reasons must comply with Construction Products Directive (CPD / CPR). Accordingly, the wood material used to realize this project must be listed in EN 14081-1. Hence, the building consists of a timber frame, and the material is solid timber; therefore, this standard finds application for this project. Based on the relevant standard for solid timber, it becomes possible to check whether the particular dimensions, for example, the studs, perform correctly when the load is on them.

For the design of concrete structures, light-weight concrete blocks and cast-in-situ will be used.

7.3 Responsibility for designing structures

The responsibility generally falls on the engineer who will design the structures. Through calculations based on Eurocode and in the National Annex of Finland, the engineer ensures that the structure will achieve sufficient strength, durability, and serviceability. These must comply with customer requirements as well as the Finnish Building Code. A structural engineer makes it possible to design and control load-bearing structures.

7.4 Use of building and location

The geographical coordinates of the building should be determined in the initial stages of the project based on the position of the building; it is possible that the various actions that include the loads coming into the structure to be determined following the relevant Eurocode as well as the Nation Annex of Finland. For instance, Finland is known for a cold climate where snow density is high during the winter period. The thickness of snow is not linear everywhere in Finland but varies according to the geographical position; when in its northern part, the density of snow increases. This would result in overload on the structure, where control must be precise to ensure the structure's integrity. So, the essential information must be precisely defined for the structure to be correctly controlled in the initial stages of the project.

7.5 Data collection of building's design life

The design life expectancy of the building is specified in SFS-EN 1990: Eurocode: Basis of structural design. From table 2, it can be seen that the designed project is in design working life category 4 (Commitee, 2002). The intended project is a residential building and belongs to building structures example.

Table 2. Indicative designing working life (Commitee, 2002).

Design working life category	Indicative design working life (years)	Examples
1	10	Temporary structures[(1)]
2	10 to 25	Replaceable structural parts, e.g. gantrygirders, bearings
3	15 to 30	Agricultural and similar structures
4	50	Building structures and other common structures
5	100	Monumental building structures, highway and railway bridges, and other civil engineering structures
[(1)]Structures or parts of structures that can be dismantled with a view of being re-used should not be		

7.6 Design situations

For the design of structures, it is necessary to provide sufficient stability, strength, and serviceability. It is emphasized that during the construction phase of the project, the capacity of the structures is not the same as what is calculated based on Eurocodes and the National Annex of Finland. This is known as the execution phase, where the elements are being put together, so they can not develop the total capacity for which they were designed. In this case, the structure must withstand the loads that may come on it during the execution phase.

After the execution phase, the structure's design during its lifetime comes. The project is built and manages to resist all loads according to calculations made following the relevant Eurocodes.

Additionally, design in accidental situations should be considered where other terms or events may be included. Finland is a seismic region with low earthquake recurrence rates and small magnitudes (VARPASUO, 2008). Therefore, for a project of this scale, the calculations for accidental loads can be neglected. In case of design for accidental loads, SFS-EN 1998 Eurocode 8: Design of structures for earthquake resistance must be followed.

7.7 Stability design of the building

The wood material is light in weight, and stability control is significant for this material. Structures must withstand vertical loads, and one of the main problems is buckling in different modes. Additionally, the wind plays a vital role in the phenomena, including uplifting, overturning, and sliding forces; therefore, the structure must be able to resist these actions.

Note. For the control and design of elements under tension, favorable action must be followed. This offers more instability in terms of vertical load action, and at the same time, wind action offers more instability, resulting in the increased bending moment. This makes the structure under tension more destabilized; therefore, designing this way would provide a more reliable strength, durability, and serviceability.

7.8 Environmental effects

Given that the building will be designed as a timber structure, environmental effects will affect the wood material. These effects include moisture, thermal, and creep effects explained in the following chapters.

7.8.1 Moisture effect

Wood is a heterogeneous material consisting of compounds that are not evenly distributed inside the material. For this reason, the level of moisture in the wood depends on factors such as temperature and humidity. When there is a variation of these two factors, the material along the grain will swell or shrink. This phenomenon goes in proportion to the moisture level. Therefore, an increase in the moisture level would result that the grain of timber would expand, and on the other hand, with a decrease in the moisture content, the grain of wood would dry, making it shrink. As a reference, during the drying phase, the wood shrinks from the wet phase to the arid phase, which makes it possible for the wood to shrink on average about 8% in the tangential direction, with 4% in the radial direction as well as 0.2% to 0.4% in the direction of the grain (Puuinfo, n.d.). If needed, protective coatings can be used to protect from moisture effects.

7.8.2 Thermal effect

The thermal effect on wood material is usually negligible as the linear coefficient for thermal expansion is lower than other materials such as iron or concrete. As a result, the change in the dimensions of the wood can come as a consequence of the shrinking effect in cases when the wood transits from a wet phase to an arid phase.

7.8.3 Creep effect

Creep is a deformation that would occur in different construction materials under loading. This phenomenon occurs under long-term load, which would result in deformations of the material. Serviceability Limit State should be checked for timber member for deformations of the member.

8 Robustness

Wooden structures can withstand the impact of accidental load. Consequently, it must be guaranteed that the whole structure should not collapse due to the loss of some elements. This clause finds application for large and complex projects in most cases. For projects of this scale, including residential houses, the relevant standard can be neglected if there is no possibility for accidental loads; otherwise, SFS-EN 1991 Part 1-7: General action - Accidental action must be followed.

8.1 Fire-resistance of building materials

Under the influence of fire, timber material loses its properties, damaging the structure. The strength capacity for timber material under fire conditions depends on many factors. Despite this, timber assemblies exposed to fire conditions must meet specific requirements not to damage the structure's integrity.

Mechanical resistance through which the structure's integrity is maintained to evacuate people inside the apartment for a specific time is critical. In most cases, the time for which the element manages to keep its properties in fire conditions depends on the type of wood. There are wood materials with fire resistance starting from 30 minutes and more. More insulation can be used, and sacrificial timber can improve fire resistance. Insulation provides the ability not to spread fire in buildings that may be near the apartment in fire conditions. While sacrificial wood can be placed around the load-bearing element, in case of fire, they are initially burned, protecting the load-bearing part of the structure. This type of timber is used in columns and offers good protection conditions for the structural part of the element.

8.2 Requirements for thermal, acoustic, and airtightness

The structure's design meets the conditions described in the National Building Code of Finland regarding acoustics and thermal and airtightness conditions. After the construction phase is completed, an airtightness test, otherwise known as the blower door test, can be conducted to see if the building fulfills the requirements of the National Building Code of

Finland. Furthermore, observations with a thermal camera can check the wall and its insulation. These factors are not directly related to structural design, and therefore, information on them is limited.

8.3 The durability of the structures

The designed structures of the building, including the walls, foundations, and slabs, must provide safety throughout the life of the building. To provide such a thing, structures must be protected from the phenomenon of permanent frost and moisture. Various insulators preserve the foundations that protect the concrete from frost or moisture. Also, it becomes possible for the walls to ventilate various cavities to protect from moisture by bringing air, which minimizes the risk of this phenomenon.

In conclusion, designing the building this way makes it possible for these structural elements to perform in the best possible way while reducing reasonable maintenance costs.

9 Data collection of service's building class

Timber is a material whose strength and deformation depend on environmental conditions. For example, the moisture phenomenon makes timber lose strength, and the chances of the creep phenomenon happening are increased. In addition to that, the moisture level is a phenomenon that depends on environmental conditions such as temperature or humidity, which would make the timber go into swelling or shrinking cycles. The serviceability of the building intended to be designed belongs to Service Class 1 (Table 3).

Table 3. Serviceability class (AUSTRALIAN SUSTAINABLE HARDWOODS (ASH), 2021).

Service class	Environmental conditions	Situations
Service class 1 (Interior, above ground)	Moisture content in the material corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeing 65% for a few weeks per year. The average moisture content in most species will not exceed 12%.	Inside with no exposure to weather or wetting. Attack from insects is possible dependant on the geographical region
Service class 2 (Exterior, under cover within 30° of roof line, above ground)	Moisture content in the material corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeing 85% for a few weeks per year. The average moisture content in most species will not exceed 20%.	Subject to occasional wetting such as condensation or UV degradation but is undercover and not exposed to persistet wetting. Attack by fungi is possible and attack from insects is possible dependant on the geographical region.
Service class 3 (Exterior fully exposed, above ground)	Is characterized by higher moisture contens than Service Class 2	Subject to full weather. Attack by fungi and UV degradation is expected and attack from insects is possible dependant on the geographical region.

9.1 Load-duration classes

Load durations are different because they operate on structures at different periods. An increase in load duration would reduce the loads that the timber structure could withstand, and consequently, the creep phenomenon would increase, but the stiffness, on the contrary, would be reduced.

Based on the load duration class, the appropriate k_{mod} coefficient is selected based on the SFS-EN 1995 Design of Timber Structures. As a result of sensitivity to environmental conditions over a long period, the timber material loses strength. Consequently, this must be considered; therefore, load duration classes are also presented in Eurocode to consider the fact mentioned above.

When checking timber structures in Ultimate Limit State (ULS) combinations, there will be a combination of several loads in which the shortest term action will be used to select the k_{mod}. For the control of wall studs, in this document are shown the different combinations of loads from which the shorter load term action is selected.

9.2 Data collection of modification factor

Modification factor k_{mod}, considers the effect of the load duration and moisture content (Table 4). The timber material for this project is solid timber, and the coefficient can be found in the table below.

Table 4. Modification factor (CEN/TC250, 2015).

Material	Standart	Service class	Load-duration class				
			Permanent action	Long term action	Medium term action	Short term action	Instanta-nous action
Solid timber	EN 14081-1	1	0.6	0.7	0.8	0.9	1.1
		2	0.6	0.7	0.8	0.9	1.1
		3	0.5	0.55	0.65	0.7	0.9
Glued laminated timber	EN 14080	1	0.6	0.7	0.8	0.9	1.1
		2	0.6	0.7	0.8	0.9	1.1
		3	0.5	0.55	0.65	0.7	0.9
LVL	EN 14374, EN 14279	1	0.6	0.7	0.8	0.9	1.1
		2	0.6	0.7	0.8	0.9	1.1
		3	0.5	0.55	0.65	0.7	0.9
Plywood	EN 636						
	Part 1, Part 2, Part 3	1	0.6	0.7	0.8	0.9	1.1
	Part 2, Part 3	2	0.6	0.7	0.8	0.9	1.1
	Part 3	3	0.5	0.55	0.65	0.7	0.9
OSB	EN 300						
	OSB/2	1	0.3	0.45	0.65	0.85	1.1
	OSB/3, OSB/4	1	0.4	0.5	0.7	0.9	1.1
	OSB/3, OSB/4	2	0.3	0.4	0.55	0.7	0.9
Particle-board	EN 312						
	Part 4, Part 5	1	0.3	0.45	0.65	0.85	1.1
	Part 5	2	0.2	0.3	0.45	0.6	0.8
	Part 6, Part 7	1	0.4	0.5	0.7	0.9	1.1
	Part 7	2	0.3	0.4	0.55	0.7	0.9
Fiberboard, hard	EN 622-2						
	HB.LA, HB.HLA 1 or 2	1	0.3	0.45	0.65	0.85	1.1
	HB.HLA1 or 2	2	0.2	0.3	0.45	0.6	0.8
Fiberboard, medium	EN 622-2						
	MBH.LA1 or 2	1	0.2	0.4	0.6	0.8	1.1
	MBH.HLS1 or 2	1	0.2	0.4	0.6	0.8	1.1
	MBH.HLS1 or 2	2	-	-	-	0.45	0.8
Fiberboard, MDF	EN 622-5						
	MDF.LA, MDF.HLS	1	0.2	0.4	0.6	0.8	1.1
	MDF.HLS	2	-	-	-	0.45	0.8

9.3 Data collection of deformation factor

A deformation factor can be used for the timber installed at or near its fiber saturation point, which would probably dry out under load (CEN/TC250, 2015). For this two-apartment house, the building material is solid timber, and it belongs to service class 1; therefore, the value of k_{def} is taken as 0.6 (Table 5).

Table 5. Deformation factor for timber and wood-based material (CEN/TC250, 2015).

Material	Standart	Service class		
		1	2	3
Solid timber	EN 1408-1	0.6	0.8	2
Glued Laminated timber	WN 14080	0.6	0.8	2
LVL	EN 14374, EN 14279	0.6	0.8	2
Plywood	EN 636 A1>			
	Type EN 636-1	0.8	-	-
	Type EN 636-2	0.8	1	-
	Type EN 636-3 <A1	0.8	1	2.5
OSB	EN 300			
	OSB/2	2.25	-	-
	OSB/3, OSB/4	1.5	2.25	-
Particleboard	EN 312 A1>			
	Type P4	2.25	-	-
	Type P5	2.25	3	-
	Type P6	1.5	-	-
	Type P7 <A1	1.5	2.25	-
Fibreboard, hard	EN 622-2			
	HB.LA	2.25	-	-
	HB.HLA1, HB.HLA.A2	2.25	3	-
Fibreboard, medium	EN 622-3			
	MBH.LA1, MBH.LA2	3	-	-
	MBH.HLS1, MBH.HLS2	3	4	-
Fibreboard, MDF	EN 622-5			
	MDF.LA	2.25	-	-
	MDF.HLS	2.25	3	-

10 Load actions

Different loads will come into the structures due to different load actions. In Finland, in general, loads that go into the structure include permanent loads that represent the weight of the structure itself, finishing layers, and elements such as partition walls that are immovable unless otherwise specified. Permanent loads in this document are represented by the capital letter $G_{k.j}$:

Where

- j take values from j=1,2…n.

In addition, there are variable actions for this project consisting of snow, live, and wind load. The variable loads in this document are represented by the capital letter $Q_{k.i}$:

Where

- i take values from i=1,2…n.

Timber material is composed so that the load it can carry depends on various factors. This means that the load capacity that the wood can carry depends on the time for which this load is applied. In cases when load actions act together, then any possible combination of these individual load actions must be calculated. In Finland, during the stud checking process control process, this method is used, and based on the shorter load term, the appropriate coefficients will be selected to check if the structure can withstand the loads.

10.1 Characteristic and design loads (ULS)

Characteristic loads are calculated based on the different dimensions of the material and the relationship with the density of the material. Also, for other occasions such as snow, wind, or live load, the respective Eurocodes have been followed to get their values. Once all these loads have been selected, ULS combinations will offer the opportunity to design different structures and check if these structures manage to withstand these loads. Actions can be favorable or unfavorable in the Ultimate Limit State (ULS). In cases where unfavorable action conditions are dominant, the coefficients used to find the design load is higher. On the other hand, if a load creates favorable action, the coefficients take a smaller value because favorable action tends to make the structure more stable.

As a structural designer, the objective is to place the structure in the worst possible condition so that the design of the various elements is as safe as possible. For instance, to check whether there is tension in the foundation, the concept is that it should be applied to the structure with as little vertical load action as possible and as much horizontal load action. Analyzing in this way, the structure will be in the worst situation, which would offer the possibility of a conservative design but also higher safety for the structure. Ultimate Limite State combinations are retrieved from the National Annex of Finland (Table 6). In variety, it can be seen that both adverse and favorable actions are taken into account. If this action co-occurs, they must be considered both in the equation; otherwise, if we have only one of them, dead load action and snow load action create unfavorable action. Favorable action can be neglected on the given equations. The equation also shows a K_{FI} coefficient, which depends on the reliability class of the building. This factor is crucial as it considers the consequences of human losses and economic damage in the event of a casualty. Depending on the size of the project, the consequence class of the building can be determined and used respectively on equations 6.10a and 6.10b

Table 6. Design value of actions (Ministry of the Environment, 2007).

Persisten and transient design situations	Permanent actions		Leading variable action (*)	Accompanying varibale actions (*)	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10a)	$1.35 \cdot K_{FI} \cdot G_{kj.sup}$	$0.9 \cdot G_{kj.inf}$			
(Eq. 6.10b)	$1.15 \cdot K_{FI} \cdot G_{kj.sup}$	$0.9 \cdot G_{kj.inf}$	$1.5 \cdot K_{FI} \cdot Q_{k.1}$		$1.5 \cdot K_{FI} \cdot \psi_{0.i} \cdot Q_{k.i}$
(*) 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: $1.15 \cdot K_{FI} \cdot G_{kj.sup} + 0.9 \cdot G_{kj.inf} + 1.5 \cdot K_{FI} \cdot Q_{k.1} + 1.5 \cdot K_{FI} \cdot \sum\psi_{0.i} \cdot Q_{k.i}$ $1.35 \cdot K_{FI} \cdot G_{kj.sup} + 0.9 \cdot G_{kj.if}$					

10.1.1 Data collection of the building's consequent class

According to the Eurocode based on structural design, buildings are divided into different categories regarding importance from life losses, economic damage, and social and human impact. Therefore in the National Annex of Finland, the consequence class of this building is also defined as class CC2 (Table 7).

Table 7. Definition of consequences classes (Ministry of the Environment, 2007).

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 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 storeys2) – 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.

The partial factor is selected from the National Annex based on the appropriate building class. As the building belongs to consequent class CC2, the partial factor will be one (Table 8).

Table 8. Partial safety factors according to consequent class.

Consquent class	K_{FI}
RC3	1.1
RC2	1
RC1	0.9

10.2 Characteristic and design loads (SLS)

To check whether a structure can perform correctly when it carries loads, the various design situations must be checked in terms of elements, following the relevant Eurocodes and the National Annex of Finland.

As in the case of the Ultimate Limit State, the combination of loads with different factors is the same for Serviceability Limit State. In this load combination, the results approximate reality and are related to vibrations, creep, deformation, and various cracks in the elements.

Characteristic combination

$$\sum_{j\geq 1} G_{k.j}+P+\psi_{1.1}\cdot Q_{k.1}+\sum_{i>1}\psi_{2.\,i}\cdot Q_{k.1}$$

Frequent combination

$$\sum_{j\geq 1} G_{k.j}+P+Q_{k.1}+\sum_{i>1}\psi_{0.\,i}\cdot Q_{k.i}$$

Quasi-permanent combination

$$\sum_{j\geq 1} G_{k.j}+P+\sum_{i>1}\psi_{2.\,i}\cdot Q_{k.i}$$

Where:

- $G_{k.j}$ Dead loads.
- $Q_{k.i}$ Other variable loads.
- ψ Coefficient for buildings.

The load combination considers prestressing load classified as actions caused by controlled forces or controlled deformations (Commitee, 2002).

The respective coefficients must be taken into account from the combination of loads according to the Eurocode in the formulas for the accompanying variables (Table 9). The data presented in the table below are selected based on the type of load and the type of building.

For this project designed in this document, the building belongs to category A, of which residential buildings are part. For the snow load, according to the calculations made, the value of snow is less than 2.5 kN/m^2. Furthermore, coefficients will be selected from the table below, respectively. In addition, for wind loads, the table below can be used to determine the required coefficients. For Ultimate Limit State ψ_0 the factor is used. Contrastingly, in Serviceability Limit State, the other coefficients will be used respectively, where ψ_1 and ψ_2 are used for the frequent and quasi-permanent combination.

Table 9. Values of ψ factors for buildings.

Load	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A: areas in residential buildings	0.7	0.5	0.3
Category B: office areas	0.7	0.5	0.3
Category C: congregation areas	0.7	0.7	0.3
Category D: shopping areas	0.7	0.7	0.6
Category E: storage areas	1	0.9	0.8
Category F: traffic area, vehicle weight ≤ 30 kN	0.7	0.7	0.6
Category G: traffic area, 30 kN < vehicle weight ≤ 160kN	0.7	0.5	0.3
Category H: roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*), when			
s_k<2.75 kN/m^2	0.7	0.4	0.2
s_k<≥2.75 kN/m^2	0.7	0.5	0.3
Ice loads **)	0.7	0.3	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

*) Outdoor terraces and balconies ψ_0 = 0 combined with categories A, B, F and G.
Note: In case there are different categories of loads in one building, which cannot clearly
be separated into different sections, values for ψ factors giving the most unfavourable effect should be used.
**) Added in the Finnish National Annex

11 Data collection of structural material

This two-apartment house will be a combination of timber and concrete materials. The client chooses the class of wood and concrete materials. All properties of these materials are obtained from the relevant Eurocodes, and structural verifications can be done.

11.1 Structural timber selection

Timber material can be divided into different categories starting from the type of wood, size, and other factors. In this project, the timber used will be taken from the land where this project will be built. The land consists of trees that will be cut and then appropriately treated in the factory to provide control and ensure that the timber will perform correctly when the load has been applied (Figures 19 and 20). So, the type of wood for this project will be sawn timber that belongs to the softwoods category, and the class will be C24.

Figure 19. Plot view during summer season.

Figure 20. Plot view during winter season.

11.1.1 Softwood C24 timber properties

This information is essential because, through the selection of timber material, it becomes possible to follow the relevant Eurocode to get all the properties of timber that will be needed when checking the various structures of the building. As mentioned above, the timber class is C24 which belongs to the softwood timber class. To get all the data regarding strength properties, stiffness properties, and density of this timber, SFS-EN 338 Structural timber - Strength classes are followed (Table 10).

Table 10. Strength classes for softwood based on edgewise bending tests – strength, stiffness and density values (CEN/TC124, 2016).

		Softwood species											
		C14	C16	C18	C20	C22	C24	C27	C30	C35	C40	C45	C50
Strength properties (in N/mm^2)													
Bending	$f_{m.k}$	14	16	18	20	22	24	27	30	35	40	45	50
Tension parallel	$f_{t.0.k}$	8	10	11	12	13	14	16	18	21	24	27	30
Tension perpendicular	$f_{t.90.k}$	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.1	0.4	0.4	0.4
Compression parallel	$f_{c.0.k}$	16	17	18	19	20	21	22	23	25	26	27	29
Compression perpendicular	$f_{c.90.k}$	2	2.2	2.2	2.3	2.4	2.5	2.6	2.7	2.8	2.9	3.1	3.2
Shear	$f_{v.k}$	3	3.2	3.4	3.6	3.8	4	4	4	4	4	4	4
Stiffness properties (in kN/mm^2)													
Mean modulus of elasticity parallel	$E_{0.mean}$	7	8	9	9.5	10	11	12	12	13	14	15	16
5% modulus of elasticity parallel	$E_{0.05}$	4.7	5.4	6	6.4	6.7	7.4	7.7	8	8.7	9.4	10	11
Mean modulus of elasticity perpendicular	$E_{90.mean}$	0.2	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.5	0.5	0.5
Mean shear modulus	G_{mean}	0.4	0.5	0.6	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1
Density (in kg/m^3)													
Denisity	ρ_k	290	310	320	33	340	350	370	380	400	420	440	460
Mean density	ρ_{mean}	350	370	380	390	410	420	450	460	480	500	520	550
NOTE 1 Values given above for tension strength, compression strength, shear strength, 5% modulus of elasticity, mean modulus of elasticity perpendicular to grain and mean shear modulus, ave been calculated using the equations given in Annex A. NOTE 2 The tabulates properties are compatible with timber at a moisture content consisten with a temperature of 20 degress celsius and a relative humidity of 65 %. NOTE 3 Timber conforming to classes C45 and C50 may not be readily available. NOTE 4 Characteristic values for shear strength are given from timver without fissuers, according to EN 408. The effect of fissures should be covered in design codes.													

11.2 Concrete C25/30 properties

Concrete will be the material used to build the foundation of the building. In addition, all floors will be made of concrete. To perform the necessary checks regarding the design of the foundations and floors, data on the properties of the concrete class to be used must be obtained in advance. Based on the formulas provided in SFS-EN 1992-1-1, the following table is obtained.

Note! The value for α_{cc} coefficient can be taken 1 in cases when their value is not specified in the National Annex. Contrary, the value of this coefficient is set in the National Annex of

Finland; therefore, the result must be obtained by using the coefficient specified there. The value of the design compressive strength of concrete must be calculated by taking the value of α_{cc} as 0.85.

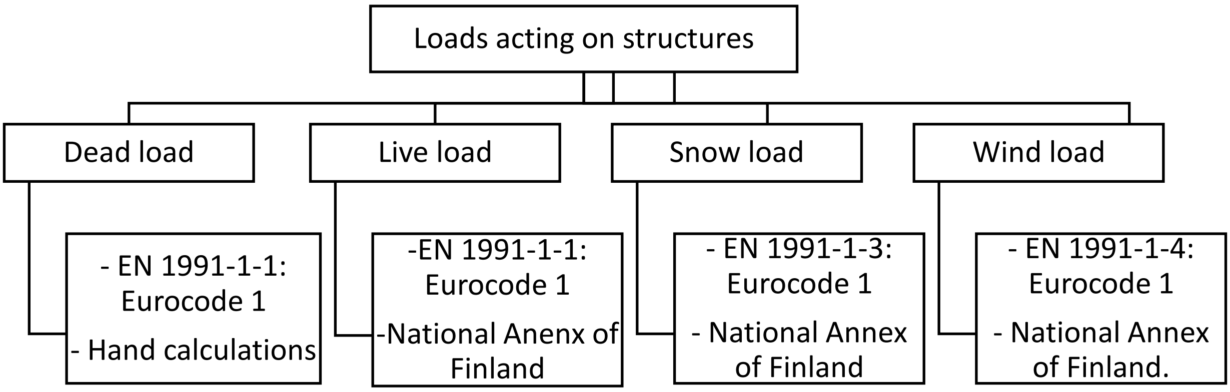
Table 11. Concrete Design Properties according to EN1992-1-1 (EurocodeApplied.com, n.d.).

Symbol	Description	C12/15	C16/20	C20/25	C25/30	C30/37
f_{ck} (MPa)	Characteristic cylinder compressive strength	12	16	20	25	30
$f_{ck,cube}$ (MPa)	Characteristic cube compressive strength	15	20	25	30	37
f_{cm} (MPa)	Mean cylinder compressive strength	20	24	28	33	38
f_{ctm} (MPa)	Mean tensile strength	1.57	1.9	2.21	2.56	2.9
E_{cm} (MPa)	Elastic modulus	27085	28608	29962	31476	32837
f_{cd} (MPa) (for α_{cc}=1.00)	Design compressive strength (for α_{cc}=1.00)	8	10.67	13.33	16.67	20
f_{cd} (MPa) (for α_{cc}=0.85)	Design compressive strength (for α_{cc}=0.85)	6.8	9.07	11.33	14.17	17
f_{ctd} (MPa) (for α_{ct}=1.00)	Design tensile strength (for α_{ct}=1.00)	0.73	0.89	1.03	1.2	1.35
ρ_{min} (%)	Minimum longitudinal tension reinforcement ratio	0.13	0.13	0.13	0.133	0.151
$\rho_{w,min}$ (%)	Minimum shear reinforcement ratio	0.055	0.064	0.072	0.08	0.088

12 Analyses and data collection of load acting on structures

In structure, different loads make it possible for the behavior of the structure to vary over time and in specific moments to behave in different ways. Consequently, the loads going to the structure must be calculated according to the relevant Eurocode and the National Annex of Finland (Figure 21). For this building, the loads that will act on the structure will be dead, live and wind loads.

Figure 21. Reference documents to define loads on structures.



12.1 Dead loads on structures

For calculating the dead load that comes to the upper structure of the building on the roof, the load is calculated from the weight of all materials that will be above the truss, calculating the weight of the truss itself also. To provide a safe design, weights of different elements are taken on a conservative side.

To calculate the self-weight of the truss, the dimensions and the density of the timber material can be used. On the contrary, a conservative method to calculate truss weight can be done using the following formula.

$$Truss_{weight}=\left(\frac{L_{truss}}{3}+5\right)$$

Where:

L_{truss} – the span of the roof truss.

The weight of the truss is calculated, taking into account the longest part of the span of the building. This value is used in all calculations, and it is assumed that this load is applied to the wall structure. This shows the conservative way of calculating loads which offers a safer design.

The calculations are done in detail for the values presented in the table below, considering the materials' density and dimensions (Appendix 1). The below table shows the values of the characteristic loads and not the design values.

Note! 30% of the total dead load is considered a conservative value for loads on the eaves side.

Table 12. Dead loads.

Material	Characteristic weight	Unit
Bitumen felt	0.001	kN/m^2
Roof wood board	0.097	kN/m^2
Truss element	0.09	kN/m^2
Insulation	0.15	kN/m^2
Ceiling wood board	0.095	kN/m^2
Ceiling frame	0.25	kN/m^2
Summation	0.683	kN/m^2

It is necessary to use helping drawings to design different structures, as shown below (Figure 22). This offers the opportunity to understand better how the load is distributed in other structures based on which the checking is done and the structure's design.

Figure 22. Dead load distribution.

Note! In the upper part of the roof, loads fall into the live load. In SFS EN-1991-1-1, roofs are divided into different categories. Based on this Eurocode roof belonging to the H category, roofs are not accessible except for regular maintenance and repair (Commitee, 2002). Specifications for the live load value for this category are given in the National Annex of Finland (Table 13). This load can be ignored for the building being designed; it is enough to calculate dead loads on the roof for a residential installation. Usually, this category belongs to tops, which have heavy equipment, such as ventilation machines, solar panels, and others, which may need maintenance.

Table 13. Imposed loads on roofs of category H.

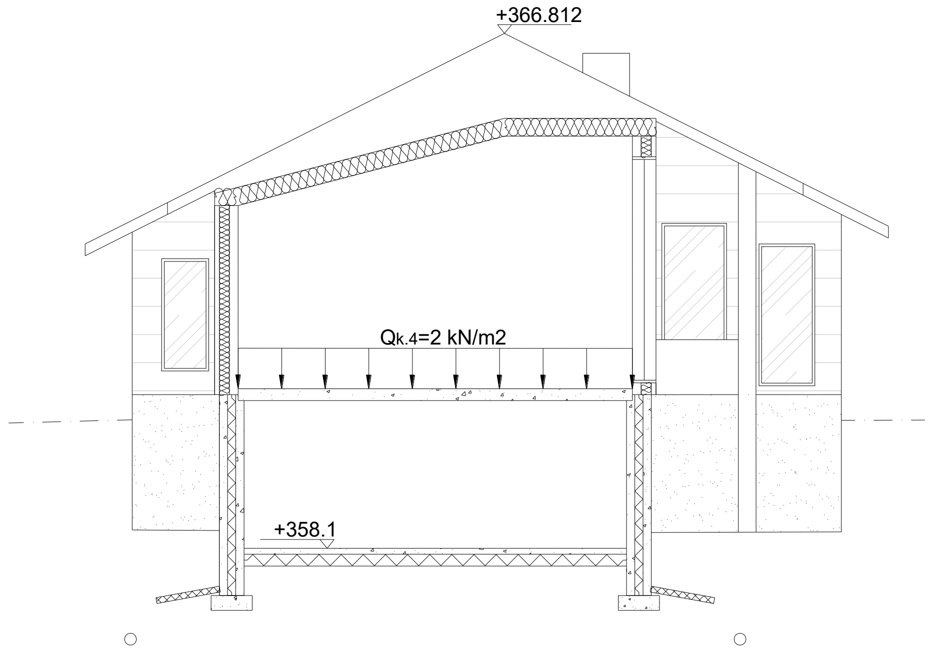
Roof	q_k [kN/m^2]	Q_k [kN]
Category H	0.4	1
Note: q_k may be assumed to act on an area not greater than 10 m^2.		

12.2 Live load on structures

Determining live load is essential in cases where a structure will be designed in which a load moves in most cases. The category of live load includes people, movable furniture, and others. Consequently, these load actions must be considered for different structures to be checked and designed safely.

A live load will be used in the concrete slab design for the first floor of this building. SFS-EN 1991-1-1 Action on Structure provides the values of imposed load on the floor if no specification is mentioned in the National Annex of the country. As mentioned in previous chapters, this building will be used for residential purposes and belongs to category A. Therefore, the live load will be selected by the National Annex of Finland. Live load is applied in all parts of the building, but this value will be used in the concrete slab design for the first floor, below which is the basement (Figure 23). This is because this concrete slab will be more critical. When the critical concrete slab is designed, this type of slab can be used in other building parts. The figure below shows the live load distribution only for the critical slab designed according to the relevant Eurocode and the National Annex of Finland.

Figure 23. Live load applied to the slab considered for design.



Imposed live load for the floor structures is taken from the National Annex of Finland (Table 14).

Table 14. Imposed loads on floors, balconies and stairs in buildings (Ministry of the Environment, 2007).

Categories of laoded areas	q_k [kN/m^2]	Q_k [kN]
Category A		
Floors	2	2
Stairs	2	2
Balconies	2.5	2
Category B	2.5	2
Category C		
C1	2.5	3
C2	3	3
C3	4	4
C4	5	4
C5	6	4
Category D		
D1	4	4
D2	5	7

12.3 Snow load on structures

Finland is a country with long winters accompanied by heavy snowfall. A layer created during snowfall is a load that the roof structure must support and transfer safely to other building elements. SFS EN-1991-1-3 Snow load is used to determine snow load. Based on this Eurocode, the snow load is calculated according to the following formula:

$$s=\mu_i \cdot C_e \cdot C_t \cdot s_k$$

Where:

μ_i - is the snow load shape coefficient.

C_e - is the exposure coefficient.

C_t - is the thermal coefficient.

s_k- is the characteristic value of snow load on the ground.

The above coefficients are determined by the relevant Eurocode and the National Annex of Finland. The snow load coefficient for this building is 0.8 based on the roof's slope. The roof slope is 1: 2; therefore, the angle can be calculated.

$\alpha = \tan^{-1}\left(\frac{1}{2}\right) = 26.565°$

The snow load shape coefficient value is taken from the table provided in SFS EN-1991-1-3 (Table 15).

Table 15. Snow load shape coefficients.

Angle of pitch of roof α	$0° \leq \alpha \leq 30°$	$30° < \alpha < 60°$	$\alpha \geq 60°$
$\mu_1(\alpha)$	$\mu_1(0°) \geq 0.8$	$\mu_1(0°) \cdot \frac{(60° - \alpha)}{30°}$	0
$\mu_2(\alpha)$	0.8	$0.8 \cdot \frac{(60° - \alpha)}{30°}$	0
$\mu_3(\alpha)$	$0.8 + 0.8 \cdot \alpha / 30°$	0.6	-

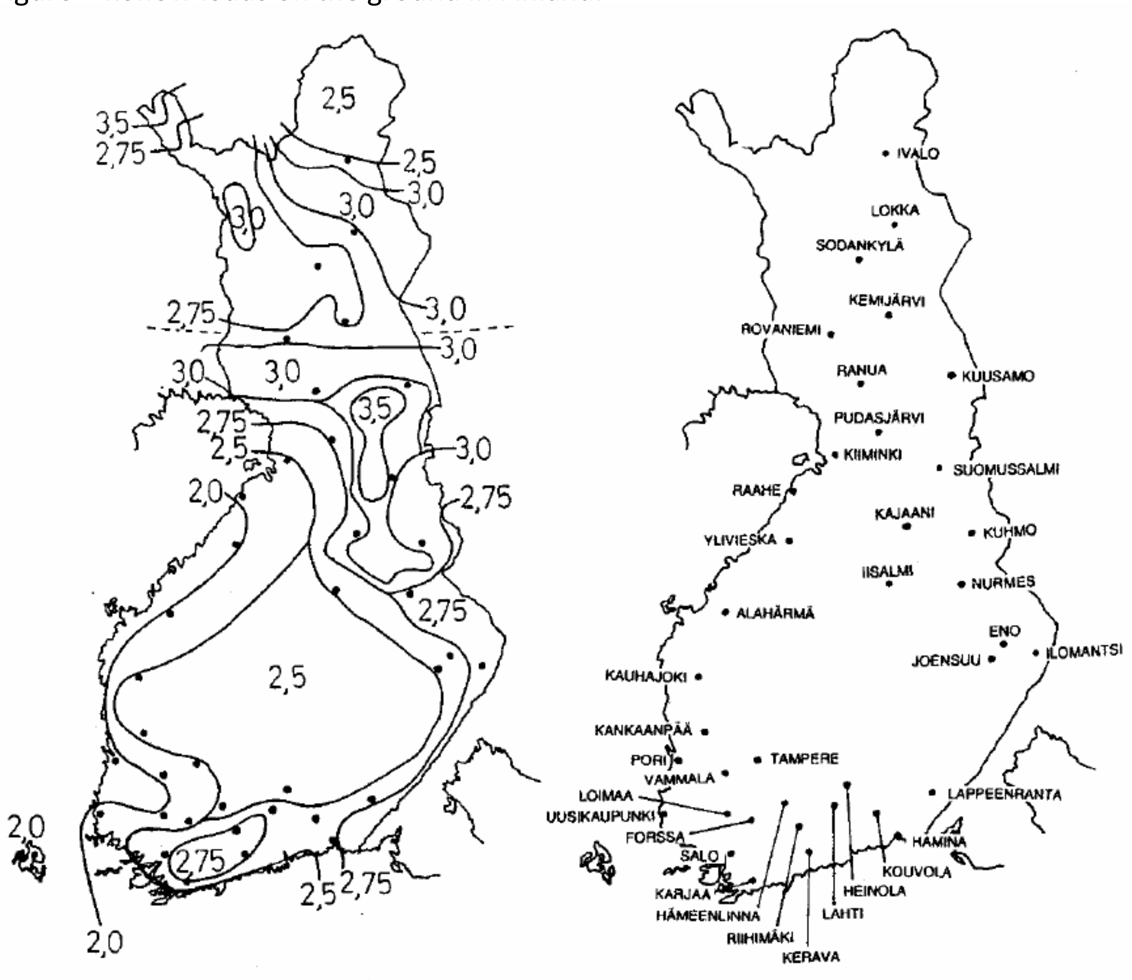
The exposure coefficient is taken as one. On the other hand, the thermal coefficient is taken as one as the topography of the place is typical and includes areas where there is no effective snow removal by the wind, as trees cover the terrain and prevent the removal of snow (Commitee, 2002).

Table 16. Recommended values of C_e for different topographies (Commitee, 2002).

Topography	C_e
Windswept[a]	0.8
Normal[b]	1
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	

To find the characteristic value of snow load on the ground, the National Annex of Finland is used (Figure 24). The density of snow depends on the geographical position of the building, as in the northern part of Finland, the density of snow is on average, higher. This building is located in Kolari, in the north part of Finland, where the snow load is high. Since the roof will be a duo-pitched roof, according to the Eurocode, coefficients are determined in relation to the distribution of snow load on the roof (See appendix 2). To be as conservative as possible in design, it is assumed that the entire snow load is evenly distributed throughout the roof.

Figure 24.Snow loads on the ground in Finland.



Snow load on the ground for this building is between 2.75 kN/m^2 to 3 kN/m^2. The National Annex of Finland specifies that in cases where the building to be constructed is located in an area where the value of snow load on the ground is not constant and then linear interpolation can be used in proportion to distances from the closest curves (Ministry of the Environment, 2007).

For this project to be conservative, a more significant value is used. Therefore snow load on ground level for this project is considered as 3 kN/m^2.

In conclusion, the characteristic snow load to be used for this building is 2.4 kN/m^2, substituting variables into the corresponding formula.

12.4 Wind load on structures

Timber is a lightweight material, and consequently, one of the most common problems observed in timber structures is lateral structure stability. When the structure is under wind load pressure, this can cause discomfort and risk to the persons occupying the building. Therefore, to provide lateral stability to the building bracing or stiffening walls are used. In most cases, one method, which is also used in this project, is the use of sheathing, which consists of the material of wood-based panels connected to the frame of the timber wall offering enough stiffness for the wind coming from different directions.

To calculate wind load, SFS-EN 1991-1-4 Wind actions can be used. Eurocode provides in detail how the wind load should be calculated coming to the building in a different direction.

To better understand how the wind load is calculated, this document presents both methods, consisting of specific calculations according to the respective Eurocode and conservative calculations used to check and design structures. This project is not very complex, so some calculation methods are done conservatively, which offers a more secure design. This chapter shows the conservative way. Also, in this document is shown the calculation of the load according to the areas defined in Eurocode. These calculations are conducted to understand better how the exact wind load calculations are made (Appendix 5 and 6).

12.5 Simplified wind calculation method

The way buildings behave is known for buildings of this scale; therefore, assumptions can be used to calculate wind pressure. This document explains the exact calculations to understand the Eurocodes (Appendix 5 and 6).

12.5.1 Wind velocity

For the calculation of wind velocity, the following formula is used.

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

Where:

c_{dir} - directional factor.

c_{season} - seasonal factor.

$v_{b.0}$ - the basic value of wind velocity.

12.5.2 Mean wind

The building is categorized under terrain category III, as the location is in the northern part of Finland. Factors depending on the terrain category are taken from the relevant Eurocode (Table 17).

Table 17. Terrain categories and terrain parameters (CEN, SFS-EN 1991-1-4 Wind actions, 2004).

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	10
NOTE: The terrain categories are illustrated in A.1.			

To calculate the mean wind velocity, the following formula is used.

$$v_m = v_b \cdot c_0 \cdot c_r$$

Where:

v_b - wind velocity.

c_0 - orography factor.

c_r - terrain roughness factor is calculated as follows.

$$c_r = K_r \cdot \ln\left(\frac{z}{z_0}\right)$$

Where:

K_r - Terrain factor is based on the roughness length, calculated from the following formula.

$$K_r = 0.19 \cdot \left(\frac{z_0}{z_{0.II}}\right)^{0.07}$$

Where:

z_0 - Factor corresponding to terrain category III.

$z_{0.II}$ – Factor corresponding to terrain category II.

For this building mean wind value is 15.25 m/s (Appendix 3).

12.5.3 Wind turbulence

The turbulence intensity, which is the standard deviation of the turbulence divided by the mean wind velocity, is calculated from the following formula. (Eurocodeapplied.com, n.d.).

$$I_v = \frac{K_I}{c_0 \cdot \ln\left(\frac{z}{z_0}\right)}$$

Where:

K_I - Turbulence factor.

c_0 - Orography factor.

z - Reference height of the building.

z_0 - Orography factor.

For this building, turbulence intensity is 0.311 (See appendix 3).

12.5.4 Peak velocity pressure

The following formula is defined in the relevant Eurocode for calculating wind pressure. Once this value is determined, the estimates of characteristic wind pressure on the walls in relevance to their direction are performed.

$$q_p=(1+7\cdot I_v)\cdot\frac{1}{2}\cdot\rho\cdot {v_m}^2$$

Where:

I_v - Wind turbulence.

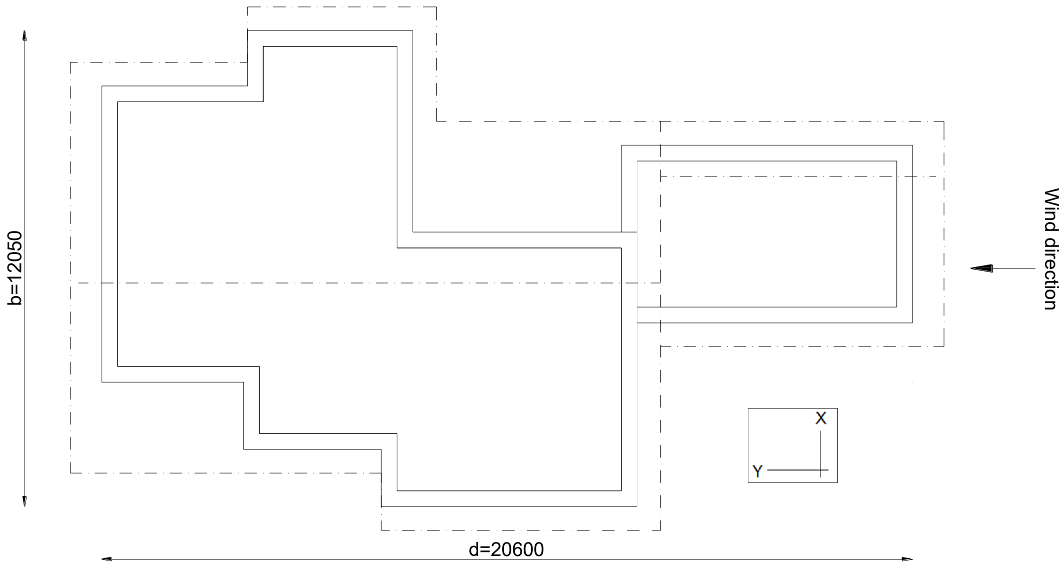
ρ - air density, given as 1.25 kN/m^3.

v_m - mean wind velocity.

12.5.5 Characteristic pressure on the wall in Y-direction

The wind comes in different directions in the structure, as it is considered a dynamic action. For this building, the wind that comes to the respective walls is calculated according to two principles. The different directions for which the wind will be considered make it possible to determine the dimensions of the structural members (Figure 25).

Figure 25. Top view of the building and wind on Y-direction.



The wind pressure value in the Y-direction is calculated with the following formula.

$F_{w.Ydirection}=c_f c_s \cdot c_d \cdot q_p$

Where:

$c_f c_s$ - a coefficient value of which, for buildings with a height less than 15 meters, may be taken as 1 (CEN, 2004).

c_f - force coefficient of structural elements of rectangular section with the wind normally blowing to a face (CEN, 2004).

q_p - peak velocity pressure.

Note! The value of the force coefficient can be determined by following the relevant Eurocode, but for the building of this scale, conservative calculations can be done based on RIL 201-1-2008. Slenderness can be calculated from the following formula.

$$\lambda_{RIL} = \frac{2 \cdot h}{b}$$

Where:

h - the height of the building.

b - the breadth of the building.

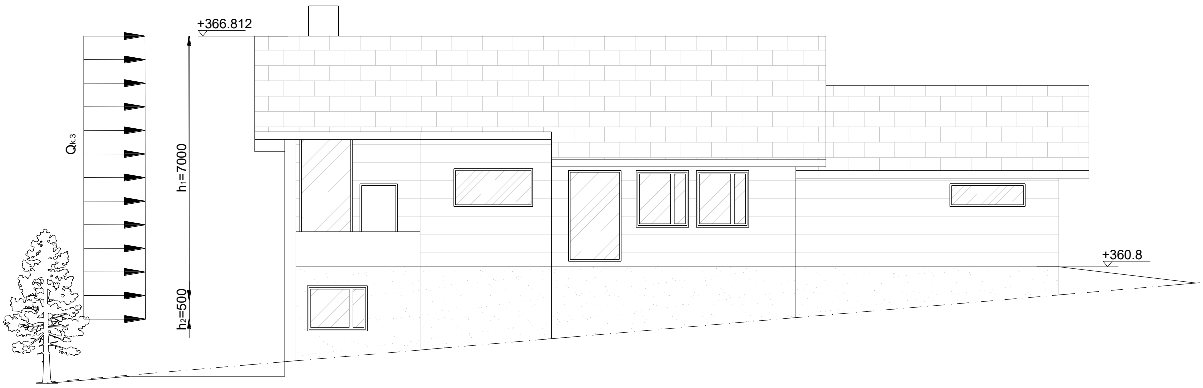
From the value obtained by the above equation, interpolation can be done to find the value of the force coefficient. The following table can be used for typical buildings to perform the interpolation.

Table 18. Values to be taken for the interpolation.

	d/b								
λ	0.1	0.2	0.5	0.7	1	2	5	10	50
≤ 1	1.2	1.2	1.37	1.44	1.28	0.99	0.6	0.54	0.54
3	1.29	1.29	1.48	1.55	1.38	1.07	0.65	0.58	0.58
10	1.4	1.4	1.6	1.68	1.49	1.15	0.7	0.63	0.63

Based on interpolation of the above value, it was determined that the force coefficient for this building would be 1.028, and the characteristic wind pressure coming to the walls in this direction would be 0.474 kN/m^2 (Appendix 3). A presentation of how wind forces will come to structures in this direction is presented below (Figure 26).

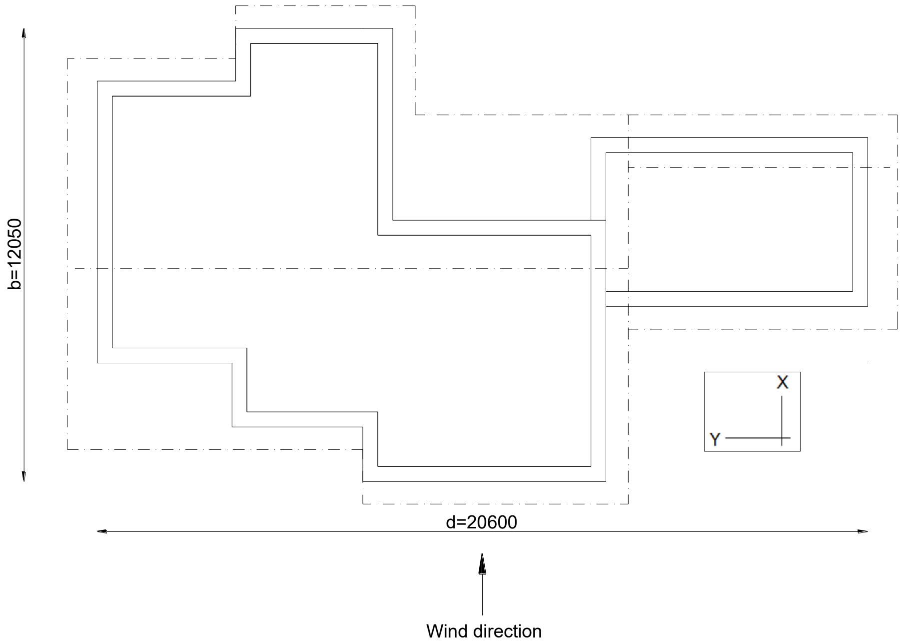
Figure 26. Wind pressure coming to the walls in Y-direction.



12.5.6 Characteristic pressure on the wall in X-direction

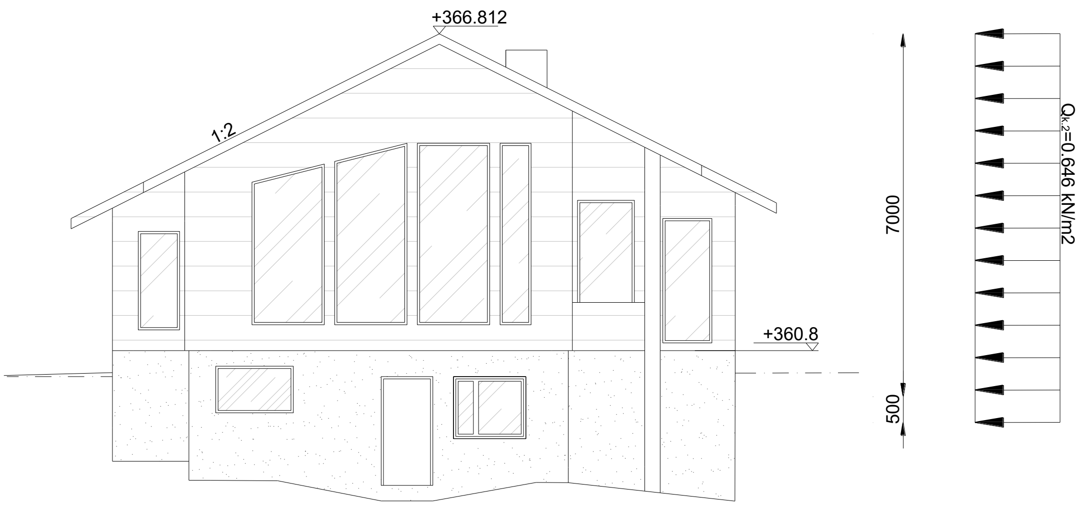
In the other direction, the same principle is used, but the only thing that should be considered differently compared to the other law is the breadth of the building (Figure 27). The wind face will change with the change of direction of wind force.

Figure 27. Top view of the building and wind on X-direction.



Therefore, by interpolation value of the force coefficient for the wind in this direction would be 1.4, and the characteristic wind pressure will be 0.646 kN/m^2 (Appendix 4). Below is shown how the wind will be distributed in this direction (Figure 28).

Figure 28. Wind pressure coming to the walls in X-direction.



12.5.7 Simplified method and exact method

The wind load calculation is done in this document because approximate assumptions can be used for projects of this size and complexity. This is not always true, as many factors affect the design process, but for this project, it was decided to use a simplified method for the following reasons:

- It is more conservative.
- It provides simplicity in the method of calculating wind load.
- Being a conservative method, it offers a higher value of wind load, which enables the design of structures more safely.

Over the years, with the advancement of engineering, books have been written which contain conservative ways of calculating to make the design process more accessible and safer. The application of the conservative method is based on the application of the rules, which are defined in the book Design criteria and structural loads Eurocode, RIL 201-1-2008. Contrastingly, the exact way of calculating the wind load is based on exact calculations where the division is made in different areas for the walls of the building. External pressure coefficients are divided into two parts which include, $C_{pe.1}$ and $C_{pe.10}$. If the analysis takes a tiny element of a structure, for example, roof fixing should be considered the values of $C_{pe.1}$. For study reasons and practice, this document is shown the solution of both ways. The simplified method is shown in Appendices 3 and 4. The exact calculation method is shown in Appendices 5 and 6.

Below is the result of these two methods to show that the conservative way gives more excellent value in terms of wind load by offering a more conservative design phase (Table 19).

Table 19. Result of both wind calculation methods.

Comparission of wind methods			
Calculation method	Direction	Value	Unit
Simplfied method	X	0.646	kN/m^2
	Y	0.474	kN/m^2
Exact method	X		
	Zone D	0.4819	kN/m^2
	Zone E	0.3067	kN/m^2
	Zone A	0.6133	kN/m^2
	Zone B	0.4381	kN/m^2
	Y		
	Zone D	0.4866	kN/m^2
	Zone E	-0.907	kN/m^2
	Zone A	-0.6194	kN/m^2
	Zone B	-0.4424	kN/m^2
	Zone C	-0.3907	kN/m^2

12.6 Summary of characteristic loads

To start the design procedures, all the characteristic loads must be defined. Once these characteristic loads are found, they can then be used in the right combinations to start the design phase of the structural elements. All the characteristic loads that come into the structure will also be used to design the different elements of the building (Table 20).

Table 20. Loads acting of the building structures.

Symbol	Load description	Value	Unit
$G_{k.1}$	Characteristic dead load from the roof	0.683	kN/m^2
$G_{k.2}$	Characteristic dead load at the upper eaves	0.2	kN/m^2
$G_{k.1}+G_{k.2}$	Sum of dead loads	0.883	kN/m^2
$Q_{k.1}$	Characteristic snow load.	2.4	kN/m^2
$Q_{k.2}$	Characteristic wind load on X-direction	0.646	kN/m^2
$Q_{k.3}$	Characteristic wind load on Y-direction	0.474	kN/m^2

13 Data collection and analyses of the stud design process

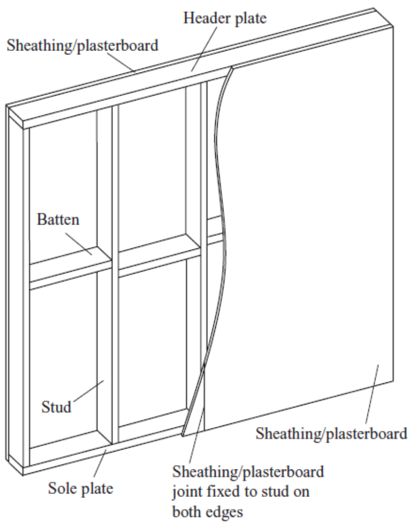
The exterior walls of the building will be the primary structure that will take the load from the roof, which will then be transferred to the foundation. To enable all loads to be safely transferred to the foundation, the wall structure must meet all the conditions specified in SFS-EN 1995-1-1 Eurocode 5, Design of Timber structures. This document will show the calculations and checks of wall structures in the Ultimate Limit State.

13.1 Load-bearing walls

Walls are subject to axial stress as well as the combination of axial stress and bending stress as a result of the effect of out of plan action; whereas an example, the wind load can be mentioned; it should be possible for this structure to be calculated following the relevant Eurocode as well as the National Annex of Finland.

In this project, the external wall takes the load coming from the roof, and consequently, these walls are included in the category of load-bearing walls. The wall is composed of vertical elements separated from each other at equal intervals, whereas in this project, the distance of the studs is 600 millimeters. These vertical elements are secured at their end by continuous members, which are called header and sole plates (Figure 29). The vertical members that receive the load are studs and are referred to as stud walls.

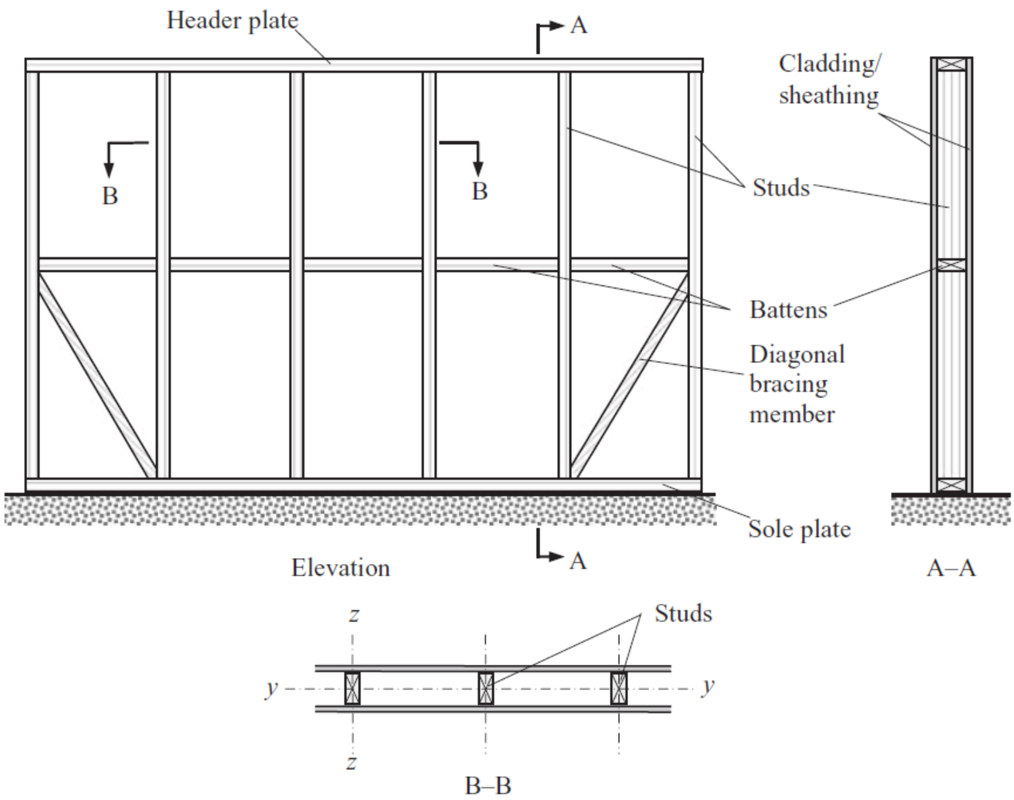
Figure 29. Structure of a timber wall (Porteous & Kermani, 2007).



The stronger axis (y-y) is parallel to the face of the wall, and as mentioned above, they are held in their proper positions by the header and sole plates. In cases where buckling phenomena may occur, secondary elements called battens are used to give more stiffness to the structure so that lateral buckling does not occur. These secondary elements can be used when the sheathing material cannot provide sufficient lateral stiffness around the weaker z-axis. In this project, the sheathing material provides sufficient lateral support, and as a result, the risk of the studs buckling along the weak axis is neglected.

Regarding the strong axis (y-y), the stud elements are fixed by fixings in the header and soleplate, but these elements rotate freely in these positions. Therefore, in this case, the effective length of the element to be considered according to the strong axis (y-y) is the same as the length of the element itself (Porteous & Kermani, 2007).

Figure 30. Typical stud wall construction (Porteous & Kermani, 2007).



13.1.1 Design of studs subjected to axial compression

For the design of studs under the axial force, it is essential to understand the fact that in the weak axis, sheathing material provides enough lateral stiffness, and this ensures that the value of the buckling deduction coefficient that takes into account the buckling effect, in the weak axis ($k_{c.z}$), will always be greater than ($k_{c.y}$). Therefore the condition to check the stud element under axial compression must be:

$\sigma_{c.0.d} \leq k_{c.y} \cdot f_{c.0.d}$

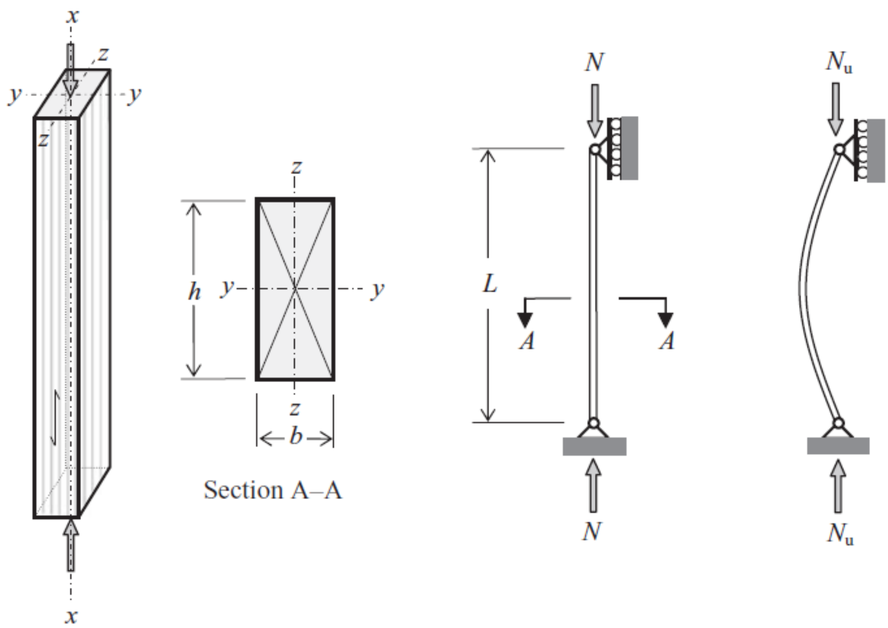
Where:

$\sigma_{c.0.d}$ - Compressive stress along the grain.

$k_{c.y}$ - Buckling deduction coefficients to consider the buckling effect.

$f_{c.0.d}$ - Design compressive strength along the grain.

Figure 31. Axially loaded stud.



13.1.2 Studs subjected to combined out of plane bending and axial compression

Although load-bearing walls will take the load vertically, causing axial compression in these elements, the wind load will cause bending. The wind load comes in four different directions; as a result, the stud element will be under bending, so it must be checked if the element manages to withstand this load in a combined way. To control this clause, the conditions will depend on the value of the slenderness ratio (λ_y, λ_z) and the relative slenderness ratio for bending ($\lambda_{rel.m}$). For this project, it is mentioned that in the weak axis, sheathing material provides enough lateral support; therefore, the equation considering λ_z will be neglected, and the check will be followed for the other conditions.

In cases when λ_y>0.3, and $\lambda_{rel.m}$≤0.75 the following condition must be checked:

$$\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + \frac{\sigma_{m.y.d}}{f_{m.y.d}} \leq 1$$

Where:

$\sigma_{c.0.d}$ - Compressive stress along the grain.

$k_{c.y}$ - Buckling deduction coefficients to consider the buckling effect.

$f_{c.0.d}$ - Design compressive strength along the grain.

$\sigma_{m.y.d}$ - Bending stress about the y-y axis of the stud.

$\sigma_{m.y.d}$ - Design bending stress about the y-y axis of the stud.

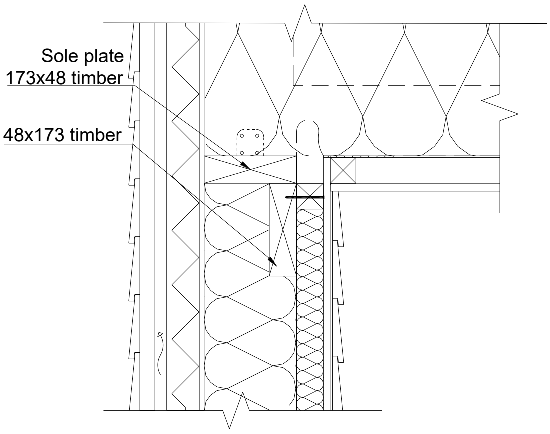
The explanation of these formulas in detail is attached at the end of the document, where the calculations performed following Eurocode are shown precisely (Appendix 7).

13.1.3 Header and sole plate design

Header and sole plates are horizontal elements placed in the lower base, usually in the foundation wall and in the upper part of the wall structure where the track elements will be placed. Since these horizontal elements will be under the influence of vertical load, they must be checked with the following equation to ensure proper performance according to the relevant Eurocode.

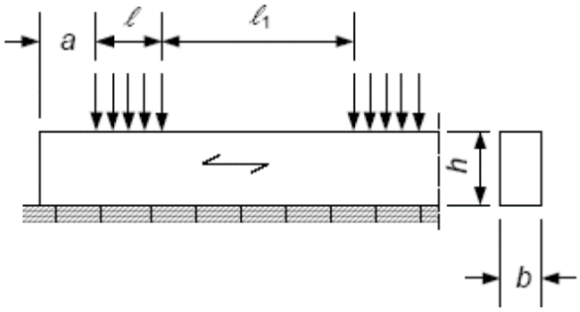
Fixings that will be used for the studs and the plates will provide sufficient lateral restraint, and the design condition to be followed corresponds to the checking of the strength of these plates under compression perpendicular to the grain. When the header plate is loaded not at the point of support with studs, the element must be checked for shear and bending. This type of load for sole plates occurs when the distance between the elements of the trusses and those of the studs is not the same, causing weak parts of the sole plates to have axial force, which will cause bending. In this case, the soleplate does not perform well in bending as in vertical forces; this bending moment will be applied to the weak axis of the soleplate, and in this way, the element fails to receive the load. In these cases, the solution offered is to use timber with the exact dimensions as the soleplate but installed vertically so that the bending moment acts about a stronger axis (Figure 32).

Figure 32. Sole plate connection.



Therefore, the header and sole plates will act as continuous members, and the following case applies (Figure 32).

Figure 33. Series of loads acting on header and sole plates (CEN/TC250, 2015).



The following equation is used to verify whether the header and sole plate have sufficient capacity to withstand the loads.

$$\frac{\sigma_{c.90.d}}{k_{c.90} \cdot f_{c.90.d}} \leq 1$$

Where:

$\sigma_{c.90.d}$ - Design compressive stress along the grain.

$k_{c.90}$ - Factor taking into account the load configuration (Finnish Association of Civil Engineers, 2009).

$f_{c.90.d}$ - Design compressive strength along the grain.

13.1.4 Design of beam elements

There will also be elements that act as a beam in this building. In this project, the use of the Kerto S-beam will take place as the window opening is considerable. This document will show the design of this type of beam for the most critical window opening. The same design will be used on the not very critical parts by designing the most critical element.

The element at the top of the opening that, in this case, will be the beam must be verified for the bending and shear. Lateral torsional buckling is not a phenomenon in this case as the Kerto S-beam at the compression edge would be connected by a nail to the bottom element, preventing this phenomenon from happening.

The rules defined in SFS-EN 1995-1-1 Eurocode 5: Design of Timber Structures apply to check the beam for bending.

$\sigma_{m.y.d} \leq f_{m.y.d}$

Where:

$\sigma_{m.y.d}$ - is the design bending stress.

$f_{m.y.d}$ - is the design bending stress.

The beam must be verified for the shear accordingly following the same Eurocode. The following expression must be satisfied.

$\tau_d \leq f_{v.d}$

Where:

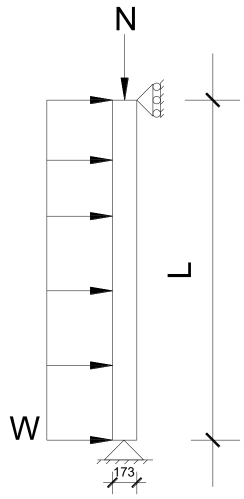
τ_d - is the design shear stress.

$f_{v.d}$ - is the design shear strength.

13.2 Verification procedure of stud elements

Designing wooden elements involves determining the modification factor, which depends on the shortest load action. Therefore, the whole load combination must be considered during the verification of wooden structures. Then the modification coefficient will be taken respectively for the shortest term load action, where the verifications defined in the relevant Eurocode will be followed. This document provides the table with the selection of the modification factor for the shortest load term duration in each load combination. The respective appendix shows the design for the most critical load combination only. For the correct calculation of the stud element, an analytical model is provided to understand better the load acting on the member (Figure 34).

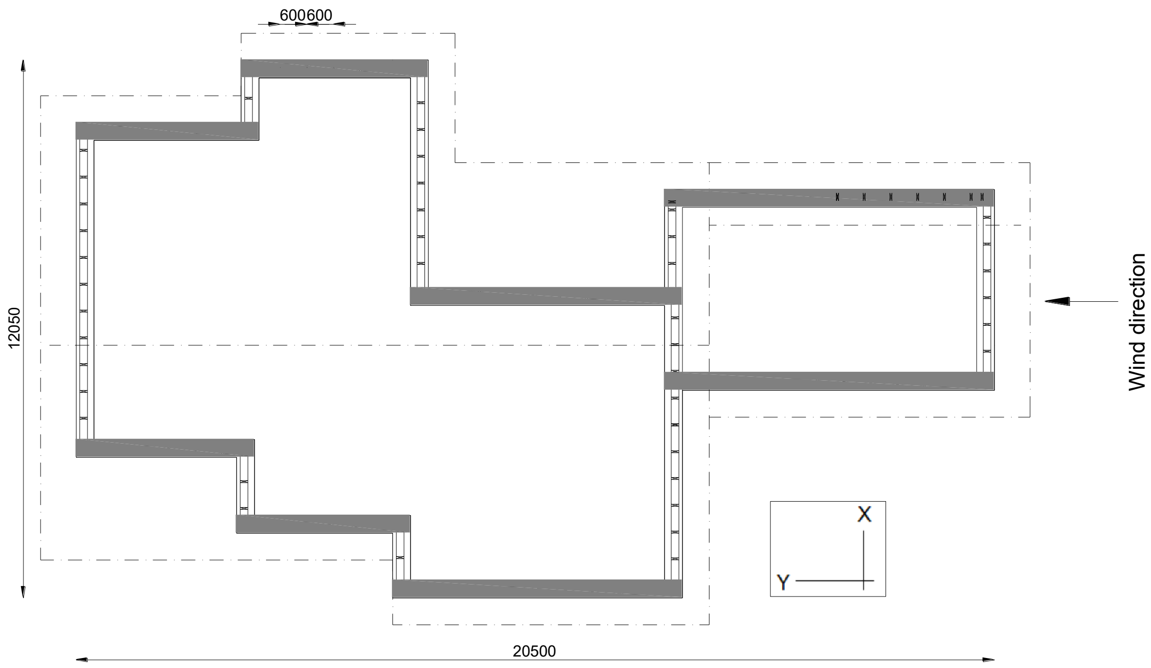
Figure 34. Analytical model of the stud element.



13.3 Stud design on Y-direction

The wind blows from different directions, and the studs considered controlled lie in the long part of the building (Figure 35).

Figure 35. Wall stud being considered for design and checking.



Since not all loads come simultaneously in the structure, this must be considered in checking the above formulas as the modification factor depends on the combination of different loads. If several loads are taken together, the determination of the modification factor will be done by the shortest load term duration (Table 21). Therefore, all combinations based on the shortest load term must be selected as modification factors.

Table 21. Result of ULS combination and modification factor.

			Vertical load+ Wind load			
Action	Permanent	Permanent+medium	Permanent+medium+short	Permanent+short	Permanent+medium+short	Unit
	Combination 1	Combination 2	Combination 3	Combination 4	Combination 5	
N_{ed}	4.309	16.685	16.685	3.671	12.781	kN
M_{Ed}	0	0	0.318	0.454	0.454	kN*m
k_{mod}	0.6	0.8	0.9	0.9	0.9	

Note! The methodology and detailed calculations are explained at the end of this document (Appendix 7).

13.3.1 Results of axial and combined actions on Y-direction

Checking is done by following the rules provided by the relevant Eurocode; the stud for this case would have enough capacity to withstand the loads (Table 22).

Table 22. Result of axial and combined actions on the stud elements.

Load combination	Axial compression	Axial compression and bending
	$UR_{StudCompression}$	$UR_{combined}$
Combination 1	7%	7%
Combination 2	20%	20%
Combination 3	17%	26%
Combination 4	4%	16%
Combination 5	13%	25%

13.3.2 Results of header and sole plates on Y-direction

Header and sole plates are checking for the compression perpendicular to the grain, and the following results were obtained (Table 23).

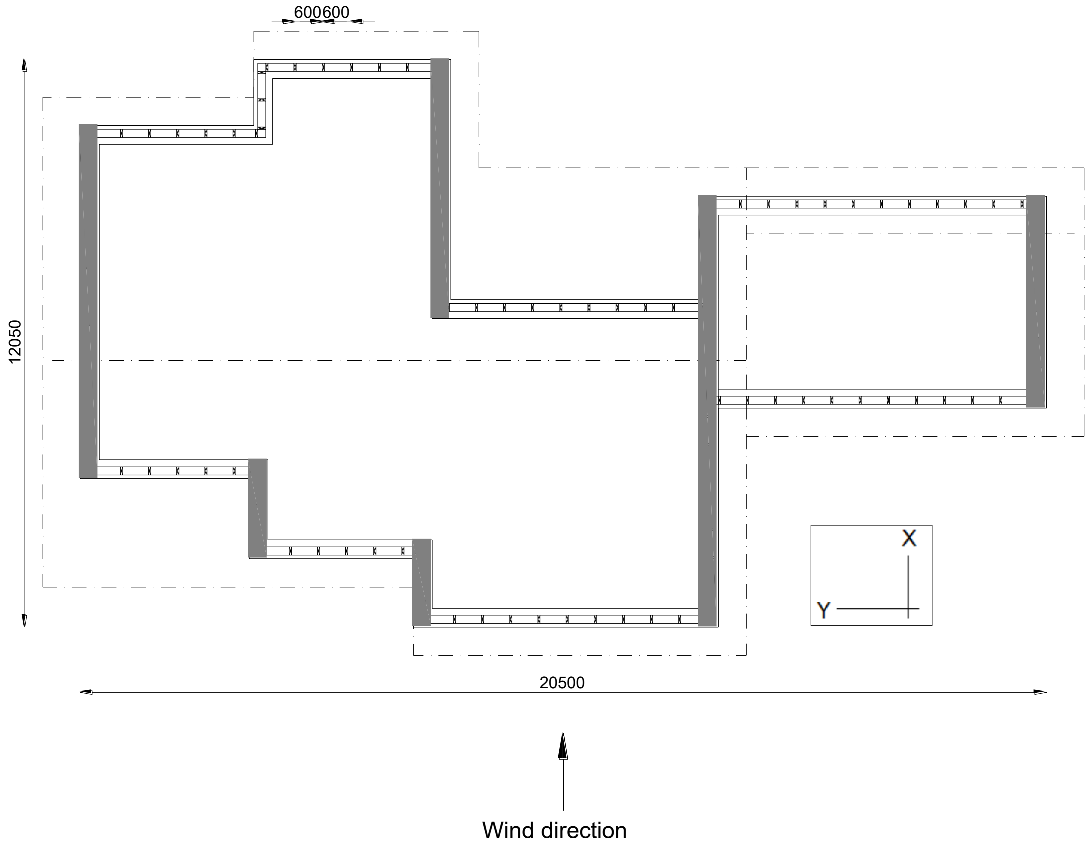
Table 23. Results of header and sole plates.

Load combination	Header plate	Sole plate
	$UR_{BearingCapacity1}$	$UR_{BearingCapacity2}$
Combination 1	22%	22%
Combination 2	64%	64%
Combination 3	57%	57%
Combination 4	13%	13%
Combination 5	44%	44%

13.4 Stud design on X-direction

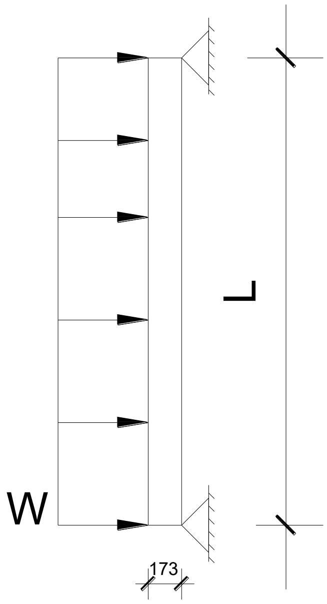
The wind blows from different directions, and the studs considered controlled lie in the short part of the building (Figure 36).

Figure 36. Wall stud being considered for design and checking.



As the studs on the direction will not be the primary structure for taking the vertical load is safe to assume the normal force acting on them is negligible. As a result, the studs will be considered only under the wind load, which would cause only a bending moment. In this case, the stud element behaves as a beam under the bending moment. An analytical model of the stud element is provided below (Figure 37).

Figure 37. Analytical model of the stud element.



The following results have been obtained, considering the combination of loads in the Ultimate Limit State (Table 24). It is worth mentioning that stud elements are considered not to take the vertical load.

Table 24. Results of ULS combination and modification factor.

			Wind load			
Action	Permanent	Permanent+medium	Permanent+medium+short	Permanent+short	Permanent+medium+short	Unit
	Combination 1	Combination 2	Combination 3	Combination 4	Combination 5	
N_Ed	0	0	0	0	0	kN
M_Ed	0	0	0.318	0.454	0.454	kN*m
k_mod	0.6	0.8	0.9	0.9	0.9	

Note! The methodology and detailed calculations are explained at the end of this document (Appendix 8).

13.4.1 Results of axial and combined actions on X-direction

The same procedure follows as in the case of the Y-direction. Only the critical combination is checked for this case as it would be enough. Header and sole plates would not be a concern for design as the vertical load coming to them are negligible.

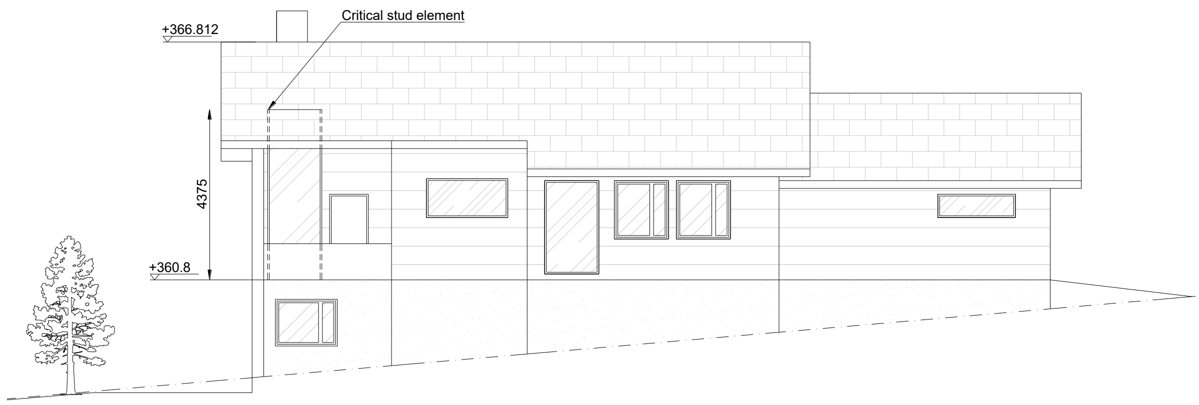
Table 25. Result for all memeber on X-direction.

Load combination	Axial compression	Axial compression and bending
	$UR_{StudCompression}$	Urcombined
Combination 1	0%	0%
Combination 2	0%	0%
Combination 3	0%	8%
Combination 4	0%	12%
Combination 5	0%	12%

13.5 Design of tall stud element in Y-direction

In this project, the level of wooden walls is not at the same level. So, in different parts of the building, the height of the walls varies due to the design of the building. In cases where structural elements, precisely stud elements, are long compared to width, the probability that the element fails due to the buckling phenomenon increases. Although sheating material provides enough resistance on the weak axis, the same can not be said for the other direction. The most critical element in the load it receives compared to its length is considered. All other less critical elements are covered as the design for the critical element is done, and there is no need for further checks for other members to be applied. Furthermore, this stud would take the load coming from the eaves with an offset of 800 millimeters.

Figure 38. Critical stud element.



Since the width of the opening in this wall will contribute to the tributary length, calculating the loads coming to this stud element must be done correctly. Below are the results of load combinations for critical stud elements (Table 26).

Table 26. Results of ULS combination and modification factor.

			Vertical load+ Wind load			
Action	Permanent	Permanent+medium	Permanent+medium+short	Permanent+short	Permanent+medium+short	Unit
	Combination 1	Combination 2	Combination 3	Combination 4	Combination 5	
N_{ed}	4.917	19.039	19.039	4.189	14.584	kN
M_{Ed}	0	0	1.1205	1.601	1.601	kN*m
k_{mod}	0.6	0.8	0.9	0.9	0.9	-

13.5.1 Results of axial and combined actions on Y-direction

An increase in utilization ratio can be seen as the length of the element has an essential role in the bearing capacity of the element itself. According to the calculations, although considerable in length, the stud element can withstand the loads (Table 27).

Table 27. Result of axial and combined actions on the stud elements.

Load combination	Axial compression	Axial compression and bending
	$UR_{StudCompression}$	$UR_{combined}$
Combination 1	16%	16%
Combination 2	45%	45%
Combination 3	40%	69%
Combination 4	9%	36%
Combination 5	31%	72%

13.5.2 Results of header and sole plates on Y-direction

The same utilization ratio is expected for the control of header and sole plates, as their calculation does not depend on the vertical element's length but the horizontal element.

Table 28. Results of header and sole plates.

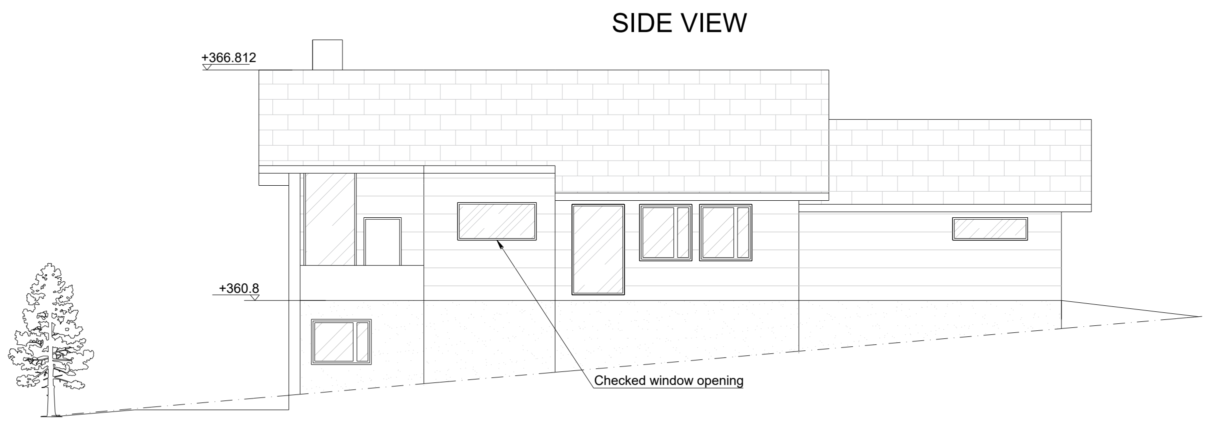
Load combination	Header plate	Sole plate
	$UR_{BearingCapacity1}$	$UR_{BearingCapacity2}$
Combination 1	25%	25%
Combination 2	73%	73%
Combination 3	65%	65%
Combination 4	14%	14%
Combination 5	50%	50%

Note! The design procedure is the same as in the case of other walls in the y-direction. The only variables that would change are the tributary length and the height of the studs. Detailed calculations are attached at the end of this document (Appendix 7).

13.6 Window opening on Y-direction

This project includes multiple windows; therefore, the load-carrying element control must be performed so that the structure can be used safely during its life. In this project, the joints extending along the Y direction are critical as the truss load enters these openings. While the stud elements are only under the influence of wind load in the X direction, even normal force is negligible. Therefore, providing a check for opening in the Y direction would be enough. This window opening has been taken into the analysis as its location is in the broadest extension of the building, where the length is 12.05 meters. As a consequence of this length, this element will take more load; therefore, the elements in this position will be more critical (Figure 39).

Figure 39. Window opening taken into analyses.

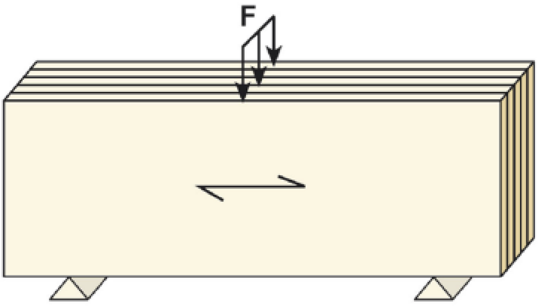


For the top part of the window opening, the Kerto S-beam will be used as the timber with dimensions of 48x173 would not be able to perform and take the loads (Figure 40).

Figure 40. Kerto S-beam (Kerto LVL manual, n.d.).

The beam will be mounted to which the bending will be parallel to the grain (Figure 41). This provides valid information from the manual from Metsä from where the correct coefficient to perform the analyses will be taken.

Figure 41. Edgewise bending, parallel to the grain (Metsäliitto Cooperative (Metsä Wood),



13.6.1 Result of bending, shear, and compression

Applying the rules denoted on the respective Eurocode, the following results were obtained from the Kerto S-beam used for this project (Table 29).

Table 29. Results of Kerto S-beam.

Condition	Combination 1	Combination 2
Bending	24.82%	72.05%
Shear	21.70%	62.99%
Compression	40.00%	30.00%
k_{mod}	0.6	0.8

13.6.2 Result of axial and combined action on Y-direction for the studs

An increase in the utilization ratio can be seen as the length between two stud elements will be more significant due to a window opening of 2100 millimeters. According to the calculations, although considerable in length, the stud element can withstand the loads (Table 30). The critical load combination is checked for the window.

Table 30. Results of window stud.

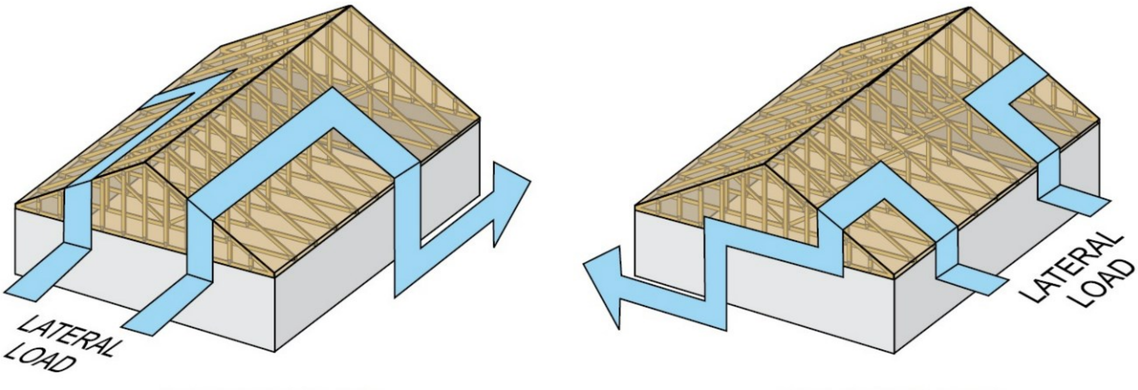
Condition	Critical load combination
Stud compression	46%
Combined action	85%

Note! The design procedure is the same as in the case of other walls in the y-direction. The only variables that would change are the tributary length and the height of the studs. Detailed calculations are attached at the end of this document (Appendix 9).

14 Data collection and analyses of stiffening walls

The building is under the effect of wind in different directions, and in order for this load to be safely transferred to the foundation, it must follow a specific load path (Figure 42). The roof is a stiff element, and it will take the wind load and transfer it to the stiffening walls. Then the stiffening wall must have sufficient resistance to withstand this load to be safely transferred to the foundation.

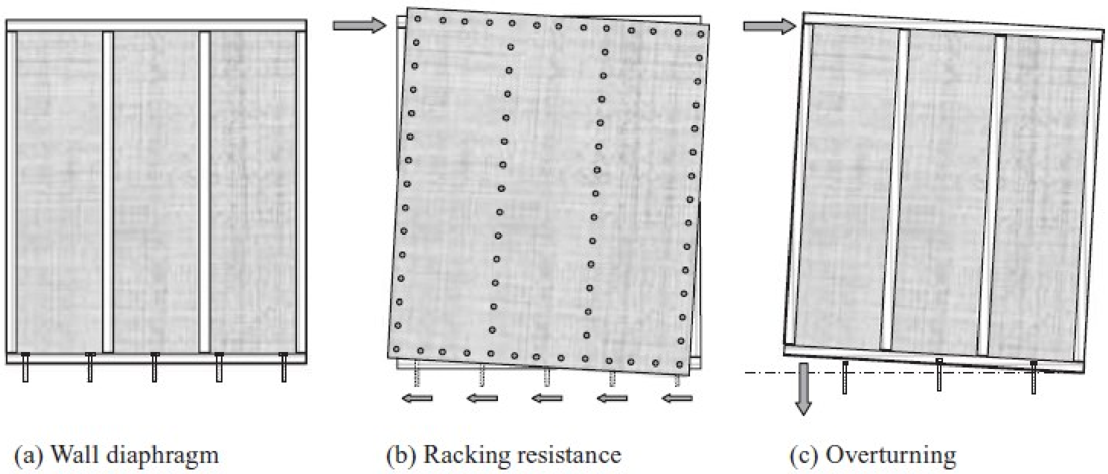
Figure 42. Load path of wind load (Sias, n.d.).



The position of the stiffening walls is crucial because they must be positioned in a place where the rotation of the wall should be prevented (Figure 43). Precision calculations must be performed in terms of load center and rigidity center to take this into account. Based on this method, it is possible to calculate the stiffness of the wall panels accurately. For this project, the resistance of the wall panels was calculated directly.

By calculating the resistance of each of the walls, the total of all resistances is added and compared with the force coming from the wind load. At the end of this document, various configurations for the stiffening wall ensure that the walls have sufficient capacity to withstand the wind load.

Figure 43. Resitance of wall panel (Porteous & Kermani, 2007).



Timber structures, in most cases, have sheathing material on both sides, which are held together by fixings. These elements make the wall behave like a rigid diaphragm. The relevant Eurocode indicates that the resistance of the timber panel can be done experimentally or follow the calculation method A or B. SFS-EN 1995-1-1 Eurocode 5, Design of Timber Structures, chapter 9.2.4 can be followed for specific information on both methods. In this project method, A is followed, making it possible to calculate the resistances of each panel (IStructE/TRADA, 2007).

14.1.1 Resistance to stiffening walls

The resistance, otherwise known as design racking load-carrying capacity, should be calculated from the following formula (CEN/TC124, 2016).

$F_{v.Rd} = \sum F_{i.v.Rd}$

Where:

$F_{i.v.Rd}$ - is the design racking load-carrying capacity of the wall panel (CEN/TC124, 2016).

14.1.2 Design racking load-carry capacity of wall panel

The walls of this project will consist of different sizes because, in different positions, there may be doors or windows, which condition the length of the wall panels. The formula calculates the individual resistance of the panels as follows (CEN/TC124, 2016).

$$F_{i.v.Rd}=\frac{F_{f.Rd} \cdot b_i \cdot c_i}{s}$$

Where:

$F_{f.Rd}$ - is the lateral design capacity of an individual fastener.

b_i - is the wall panel width.

c_i - is the fastener spacing.

and

$$c=\begin{cases} 1, \text{ for } b_i \geq b_0 \\ \frac{b_i}{b_0}, \text{ for } b_i < b_0 \end{cases}$$

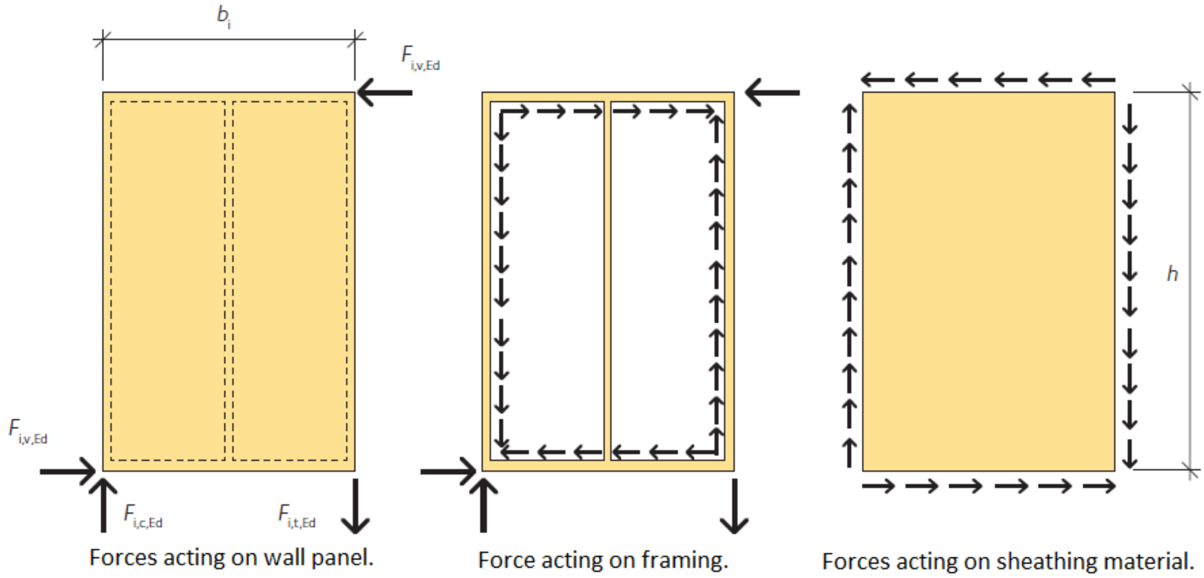
Where:

$b_0=\frac{h}{2}$

h - is the height of the wall panel.

Eurocode includes two methods to analyze stiffening walls consisting of method A and method B. Method B is more conservative as it considers the capacity of the elements with openings and the positive effect of vertical load on the wall diaphragm (Borgström, 2016). Unorthodox method A is applied with tie-downs at their ends and if the width of the sheet is greater than h / 4, where h is the height of the wall panel (Figure 44) (Borgström, 2016). For this project, this condition is valid, and the analysis to find the resistance of the wall panels is applied.

Figure 44. Force actions of the wall element.



14.1.3 Shear buckling of the sheathing

Shear buckling of the sheating for this project is not considered as the following condition is fulfilled based on the sheathing material thickness (CEN/TC124, 2016). The distance between the stud elements for this project will be 600 millimeters, and the sheathing thickness will be greater than 12 millimeters.

$$\frac{b_{net}}{t} \leq 100$$

Where:

b_{net} - is the clear distance between stud elements.

t - is the thickness of the sheet.

14.1.4 Fastener selection

The fastener selection is crucial for the stiffening wall as it will make the wall panel act as a rigid diaphragm. The central layer to which the faster would go through would consist of wood and gypsum board, giving enough information to select the suitable fastener. In addition, the wall's exposure to the outside or inside conditions should be taken into account. This would affect the service class and also the fastener selection. Fasteners will be from Gyproc manufacturers with the following shear resistance value (Table 31). Furthermore, the minimum spacing specified in the manual will be 70 millimeters used for this project.

Table 31. Fastener used for the stiffening walls.

Fastener	Plate type	Service class	Shear strength	Unit
QM-ST 32	GN 13/ Gyproc 4 Pro	1	450	kN
QU 32	GTS 9	2	400	kN

For the wall exposed to the outside weather condition, the external walls would belong to service class 2, and the following fastener can be used according to Gyproc's manual (Figure 45).

Figure 45. QU 32 fastener.

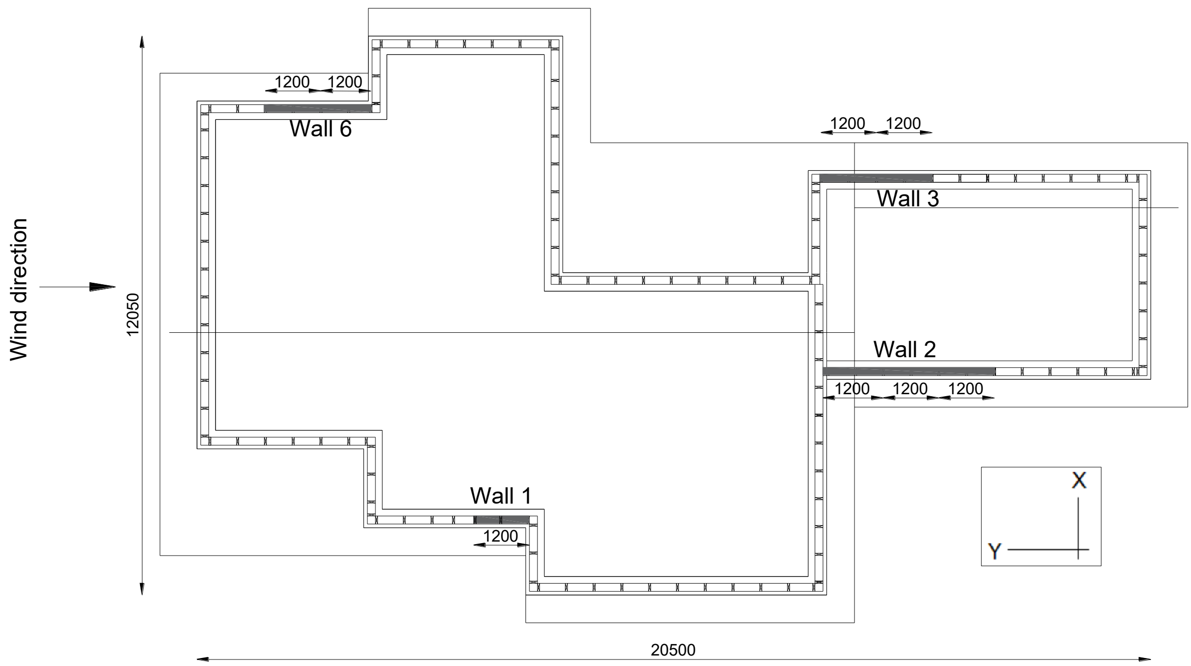
For the wall exposed to the inside conditions, the interior walls would belong to service class 1, and the following fastener can be used according to Gyproc's manual (Figure 46).

Figure 46. QM-ST 32 fastener.

14.2 Stiffening walls resistance on Y-direction

In this project, some difficulties have been encountered in the outer walls; numerous windows seem to reduce the space, which can be used to construct stiffening walls. Since in the space of the building, which will be a residential part, there will be windows, for the most part, the solution offered has to do with the fact that there will not be many windows in the storage space, leaving space to use stiffening wall panels. The plan showing the location of the stiffening walls is presented below (Figure 47).

Figure 47. Location of the wall panels.



For the walls in this direction, nails with a resistance of 450 Netwon in characteristic value and distance will be used every 70 millimeters. Based on this information, the resistance of all panels is calculated (Table 32).

Table 32. Design resistance of walls.

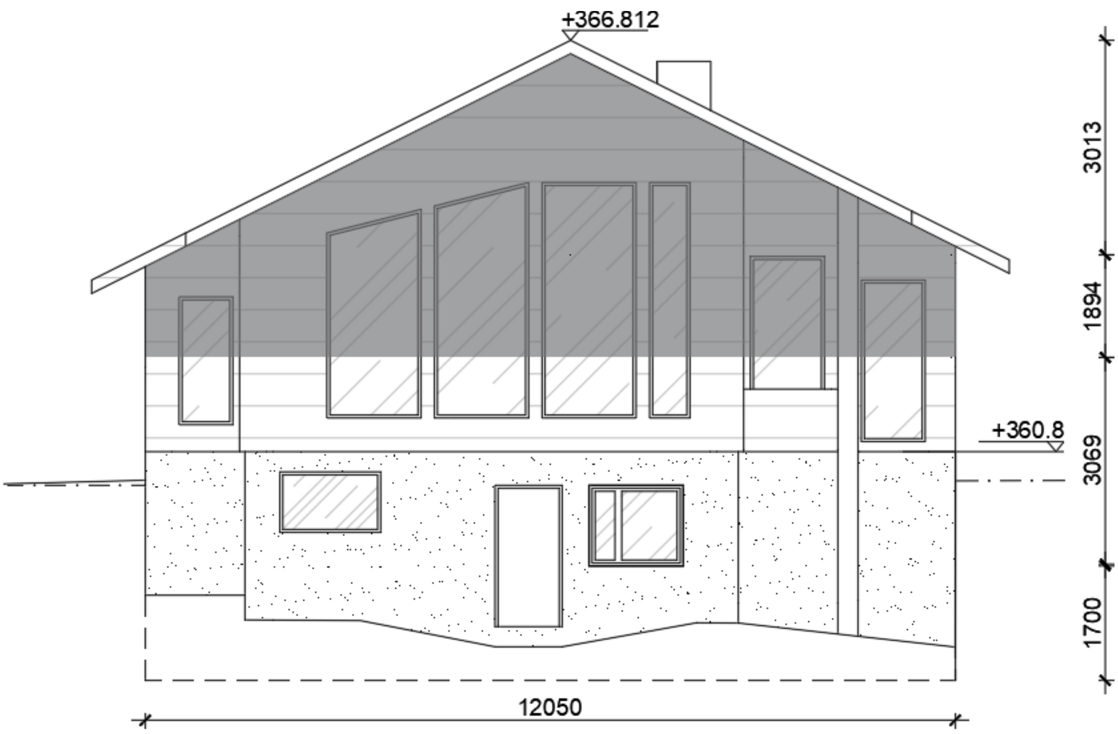
Symbol	Number of panels	Unit	Panel width	Unit	Resistance	Unit
Wall 1	1	-	1200	mm	4.594	kN
Wall 2	3	-	1200	mm	17.09	kN
Wall 3	3	-	1200	mm	11.393	kN
Wall 4	3	-	1200	mm	8.901	kN
Total resistance of walls	10	-	1200	mm	41.978	kN

Note! Detailed calculation is provided at the end of this document (Appendix 10).

14.2.1 Design of wind force in Y-direction

In this project, the resistance of the wall panels will be compared with the force coming from the wind load. The calculation of the wind force that will be taken from the walls in the Y direction is done by taking into account the surface of the wind face. Knowing the value of wind force in the Ultimate Limit State, finding the total force obtained from the stiffening wall in the Y direction becomes possible. To be conservative, the longest breadth of the building is considered while defining the value of the designed wind force.

Figure 48. Highlighted area to be considered to find wind force.



The whole wind force coming to the walls as a point load will be 29.132 kilonewtons (Table 33).

Table 33. Design wind force.

Wind load on Y-direction								
Symbol	Characterstic value	Unit	Design value	Unit	Area	Unit	Point load	Unit
$Q_{k.3}$	0.474	kN/m^2	0.711	kN/m^2	40.973	m^2	29.132	kN

14.2.2 Result of stiffening walls on Y-direction

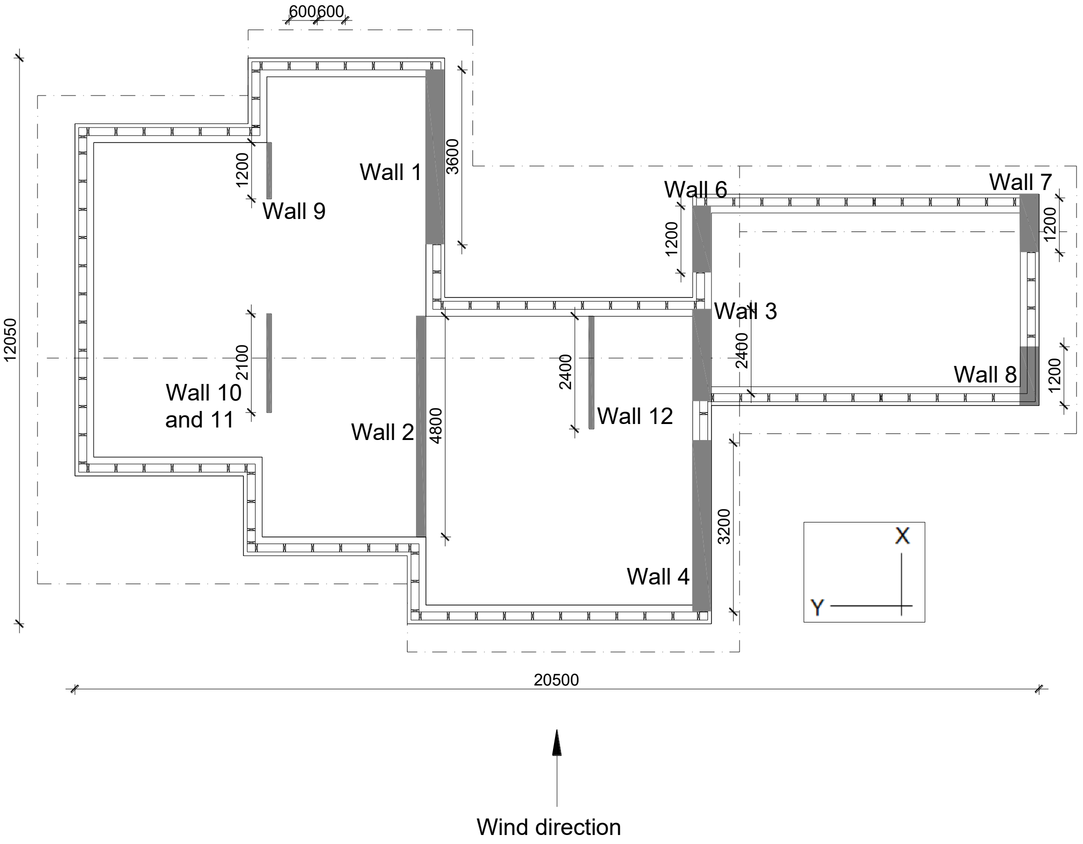
Based on the calculations made, it is concluded that the stiffening wall in the Y direction has sufficient capacity to withstand the force coming from the wind load. The utilization ratio is 69.697%.

Note! All detailed calculations are provided at the end of this document (Appendix 10).

14.3 Stiffening walls resistance on X-direction

For the stiffening walls in the X-direction, the wind load will be more significant as a result, and the breadth of the building will be bigger compared to the other direction. Therefore, more stiffening walls are needed for this direction to take the wind load (Figure 49). As a result, the partition walls of the building will be used as stiffening walls to take the wind load.

Figure 49. Location of the stiffening walls.



For the walls in this direction, the nails would be different for the inner wall as they are in dry conditions, and the service class would change to one; therefore, fasteners with 400 Netwon in characteristic value and spacing would be 70 millimeters.

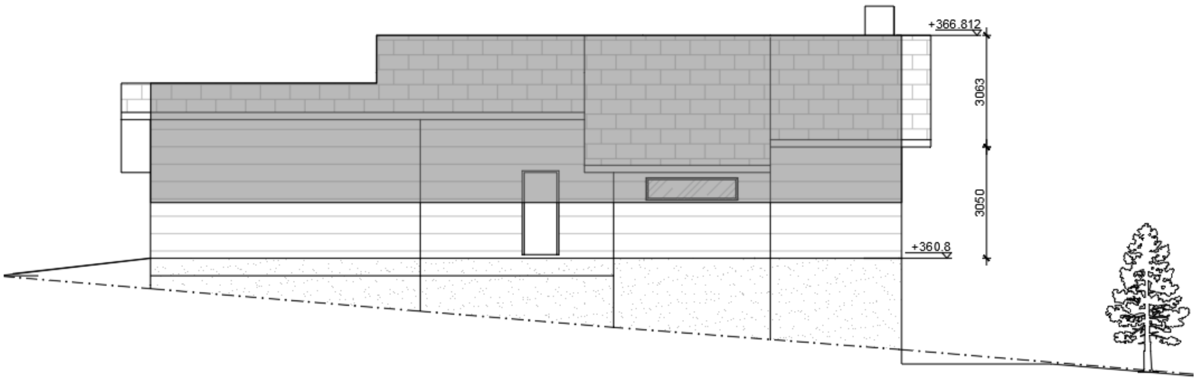
Table 34. Resistances of the walls.

Symbol	Number of panels	Unit	Panel width	Unit	Resistance	Unit
Wall 1	1	-	1200	mm	5.697	kN
Wall 2	3	-	1200	mm	21.099	kN
Wall 3	3	-	1200	mm	10.127	kN
Wall 4	3	-	1200	mm	10.173	kN
Wall 6	2	-	1200	mm	5.697	kN
Wall 7	1	-	1200	mm	5.697	kN
Wall 8	1	-	1200	mm	5.697	kN
Wall 9	1	-	1200	mm	5.697	kN
Wall 10	1	-	1200	mm	5.064	kN
Wall 11	1	-	1200	mm	5.064	kN
Wall 12	2	-	1200	mm	10.127	kN
Total resistance of walls	19	-	1200	mm	90.139	kN

14.3.1 Design of wind force on X-direction

The wind load in the X-direction would be more significant due to the longer span of the building in this direction (Figure 50).

Figure 50. Highlighted area to be considered to wind force calculation.



In this direction, the total wind force would be 86.047 Kilonewtons (Table 35).

Table 35. Design force wind.

Wind load on Y-direction								
Symbol	Characterstic value	Unit	Design value	Unit	Area	Unit	Point load	Unit
$Q_{k.3}$	0.646	kN/m^2	0.969	kN/m^2	88.8	m^2	86.047	kN

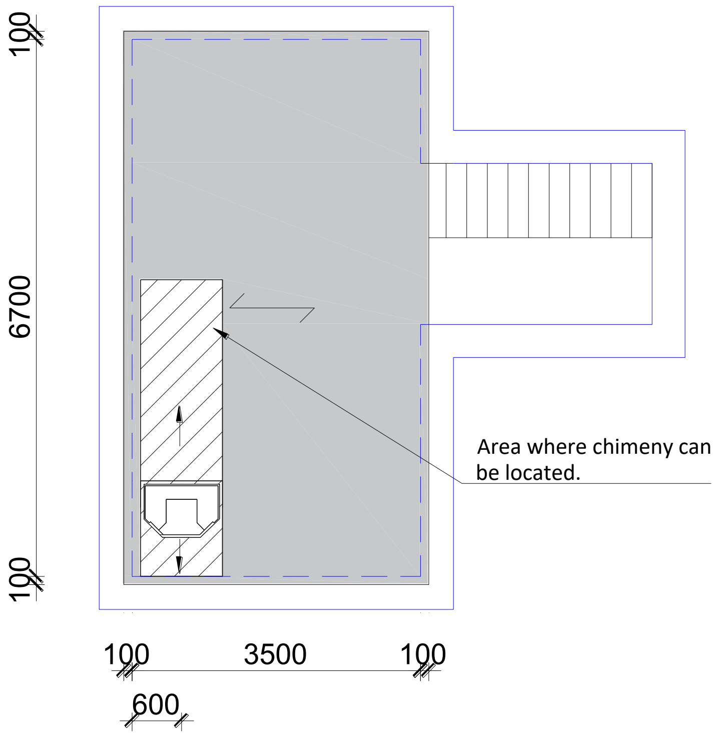
14.3.2 Result of stiffening walls on X-direction

The wind force in this direction is going more significant; therefore, more stiffening walls were designed for this direction to take the wind force. It was concluded that the utilization ratio in this direction is 95%. This utilization ratio is close to the limit, but the wall will have wooden boards on the sides that contribute to the wall element's stiffness besides the gypsum board. As there are no specific calculations about the elements, it is safe to say that the structure will be able to perform safely under the wind load. Refer to the end of the document for the detailed calculations (Appendix 11).

15 Data collection of concrete slab

For this building, the critical slab will be on the first floor. The slab in this part is not supported on the ground but will be kept on load-carrying walls. Furthermore, as shown in the figure below, a fireplace will also be in this slab, acting as a dead load (Figure 51). Therefore, this will be the critical slab to be designed. The slab will be supported on the load-bearing walls of the basement with a bearing width of 100 mm on all sides.

Figure 51. Slab layout plan.



15.1.1 Design selection

The data for which the concrete slab will be designed is based on the rules set by the relevant Eurocode and the implementation of the National Annex of Finland.

So, this slab will be designed according to the National Annex of Finland, where all the rules regarding the deflection limit will be applied, the combinations for which the structure will be controlled, and others (Appendix 13). Since this project will serve as a residential building and the slab is a structure located in the conditions of the indoor environment, the exposure class must be selected correctly (Table 36).

Table 36. Exposure classes related to environmental conditions in accordance with EN 206-1 (CEN, Eurocode 2: Design of Concrete Structures, 2004).

Class designation	Description of the environment	Informative examples where exposure classes may occur
1 No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete inside building with very low air humidty
2 Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wer, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3 Corrosion induced by chlorides		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete components exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements Car park slabs

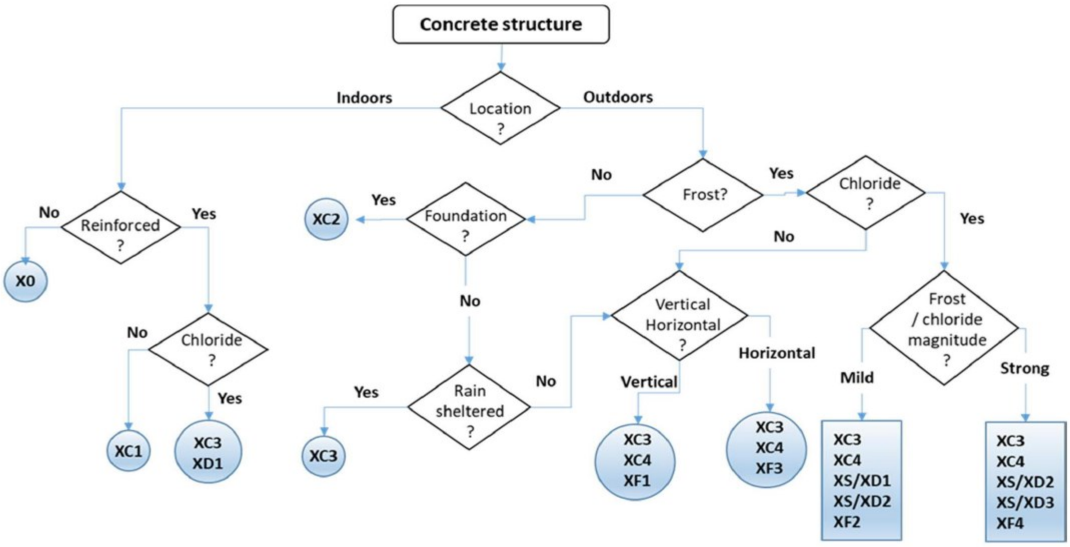
This structure will be made of concrete; therefore, it is necessary to determine the allowed crack width based on the relevant Eurocode, which for this exposure class is 0.3 millimeters (Table 37) (Ministry of the Environment, 2007).

Table 37. Values of maximum crack width in milimeters (Ministry of the Environment, 2007).

Exposure class	Reinforced members and prestressed members with unbonded tendons	Prestressed members with bonded tendons
	Quasi-permanent load combination	Frequent load combination
X0, XC1	0.4$^{(1)}$	0.2
XC2, XC3, XC4, XCD1, XS1	0.3	0.2$^{(2)}$
XD2, XD2, XS2, XS3	0.2	Decompression
Note 1: For X0, XC1 exposure classes, crack width has no influence on durability and this limit is set to guarantee acceptable appearance. In the absence of appearance conditions this limit may be relaxed. Note 2: For these exposure classes, in addition, decompression should be checked under the quasi-permanent combination of loads.		

Selecting the exposure class correctly is very important because, based on it, the necessary elements are found for the design of a concrete structure, such as determining the nominal cover and the values for maximum crack width. Below is a guiding figure which can be followed to provide an accurate selection of exposure classes of concrete (Figure 52).

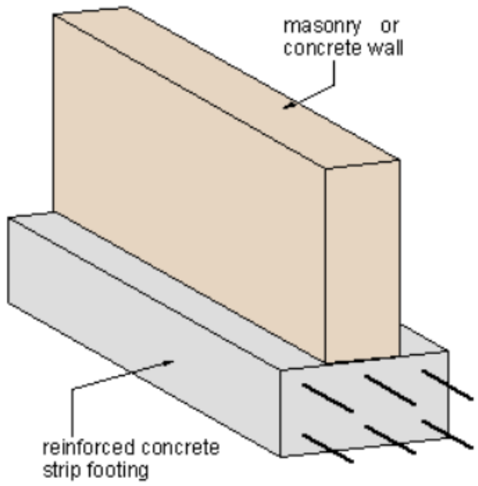
Figure 52. Selection of concrete exposure class.



16 Data collection of the strip foundation design

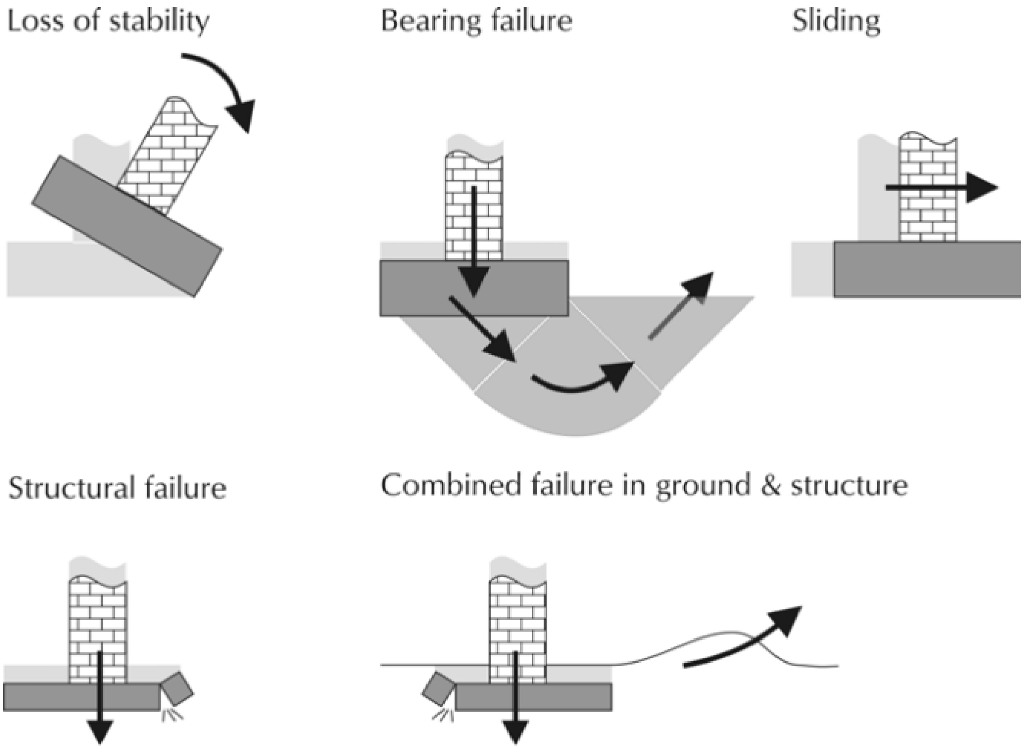
A strip foundation will be designed for this project (Figure 53). Compared to the ground level, the building will be close, and this offers the possibility of using a shallow foundation, another term for strip foundation. Like any design of other structures, even in the foundation, some conservative approximations are used to calculate the foundations in cases where information is missing, such as the type of soil. In this project, strip foundation finds a wide use because when the construction process of strip foundation is completed, it will be easy to install Leca blocks offered by Weber SAINT-GOBAIN. In cases where the soil type is strong, the dimensions of the strip footing will also be smaller if the soil is softer. For this project, the soil type is sand moraine with a capacity of 250 kN / m2.

Figure 53. Strip footing model (Concrete And Concreting, n.d.).



The strip foundation design will take one meter of its length into account. Specific calculations are attached at the end of this document (Appendix 12). Different failure modes or events may affect the structure's integrity during strip foundation design. In order to realize a safe design, loss of stability, bearing failure, sliding, structural failure, and combined failure in-ground and structure must be avoided (Figure 54) (Design of footings).

Figure 54. Failure modes of foundation (Design of footings).



16.1.1 Data collection for foundation design

The strip foundation design is done in a self-taught way, which involves research work to achieve the desired result. The collected data comes from various books, relevant Eurocode reports, and the use of various online sources. The collection of information has been done efficiently to enable the design of the foundation's structure according to the relevant Eurocode and the National Annex of Finland.

16.1.2 Strip foundation dimensions

The basic design principle of the strip foundation lies in the fact that the load will come from the top down. This project considers all the forces that create axial compression in the foundation and bending moment according to the axis of the foundation. Not all of the bottom will work effectively at soil pressure. Therefore the primary condition that should be checked to see if the footing should be moved to the right is as follows.

$P_k > P_G$

Where:

P_k - effective compressive stress on the foundation.

P_G - soil pressure.

If the above condition is correct, the footing must be moved to the right with an adequate length until the equation is proven.

$P_k < P_G$

The movement of the strip foundation is done until the condition for which the foundation can be designed is fulfilled. Not always moving the foundation to the right will result in a solution. Consequently, in cases where design is impossible, changing the concrete class or dimensions should be considered.

At the end of this document is shown the strip foundation design procedure where it is seen that new dimensions of the foundation must be selected so that the structure is safe during its life (Appendix 12).

From the calculations made, it has been concluded that the strip foundation for this project will have dimensions of 700 by 250 millimeters.

16.1.3 Strip foundation reinforcement

Usually, strip foundations, since they are in direct contact with the ground, also continuously extend the minimum of bars defined in the National Annex of Finland can be used. To calculate the minimum reinforcement bars needed for strip foundation, the following formula is used from the National Annex of Finland.

$$A_{s.min}=\min\left(\frac{0.26\cdot f_{ctm}\cdot b\cdot t\cdot d}{f_{yk}},0.0013\cdot d\cdot t\cdot d\right)$$

Where:

f_{ctm} - is the mean tensile strength of concrete.

b - is the width of the strip foundation; the width considered for design is one meter.

t - is the width of the strip foundation.

d - is the depth of foundation.

f_{yk} - is steel characteristic yield strength.

The calculation for the reinforcement concluded that with the minimum rebar diameter, the number of reinforcements is negligible; therefore, mesh reinforcement consisting of #5-150 will be used.

16.1.4 Summary of results

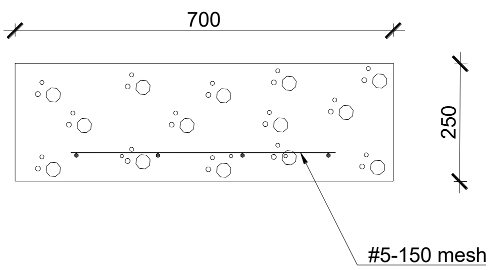
Based on the methods and principles defined in Eurocode, it became possible to design the strip foundation with the following data (Table 38).

Table 38. Strip foundation results.

Strip foundation			
Dimensions	Unit	Reinforcement	Unit
700x250	mm	Mesh #5-150	-

The detailed drawing of the design strip foundation is shown below (Figure 55).

Figure 55. Strip foundation detail.



17 Checklist of the objectives

As a work ethic, after the whole project is completed, tables should be made that detail the designed elements and the products ready to be delivered to the client. This is an individual preference, but creating these tables creates the opportunity to see the whole project, even to check if one of the elements is missing. If one of the project elements is missing, the clauses should be revised respectively. The first product sent to the customer has to do with the drawings needed for the building permit. Below is a checklist that shows if the drawings are completed and where they can be referenced in this document (Table 39).

Table 39. Checklist of the drawings.

Type of drawing	Description	Comments	Checklist	Reference	Gained knowledge through...
Building plot drawing was delivered	Location of the building	It meets the requirements according to RATU 15-10635	✓	Appendix 16	Planning of a One-Family House course
Structural types drawings were deliverd	Exterior wall of 1st floor	It meets the requirements according to RATU 15-10824	✓	Appendix 14	
	Exterior wall of basement	It meets the requirements according to RATU 15-10825	✓	Appendix 14	
	Interior wall	It meets the requirements according to RATU 15-10826	✓	Appendix 14	
	1st floor structure	It meets the requirements according to RATU 15-10827	✓	Appendix 14	
	Floor against the ground	It meets the requirements according to RATU 15-10828	✓	Appendix 14	
	Roof structure	It meets the requirements according to RATU 15-10829	✓	Appendix 14	
Detail connections were delivered	Wall to wall connection	It meets the requirements according to RATU 15-10830	✓	Appendix 15	
	Wall to roof connection	It meets the requirements according to RATU 15-10831	✓	Appendix 15	
	Wall to foundation connection	It meets the requirements according to RATU 15-10832	✓	Appendix 15	
Floor plans drawings were delivered	Spaces of the building	It meets the requirements according to RATU 15-10635	✓	Appendix 17	
Elevation drawings were delivered	Views of the building	It meets the requirements according to RATU 15-10636	✓	Appendix 18	
Section drawing	Section cut of the building	It meets the requirements according to RATU 15-10637	✓	Appendix 19	

The same thing is valid for structural calculations, where a checklist of the designed elements is done to have a clear view of the designed structures (Table 40).

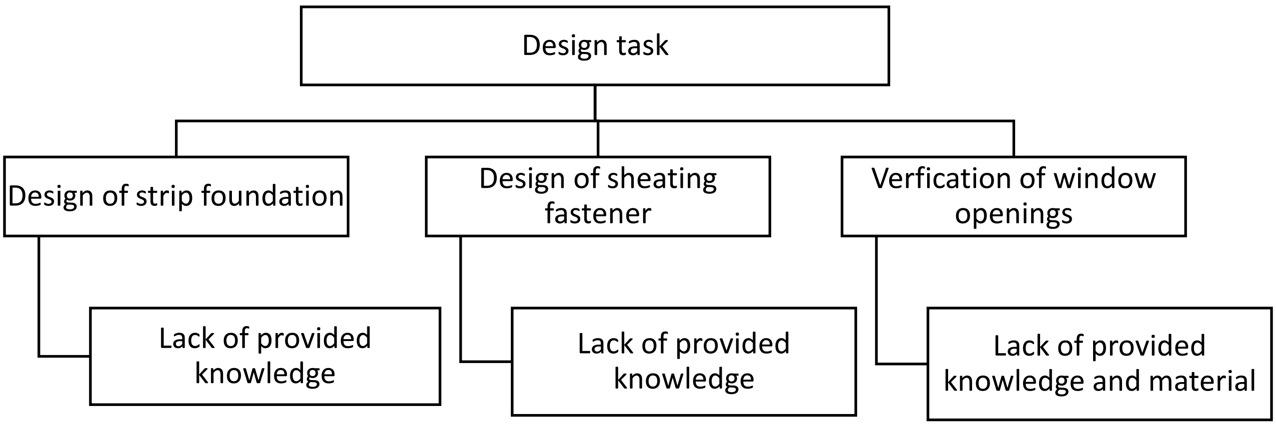
Table 40. Checklist of structural elements.

	Load description	Value	Unit	Reference	Gained knowledge through…
Characteristic load calculation were done for all actions	Dead load	0.683	kN/m^2	Appendix 1	Load bearing course
	Live load	2	kN/m^3	National Annex	
	Snow load	2.4	kN/m^4	Appendix 2	
	Wind load on X-direction	0.646	kN/m^5	Appendix 4	
	Wind load on Y-direction	0.474	kN/m^6	Appendix 5	
Designed elements	Description	Comments	Checklist	Reference	
	48x173 timber was designed for timber walls	It meets requirement according to EN 1995-1-1 Eurocode 5	✓	Appendix 7 and 8	Design of timber structures course
	173x48 timber was designed for header and sole plates	It meets requirement according to EN 1995-1-1 Eurocode 5	✓	Appendix 7 and 8	
	Kerto S-beam was designed for window openings	It meets requirement according to EN 1995-1-1 Eurocode 5	✓	Appendix 9	Research work as well as self-acquired knowledge.
	Stiffening walls were designed for X-direction	It meets requirement according to EN 1995-1-1 Eurocode 5	✓	Appendix 11	
	Stiffening walls were designed for Y-direction	It meets requirement according to EN 1995-1-1 Eurocode 5	✓	Appendix 10	
	700x250 strip foundation was designed	It meets requirement according to EN 1997-1: Eurocode 7	✓	Appendix 12	
	Concrete slab design, from Rukki's solution	It meets requirement according to EN 1992-1-1: Eurocode 2	✓	Appendix 13	

18 Evaluation of the study plan

Most of the knowledge is obtained from studies during different academic years regarding the design process. As for the design of different elements, one of the problems encountered during the design phase is the lack of learning material. The following figure shows the main drawbacks encountered in this thesis (Figure 2). Through this thesis, these elements are pointed out, and their solution is provided to offer a better learning process.

Figure 56. Identification of the missing knowledge from the study plan.



An example is the strip foundation design and the fastener selection for sheathing material. These were part of the project objectives, which had to be met, so it was necessary to do individual research work to achieve a specific result. This document shows what the steps are and the resources that have been used to achieve the design of a specific structure.

It is also necessary to mention that for the creation of drawings, even though a course has been completed regarding the creation of drawings with the AutoCad program, some rules must be met according to the conditions of Finland. During such courses, most of the materials in Finnish should be appreciated, which may slow down the process by which these drawings are created. The same thing is worth mentioning in this bag, to create the drawings, care must be taken to ensure that the drawings are in line with Finnish standards.

19 Analyses and discussions

This document presents the steps that are followed in designing a two-apartment house. Although the design process includes steps defined in Eurocode, design is a process that requires specific solutions and, in some cases, innovative solutions for reasons that may be technical and costly.

The building's mainframe consists of timber walls and roof trusses. The basement in this project will be a living space, removing the possibility of using a second floor as a living space. Having a basement as a living space is the client's wish. Therefore, timber walls and truss elements will be used for this project. The roof structure consists of truss elements obtained from local companies. With the development of technology, the only data given to the manufacturing company consists of the type of wood material, the shape of the truss, and the lattice span. If it were the opposite, for example, the basement would not be a living space, then, according to the client's wishes, a second floor had to be built. It was impossible to build the second floor because truss elements would offer a flat ceiling, leaving insufficient space for the second floor. For this reason, columns and beams elements should be used to provide enough space for the second floor. In conclusion, roof truss and timber walls were used, because the primary argument was that it would not be necessary to use space unless a second floor was needed.

One of the innovative solutions in this thesis that includes research work comparing the two methods is using a composite sheet by Ruukki. In the project's initial phase, the main objective was to design a concrete slab based on the traditional method, where the concrete would be cast in wooden formwork. Since the surface needed for the design of this floor structure is not very large, due to the simplicity of construction and because of the cost had to consider the implementation of several other ways to build the floor structure. The way that had to be chosen for the design of the floor structure had to be innovative, where the main goal would be to optimize all the elements in terms of installation of the structure and costs. To conclude, accurate cost calculations to compare the formwork method with the composite sheet have not been done.

Nevertheless, from experience and information, the method that would be more costly is using formwork to build concrete slabs. Also, from a practical point of view, formwork is not suggestable for a project of this scale where the complexity is not great. Considering these factors, it was concluded that the use of composite sheet by Ruukki company would be used in this project as the best way to maximize cost reduction and increase efficiency during the construction phase. In this way, the costs are reduced, and the time required to build this floor structure is faster. The design of the composite sheet is based on the use of the software provided by the company Ruukki; also, the design tool is called ComSlab. Based on the relevant data included in the previous chapters, the software generates results that provide valuable information on those specific parameters that the composite sheet can be used for the design. The results and analyzes for the composite sheet are attached at the end of this document (Appendix 13).

The products regarding the other building elements will be taken from local Finnish companies, such as Paroc, Finnfoam, and Leca. The selection of layers by these companies is made to offer a specific U-value for the structural type (Appendix 14). To generate the necessary drawings for the structural type, research has been done to ensure that these drawings are valid to be used in reality. So, the company that will build the project based on these drawings can build the structures. Other drawings have also been researched to ensure that the drawings comply with Finnish standards. One of the difficulties encountered when creating drawings is using the information in Finnish, making it more challenging to create drawings.

Various difficulties have been encountered regarding designing wooden and concrete structures at different project stages, increasing the sensitivity and the need for appropriate solutions. Although the procedure for designing different structures is provided in the relevant Eurocodes, the engineer is responsible for providing different solutions if a structure fails to pass the necessary verifications in terms of stability. So, Eurocode does not show how a structure should be designed if it fails, but it shows the procedures that a structure must follow to be designed in reality. In this project, various difficulties have been encountered regarding the design of structures, including the design of stiffening walls, foundations, and window openings.

In the project's initial phase, stud elements were presented with the same properties but different dimensions. It was chosen for verification and the stud design with a smaller dimension because this would reduce the necessary wood material, which automatically reduces the cost. Also, timber with a dimension of 48x173 millimeters is more convenient and finds wide use in residential buildings in Finland.

Also, in terms of stiffening walls design, some walls were placed to see if they had sufficient resistance to take the wind load coming to them in the initial stage. In the initial stages, difficulties were encountered as the walls did not have enough resistance to withstand the wind load. In the Y direction, the design of the stiffening walls did not show any issues. The same can not be said for the X direction, as in this direction, the wind load is more significant, increasing the need to use more wall panels. Even though the maximum allowed wall perimeter was used, it still failed to provide sufficient resistance for the walls. The solution offered was to increase the capacity of the screws to be used. Also, the definition of screws according to the exposure class was achieved, providing a design that manages to perform correctly. In the design phase of the stiffening wall, doing research work, it was learned how to select the screw capacity. So, it is not only about increasing the capacity of the screws in order for the verifications to be performed, but it is necessary to choose the screws that can be found in reality and be used to ensure sufficient capacity.

Furthermore, in terms of the design of the top beam in the openings of the building, such as the windows, it was tried to design the element with a size of 48x173 millimeters. During the verifications, it was concluded that this element failed to perform correctly, and a solution had to be provided. There were two options: to use double timber with a dimension of 48x198 millimeter and the other one to use Kerto S-beam. The solution offered was using Kerto S-beam, which consists of laminated veneer lumber material with better mechanical properties. The reason for that was that if double timber had been used, this would affect the overall U-value of the structure not efficiently.

Also, this document shows the design of the strip foundation, and it is worth mentioning that the design of this concrete structure needed research work as it was something new. This was one of the challenges of this project as research had to be done, and despite this, the information had to be applicable in practice. So, the result of the designed element must be applicable in reality, and this thesis shows how the design of the strip foundation is done (Appendix 12).

As mentioned above, various difficulties have been encountered in this project that require specific solutions. The information is relatively abundant, which shows the steps that must be followed from the beginning to the end of the design of a two-apartment house. The content of this thesis is essential because, in addition to the fact that it shows the implementation of knowledge learned during the academic years, it shows encountered difficulties that require specific solutions, as mentioned earlier. The integration of knowledge learned during the academic years is used to verify different structures, but the solutions come from unique ideas. There can be various solutions for different issues, but the solution must be the most efficient to enable a better design.

As a suggestion in the project's initial phase, preliminary assumptions can be made to create a general idea of the design structure. Once the project starts to advance, different solutions can be offered through precise calculations to ensure a safe design and environmentally and economically sustainable project.

To realize a project from the beginning, it is suggested to make a preliminary plan which contains the main points to be realized. Then one has to think about the research way of how the necessary material should be used to get a specific result. In this project, weekly meetings were held with the client, during which suggestions and various discussions were made to achieve a more durable design. During the project phase, it should be taken into account that different elements may change, for example, the position of a window, where the structural calculations may change through this change. In addition to that, the drawing needs to be updated accordingly.

Regarding structural calculation, it is suggested to use Mathcad software as it allows automatically changing calculations. Furthermore, the Finite Element Method is used if the opening dimensions change. This allows the changing of fewer variables, and the results can be updated faster. At the end of this document, the Finite Element Method implementation is shown (Appendix 9.)

Once all the necessary products have been completed, a thorough inspection should ensure everything is in order. The product will then be delivered to the client, who follows the project's construction phase into reality.

In conclusion, it can be said that in this facility, even though theoretical analysis was done through which the design of different elements is done, different problems are encountered during the design phase. This raises awareness that there should be environmentally and economically suitable solutions to provide a reliable design.

20 Conclusions

Over time, the construction sector is developing more, increasing the need for innovative ideas in the design process of various elements.

To conclude this thesis, it can be said that the completion and design of this two-apartment house have been successfully achieved. Objectives include performing the necessary drawings, starting from architectural drawings, floor drawings, and structural types. Also, in this document, the design of the defined structures consisting of wood materials and concrete has been realized. Furthermore, implementing an innovative method of designing the floor structure for this project raised awareness of the new technologies used in the construction field.

Most importantly, the learning process for performing this design process from start to finish is covered in this thesis. For this reason, this document presents all the steps to be followed to achieve a particular result. Moreover, exact methods are followed in the design of structures and simplified methods for understanding purposes. It is worth noting that for projects of a not very complex scale, such as this project, it is possible to make some assumptions that make the design process more manageable.

In conclusion, this thesis can be used for learning and focusing more on practical thinking in an innovative way where possible, and it emphasizes the evaluation process of different tasks to be considered for this project.

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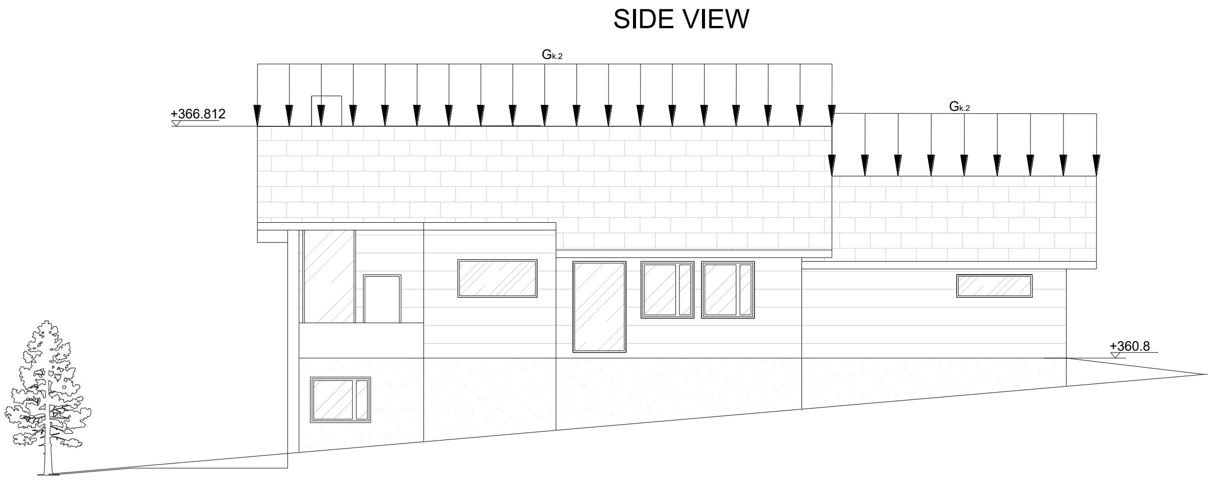
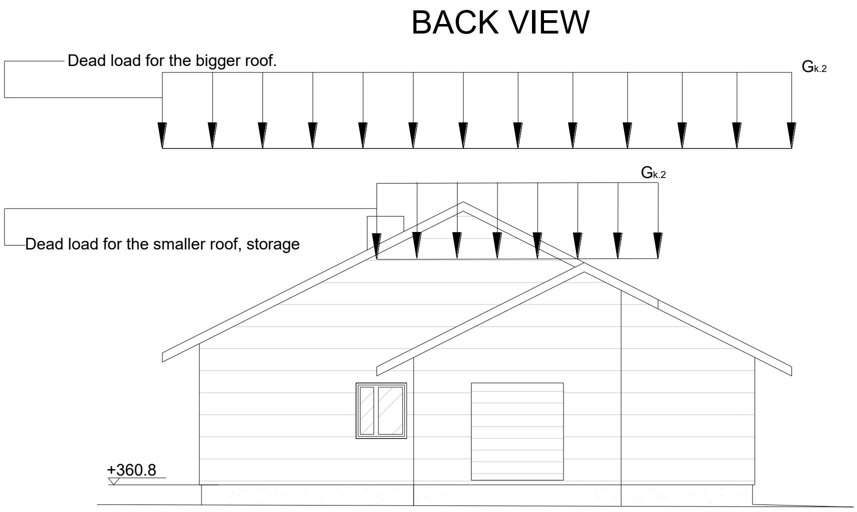
Appendix 1: Dead load calculation





Bitumen felt.

From EN1991-1-1 table A12. $\rho_{bitumen} := 0.36 \ \frac{kN}{m^3}$

The thickness of the bitumen layer; is a conservative value. $t_{bitumen} := 3 \ mm$

Dead load from bitumen felt. $q_{k.1} := \rho_{bitumen} \cdot t_{bitumen} = 0.001 \ \frac{kN}{m^2}$

Wooden boards.

Grade of timber. C24

The density of the wood. $\rho_{C24} := 4.2 \ \frac{kN}{m^3}$

The thickness of wooden boards. $t_{wood} := 23 \ mm$

Dead load from wooden boards. $q_{k.2} := \rho_{C24} \cdot t_{wood} = 0.097 \ \frac{kN}{m^2}$

Truss elements.

The span of the truss. $L_{truss} := 12 \quad m$

Dead load from truss element. $q_{k.3} := \left(\frac{L_{truss}}{3} + 5\right) \cdot 10 = 90 \ \frac{N}{m^2}$

Truss weight. $q_{k.3} := 0.09 \ \frac{kN}{m^2}$

Insulation.

Conservative value. $\rho_{insulation} := 0.25 \ \frac{kN}{m^3}$

The thickness of insulation. $t_{insulation} := 600 \ mm$

Dead load from insulation. $q_{k.4} := \rho_{insulation} \cdot t_{insulation} = 0.15 \ \frac{kN}{m^2}$

Wooden frame boards of the ceiling.

Grade of timber. C24

The density of the wood. $\rho_{C24} := 4.2 \ \frac{kN}{m^3}$

Conservative value. $q_{k.5} := 0.25 \ \frac{kN}{m^2}$

Weight of ceiling material.

Conservative value. $\rho_{wood.board} := 6.8 \ \frac{kN}{m^3}$

The thickness of insulation. $t_{wood.board} := 14 \ mm$

Dead load from insulation. $q_{k.6} := \rho_{wood.board} \cdot t_{wood.board} = 0.095 \ \frac{kN}{m^2}$

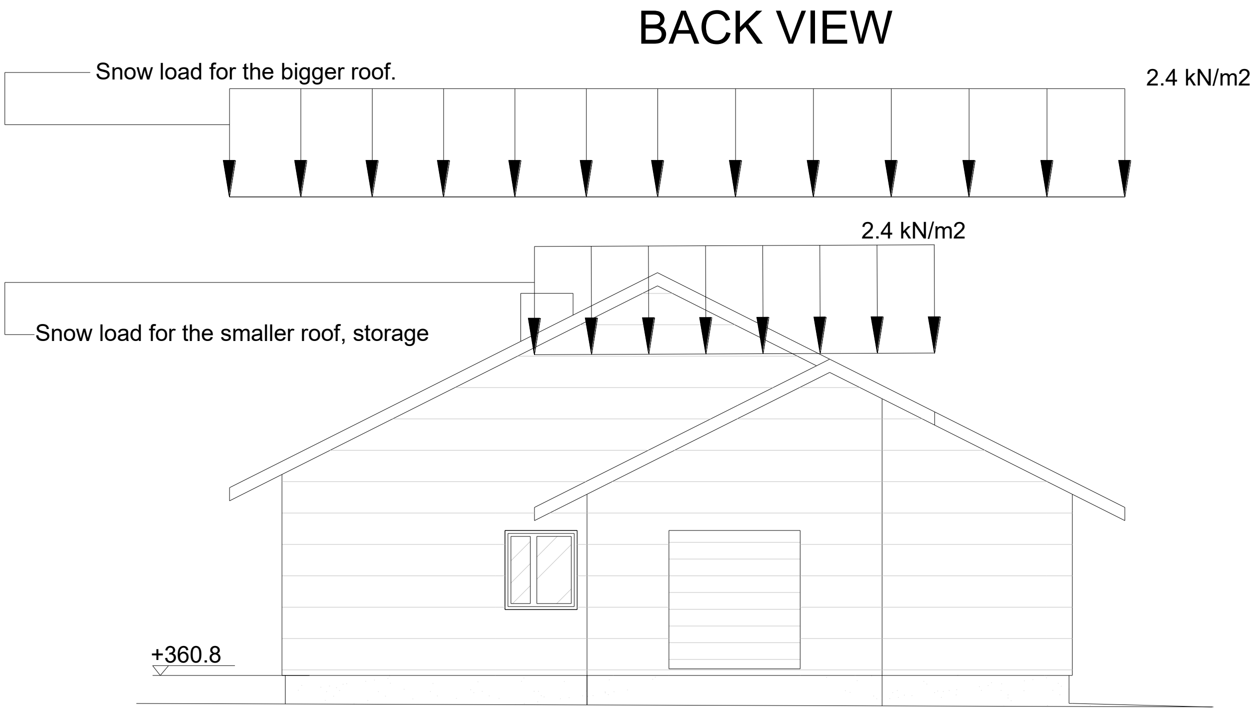
Summary of all dead loads. $q_{dead.load} := q_{k.1} + q_{k.2} + q_{k.3} + q_{k.4} + q_{k.5} + q_{k.6} = 0.683 \ \frac{kN}{m^2}$

Conclusions

Characteristic roof dead load. $G_{k.2} := q_{dead.load} = 0.683 \ \frac{kN}{m^2}$

Appendix 2: Name of Appendix

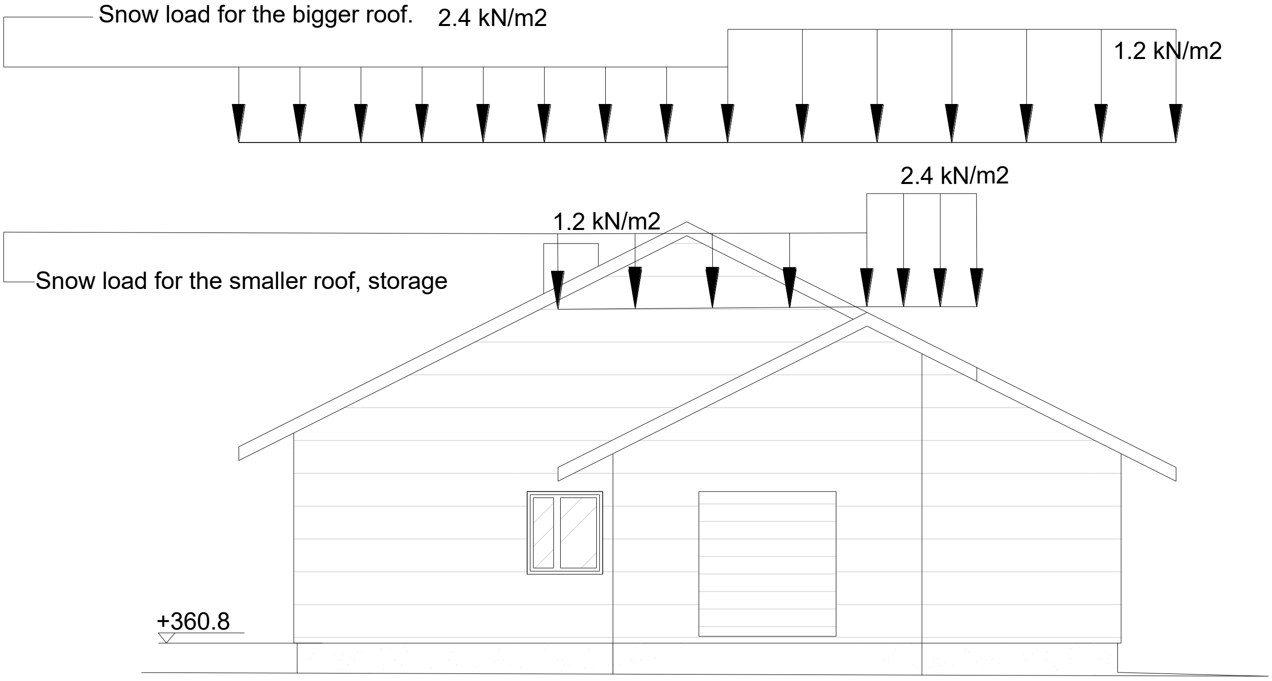
The roof slope	1:2
Roof angle in degrees.	$\alpha := atan\left(\frac{1}{2}\right) \cdot \frac{180}{\pi} = 26.565$
From EN1991-1-3, 5.2 (5.1).	$s = \mu_i \cdot C_e \cdot C_t \cdot s_k$
The characteristic value of snow in the ground, National Annex.	$s_k := 3\ \frac{kN}{m^2}$
The snow load shape coefficient EN1991-1-3, 5.3 (5.3.1).	$\mu_i := 0.8$
The exposure coefficient EN1991-1-3, 5.3 (5.3.1), (7).	$C_e := 1$
The thermal coefficient EN1991-1-3,5.3 (5.3.1), (8).	$C_t := 1$
Characteristic snow load on the roof, EN1991-1-3, 5.2 (5.1).	$S := \mu_i \cdot C_e \cdot C_t \cdot s_k = 2.4\ \frac{kN}{m^2}$
Case (i)	$Load := S = 2.4\ \frac{kN}{m^2}$



Case (ii) $Load_{left.side} := 0.5 \cdot S = 1.2 \ \frac{kN}{m^2}$

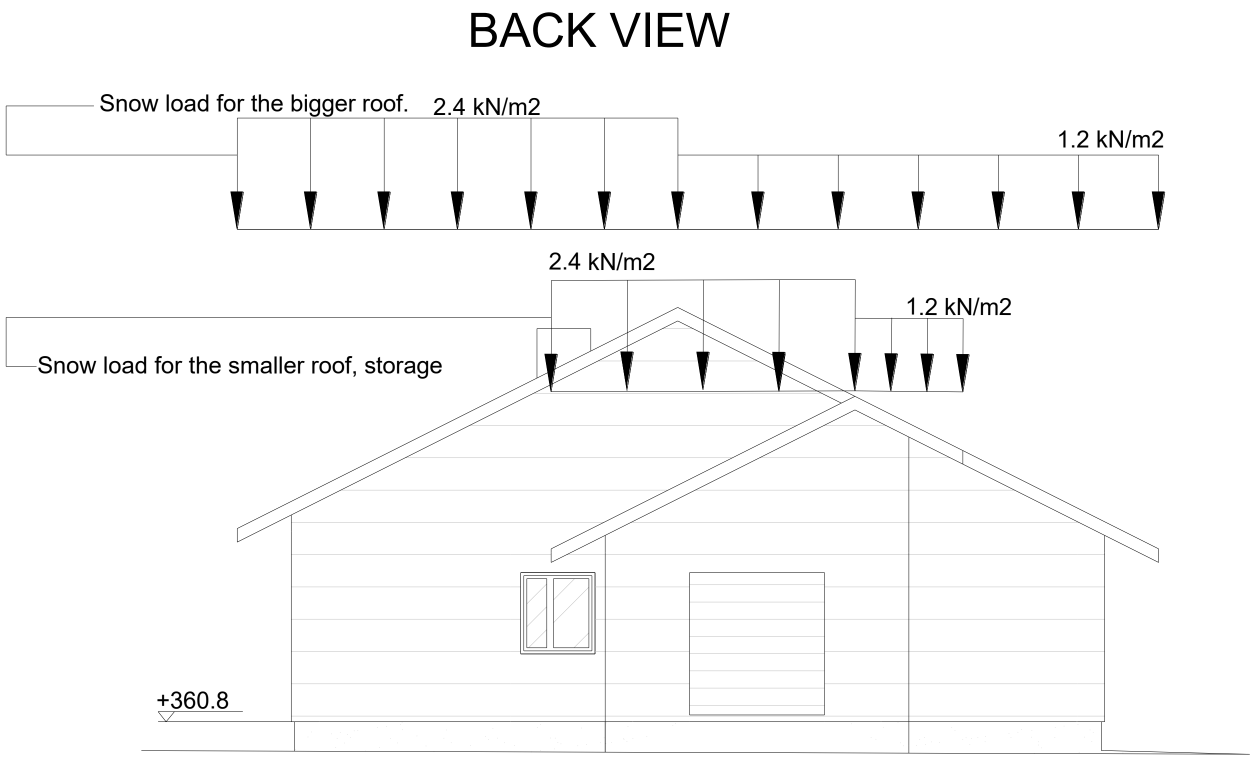
Case (ii) $Load_{right.side} := S = 2.4 \ \frac{kN}{m^2}$

BACK VIEW



Case (iii) $$Load_{right.side} := 0.5 \cdot S = 1.2\ \frac{kN}{m^2}$$

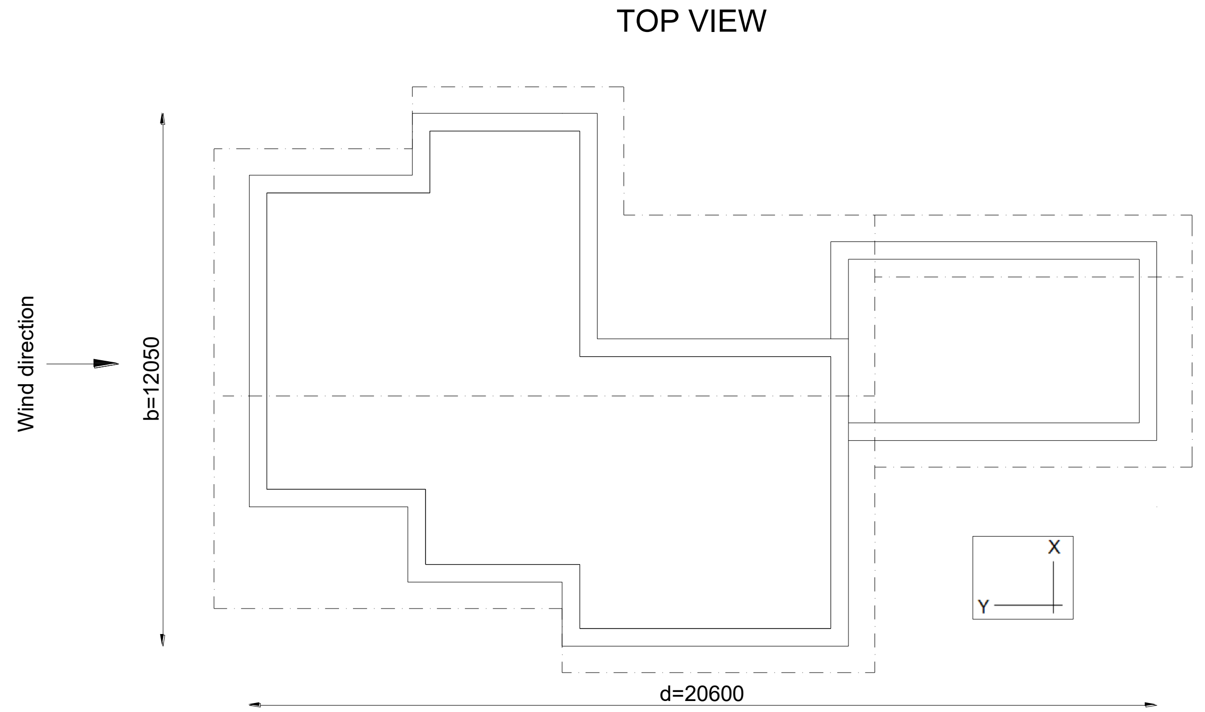
Case (iii) $$Load_{left.side} := S = 2.4\ \frac{kN}{m^2}$$

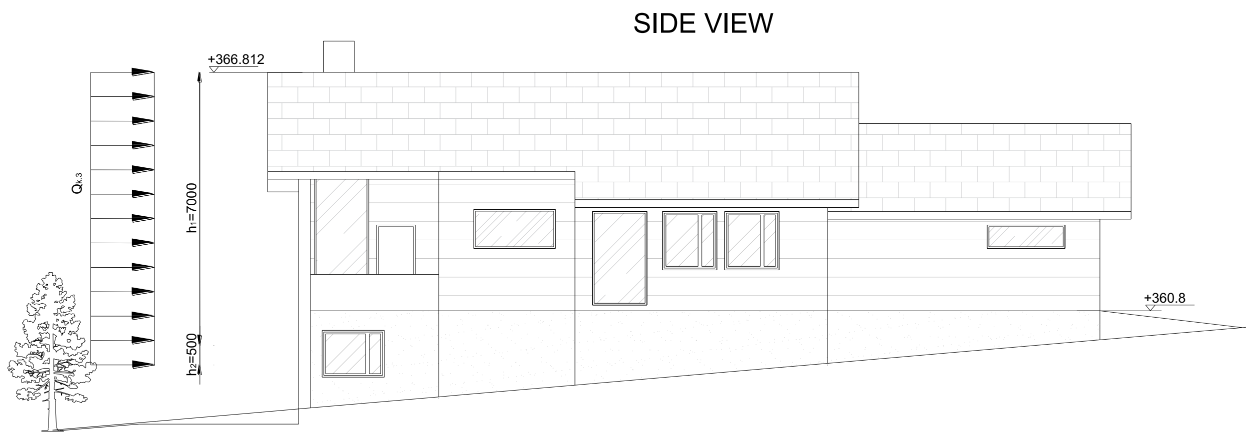


Conclusions.

Case (i) is considered for design as more load is applied to the roof.

Appendix 3: Wind load on Y-direction, simplified method





Reference height $h_1 := 7000\ mm$

Plinth visible against the ground. Recommended values are between 300 mm to 500 mm. $h_2 := 500\ mm$

Height of the building. $h := h_1 + h_2 = 7500\ mm$

Width of the building perpendicular to the wind direction. $b := 12050\ mm$

The width of the building parallel to the wind direction. $d := 20600\ mm$

Terrain parameter, terrain category III, from table 4.1 EN-1991-1-4. $z_0 := 0.3\ m$

Terrain parameter, terrain category III, from table 4.1 EN-1991-1-4. $z_{min} := 5\ m$

Terrain parameter, terrain category III, from table 4.1 EN-1991-1-4. $z_{max} := 200\ m$

Terrain category II, from table 4.1 EN-1991-1-4, 4.3.2. $z_{0.II} := 0.05\ m$

Condition to be checked according to EN-1991-1-4, 7.2.2,

$$z := \text{if } h < b \; \| \; h \; \text{else} \; \| \; \text{“check”} = 7.5\ m$$

Turbulence factor, from EN 1991-1-4, 4.4, (4.7). $K_I := 1$

Orography factor explained in 4.3.3 EN-1991-1-4. $c_0 := 1$

Air density. $\rho := 1.25\ \dfrac{kg}{m^3}$

$z_{min} \le z \le z_{max}$ therefore:

Turbulence intensity, from 4.4 EN-1991-1-4.

$$I_v := \frac{K_I}{c_0 \cdot \ln\left(\dfrac{z}{z_0}\right)} = 0.311$$

Terrain factor based on the roughness length z_0, from 4.3.2, (4.5) EN-1991-1-4.

$$K_r := 0.19 \cdot \left(\frac{z_0}{z_{0.II}}\right)^{0.07} = 0.215$$

$z_{min} \le z \le z_{max}$ therefore:

Roughness factor, 4.3.2, (4.4), EN-1991-1-4.

$$c_r := K_r \cdot \ln\left(\frac{z}{z_0}\right) = 0.693$$

Wind velocity

Directional factor. $c_{dir} := 1$

Seasonal factor. $c_{season} := 1$

The fundamental value of basic wind velocity; a conservative value. $v_{b.0} := 22 \frac{m}{s}$

Basic wind velocity, from 4.2, (4.1), EN-1991-1-4. $v_b := c_{dir} \cdot c_{season} \cdot v_{b.0} = 22 \frac{m}{s}$

Mean wind

Mean wind velocity, from 4.3.1, (4.3), EN-1991-1-4. $v_m := v_b \cdot c_0 \cdot c_r = 15.253 \frac{m}{s}$

Peak velocity pressure, from 4.5, (4.8), EN-1991-1-4. $q_p := \left[1 + 7 \cdot I_v\right] \cdot \frac{1}{2} \cdot \rho \cdot {v_m}^2$

Peak velocity pressure. $q_p = \left[461.616\right] \frac{N}{m^2}$

Characteristic pressure on the wall for checking the stability of the building

Height less than 15 meters. $h = 7500\ mm$

From EN 1991-1-4, 5.3, (5.4). $c_s := 1$

From EN 1991-1-4, 5.3, (5.4). $c_d := 1$

Interpolation using the table from RIL 201-1-2008.

	d/b								
λ	0.1	0.2	0.5	0.7	1	2	5	10	50
≤ 1	1.2	1.2	1.37	1.44	1.28	0.99	0.6	0.54	0.54
3	1.29	1.29	1.48	1.55	1.38	1.07	0.65	0.58	0.58
10	1.4	1.4	1.6	1.68	1.49	1.15	0.7	0.63	0.63

Width of the building perpendicular to the wind direction. $b = 12.05\ m$

The width of the building parallel to the wind direction. $d = 20.6\ m$

Ratio $\frac{d}{b} = 1.71$

Slenderness defined from RIL 201-1-2008 $\lambda_{RIL} := \frac{2 \cdot h}{b} = 1.245$

Defining c_f factor. $c_f := 0.99 + \frac{(1.71 - 2)}{(5 - 2)} \cdot (0.60 - 0.99) = 1.028$

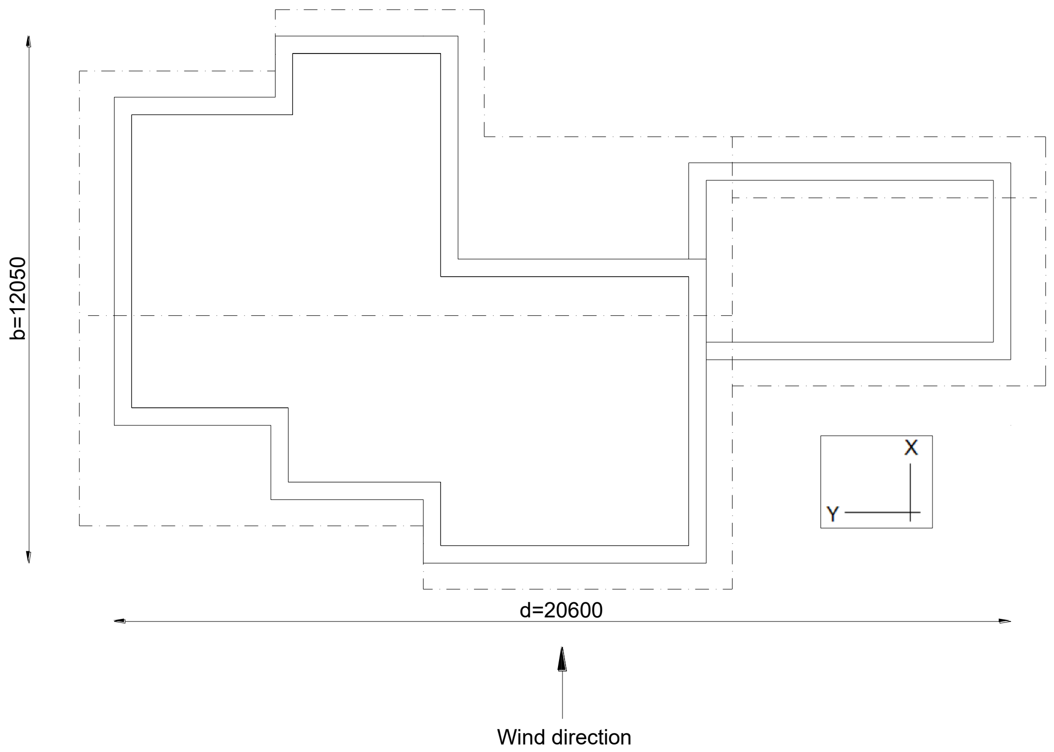
Overall wind force, EN1991-1-4(5.3). $F_{w.Ydirection} := c_f \cdot c_s \cdot c_d \cdot q_p = [0.474] \ \frac{kN}{m^2}$

Conclusion.

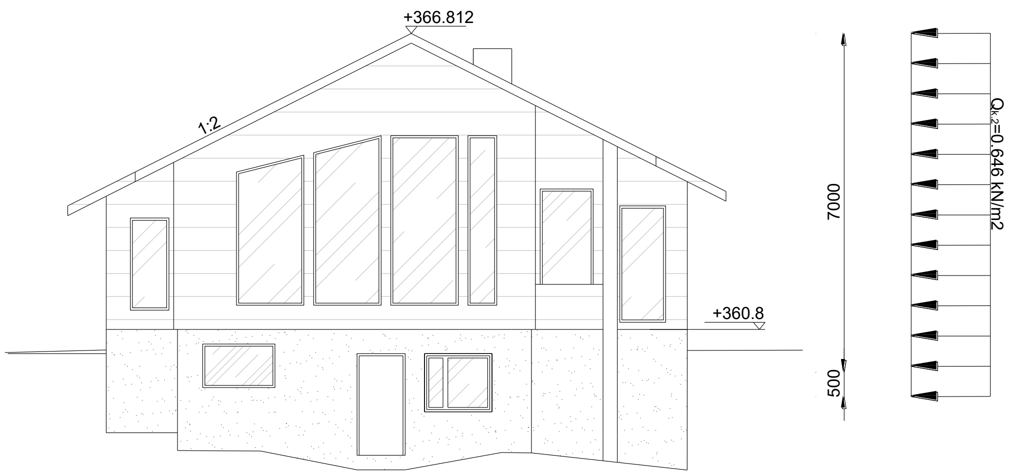
Symbol according to Eurocode. $Q_{k.3} := F_{w.Ydirection} = [0.474] \ \frac{kN}{m^2}$

Appendix 4: Wind load on X-direction, simplified method

TOP VIEW



FRONT VIEW



Reference height $h_1 := 7000\ mm$

Plinth visible against the ground. Recommended values are between 300 mm to 500 mm. $h_2 := 500\ mm$

Height of the building. $h := h_1 + h_2 = 7500\ mm$

Terrain parameter, terrain category III, from table 4.1 EN-1991-1-4. $z_0 := 0.3\ m$

Terrain parameter, terrain category III, from table 4.1 EN-1991-1-4. $z_{min} := 5\ m$

Terrain parameter, terrain category III, from table 4.1 EN-1991-1-4. $z_{max} := 200\ m$

Terrain category II, from table 4.1 EN-1991-1-4, 4.3.2. $z_{0.II} := 0.05\ m$

Condition to be checked according to EN-1991-1-4, 7.2.2,

$$z := \begin{cases} h & \text{if } h < b \\ \text{"check"} & \text{else} \end{cases} = 7.5\ m$$

Turbulence factor, from EN 1991-1-4, 4.4, (4.7). $K_I := 1$

Orography factor explained in 4.3.3 EN-1991-1-4. $c_0 := 1$

Air density. $\rho := 1.25\ \dfrac{kg}{m^3}$

$z_{min} \le z \le z_{max}$ therefore:

Turbulence intensity, from 4.4 EN-1991-1-4.

$$I_v := \frac{K_I}{c_0 \cdot \ln\left(\dfrac{z}{z_0}\right)} = 0.311$$

Terrain factor based on the roughness length z_0, from 4.3.2, (4.5) EN-1991-1-4.

$$K_r := 0.19 \cdot \left(\frac{z_0}{z_{0.II}}\right)^{0.07} = 0.215$$

$z_{min} \le z \le z_{max}$ therefore:

Roughness factor, 4.3.2, (4.4), EN-1991-1-4.

$$c_r := K_r \cdot \ln\left(\frac{z}{z_0}\right) = 0.693$$

Wind velocity

Directional factor. $c_{dir} := 1$

Seasonal factor. $c_{season} := 1$

Fundamental value of basic wind velocity, conservative value. $v_{b.0} := 22 \frac{m}{s}$

Basic wind velocity, from 4.2, (4.1), EN-1991-1-4. $v_b := c_{dir} \cdot c_{season} \cdot v_{b.0} = 22 \frac{m}{s}$

Mean wind

Mean wind velocity, from 4.3.1, (4.3), EN-1991-1-4. $v_m := v_b \cdot c_0 \cdot c_r = 15.253 \frac{m}{s}$

Peak velocity pressure, from 4.5, (4.8), EN-1991-1-4. $q_p := \left[1 + 7 \cdot I_v\right] \cdot \frac{1}{2} \cdot \rho \cdot {v_m}^2$

Peak velocity pressure. $q_p = \left[461.616\right] \frac{N}{m^2}$

Characteristic pressure on the wall for checking the stability of the building

Height less than 15 meters. $h = 7500\ mm$

From EN 1991-1-4, 5.3, (5.4). $c_s := 1$

From EN 1991-1-4, 5.3, (5.4). $c_d := 1$

	d/b								
λ	0.1	0.2	0.5	0.7	1	2	5	10	50
≤ 1	1.2	1.2	1.37	1.44	1.28	0.99	0.6	0.54	0.54
3	1.29	1.29	1.48	1.55	1.38	1.07	0.65	0.58	0.58
10	1.4	1.4	1.6	1.68	1.49	1.15	0.7	0.63	0.63

Interpolation using the table from RIL 201-1-2008.

Width of the building perpendicular to the wind direction. $b := 20.6\ m$

The width of the building parallel to the wind direction. $d := 12.05\ m$

Ratio $\frac{d}{b} = 0.585$

Slenderness defined from RIL 201-1-2008 $\lambda_{RIL} := \frac{2 \cdot h}{b} = 0.728$

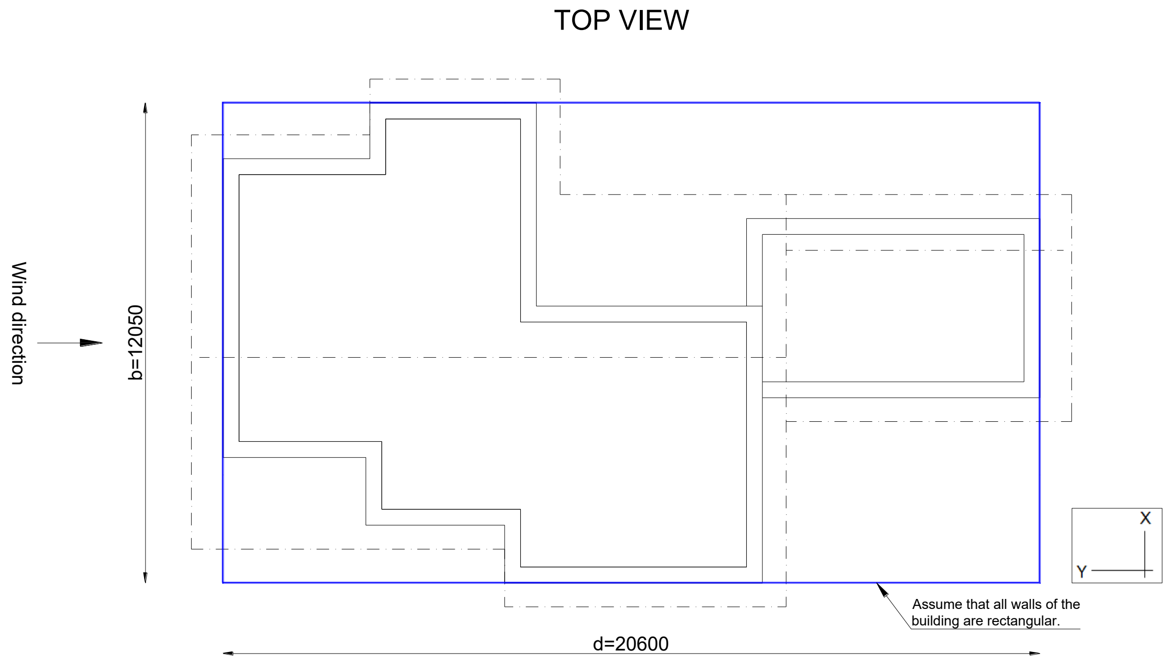
Defining c_f factor. $c_f := 1.37 + \frac{(0.585 - 0.5)}{(0.7 - 0.5)} \cdot (1.44 - 1.37) = 1.4$

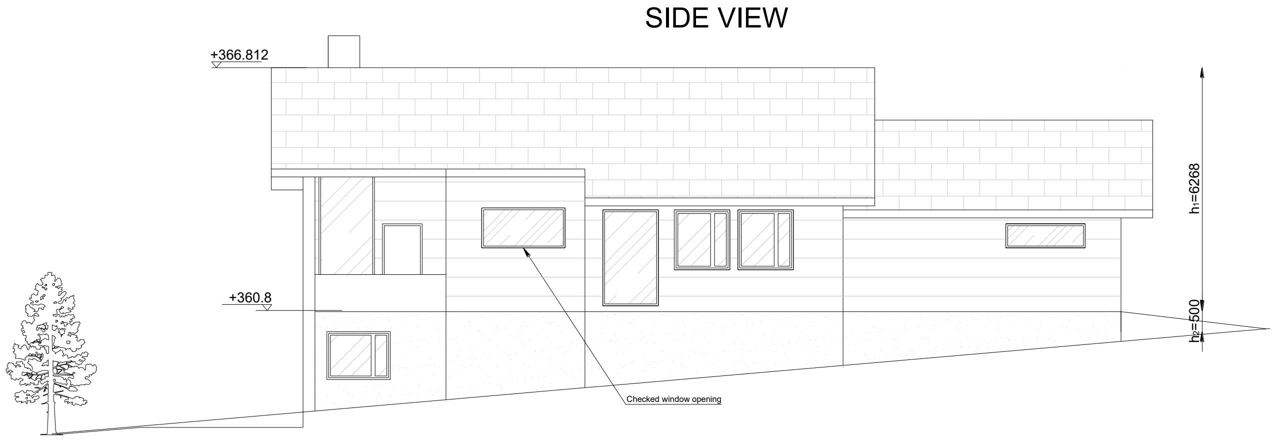
Overall wind force, EN1991-1-4 (5.3). $F_{w.Xdirection} := c_f \cdot c_s \cdot c_d \cdot q_p = [0.646]\ \frac{kN}{m^2}$

Conclusion.

Symbol according to Eurocode. $Q_{k.3} := F_{w.Xdirection} = [0.646]\ \frac{kN}{m^2}$

Appendix 5: Wind load on Y-direction, exact method





Precursory height. $h_1 := 6268\ mm$

Plinth visible against the ground. Recommended values are between 300 mm to 500 mm. $h_2 := 500\ mm$

Height of the building. $h := h_1 + h_2 = 6768\ mm$

Width of the building perpendicular to the wind direction. $b := 12050\ mm$

Width of the building parallel to the wind direction. $d := 20600\ mm$

Terrain parameter, terrain category III, from table 4.1 EN-1991-1-4. $z_0 := 0.3\ m$

Terrain parameter, terrain category III, from table 4.1 EN-1991-1-4. $z_{min} := 5\ m$

$z_{max} := 200\ m$

Terrain parameter, terrain category III, from table 4.1 EN-1991-1-4.

Terrain category II, from table 4.1 EN-1991-1-4. $z_{0.II} := 0.05\ m$

Condition.

$$z := \begin{array}{l} \text{if } h < b \\ \quad \| \; h \\ \text{else} \\ \quad \| \; \text{“check”} \end{array} = 6.768\ m$$

Turbulence factor, from EN 1991-1-4. $K_I := 1$

Orography factor explained in 4.3.3 EN-1991-1-4. $c_0 := 1$

Air density. $\rho := 1.25\ \frac{kg}{m^3}$

$z_{min} \le z \le z_{max}$ therefore:

Turbulence intensity, from 4.4 EN-1991-1-4.

$$I_v := \frac{K_I}{c_0 \cdot \ln\left(\frac{z}{z_0}\right)} = 0.321$$

Terrain factor based on the roughness length z_0, from 4.3.2, (4.5) EN-1991-1-4.

$$K_r := 0.19 \cdot \left(\frac{z_0}{z_{0.II}}\right)^{0.07} = 0.215$$

$z_{min} \le z \le z_{max}$ therefore:

Roughness factor, 4.3.2, (4.4), EN-1991-1-4.

$$c_r := K_r \cdot \ln\left(\frac{z}{z_0}\right) = 0.671$$

Wind velocity

Directional factor. $c_{dir} := 1$

Seasonal factor. $c_{season} := 1$

Fundamental value of basic wind velocity, conservative value. $v_{b.0} := 22\ \frac{m}{s}$

Basic wind velocity, from 4.2, (4.1), EN-1991-1-4. $v_b := c_{dir} \cdot c_{season} \cdot v_{b.0} = 22\ \frac{m}{s}$

Mean wind

Mean wind velocity, from 4.3.1, (4.3), EN-1991-1-4. $v_m := v_b \cdot c_0 \cdot c_r = 14.766\ \frac{m}{s}$

Peak velocity pressure, from 4.5, (4.8), EN-1991-1-4. $q_p := \left[1 + 7 \cdot I_v\right] \cdot \frac{1}{2} \cdot \rho \cdot {v_m}^2$

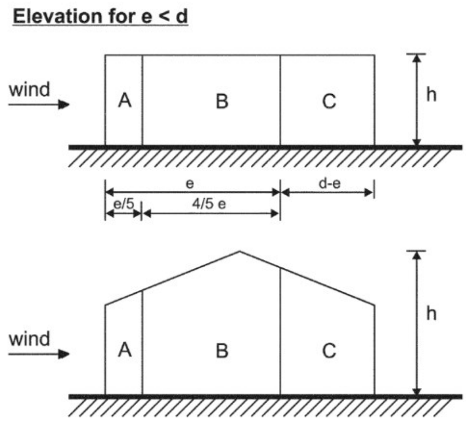
Result $q_p = \left[442.397\right]\ \frac{N}{m^2}$

EN 1991-1-4, 7.2.2 (2). $e := min\left(b, 2 \cdot h\right) = 12.05\ m$

Condition

$$\begin{array}{l|l} \text{if } e < d & = \text{“true”} \\ \quad \text{“true”} & \\ \text{else} & \\ \quad \text{“false”} & \end{array}$$

Follow the instruction according to the below figure.



Length of A face. $A := \frac{e}{5} = 2.41\ m$

Length of B face. $B := \frac{4}{5} \cdot e = 9.64\ m$

Length of C face. $C := d - e = 8.55\ m$

Check. $A + B + C = 20.6\ m$

Area is bigger than $10\ m^2$. $A_{zone.A} := A \cdot h = 16.311\ m^2$

Area is bigger than $10\ m^2$. $A_{zone.B} := B \cdot h = 65.244\ m^2$

Area is bigger than $10\ m^2$. $A_{zone.C} := C \cdot h = 57.866\ m^2$

Area is bigger than $10\ m^2$. $A_{zone.D} := b \cdot h = 81.554\ m^2$

Area is bigger than $10\ m^2$. $A_{zone.E} := b \cdot h = 81.554\ m^2$

Height to length ratio, to select the correct external pressure coefficients. $Ratio := \frac{h}{d} = 0.329$

Condition.

$$\begin{array}{l} \text{if } 0.25 \le Ratio \le 1 \\ \quad \| \text{"true"} \\ \text{else} \\ \quad \| \text{"false"} \end{array} = \text{"true"}$$

Values are taken from the below table provided in EN-1991-1-4, 7.2.2(2).

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	

From table 7.1, EN-1991-1-4. $C_{p.A} := -1.2$
From table 7.1, EN-1991-1-4. $C_{p.B} := -0.8$
From table 7.1, EN-1991-1-4. $C_{p.C} := -0.5$
From table 7.1, EN-1991-1-4. $C_{p.D} := 0.8$
From table 7.1, EN-1991-1-4. $C_{p.E} := -0.5$

Internal pressure should be taken into account. Internal pressure coefficients are +0.2 and -0.3. When there is a negative value of Cp,10 negative internal pressure should be taken into account and subtracted. The same is if the value of Cp,10 is positive, but in this case positive internal pressure should be taken into account. Therefore the final values for the calculations will be as follows:

Internal pressure coefficients.

$$C_{i.A} := 0.2$$
$$C_{i.B} := 0.2$$
$$C_{i.C} := 0.2$$
$$C_{i.D} := 0.2$$
$$C_{i.E} := 0.2$$

External pressure coefficients.

$$C_{e.A} := -0.3$$
$$C_{e.B} := -0.3$$
$$C_{e.C} := -0.3$$
$$C_{e.D} := -0.3$$
$$C_{e.E} := -0.3$$

Pressure coefficient for the external pressure.

$$C_{p.A} := C_{p.A} - C_{i.A} = -1.4$$
$$C_{p.B} := C_{p.B} - C_{i.B} = -1$$
$$C_{p.C} := C_{p.C} - C_{i.C} = -0.7$$
$$C_{p.D} := C_{p.D} - C_{e.D} = 1.1$$
$$C_{p.E} := C_{p.E} - C_{i.E} = -0.7$$

Wind pressure on surfaces, from 5.2, (5.1), EN-1991-1-4. $W = q_p \cdot C_{pe}$ therefore:

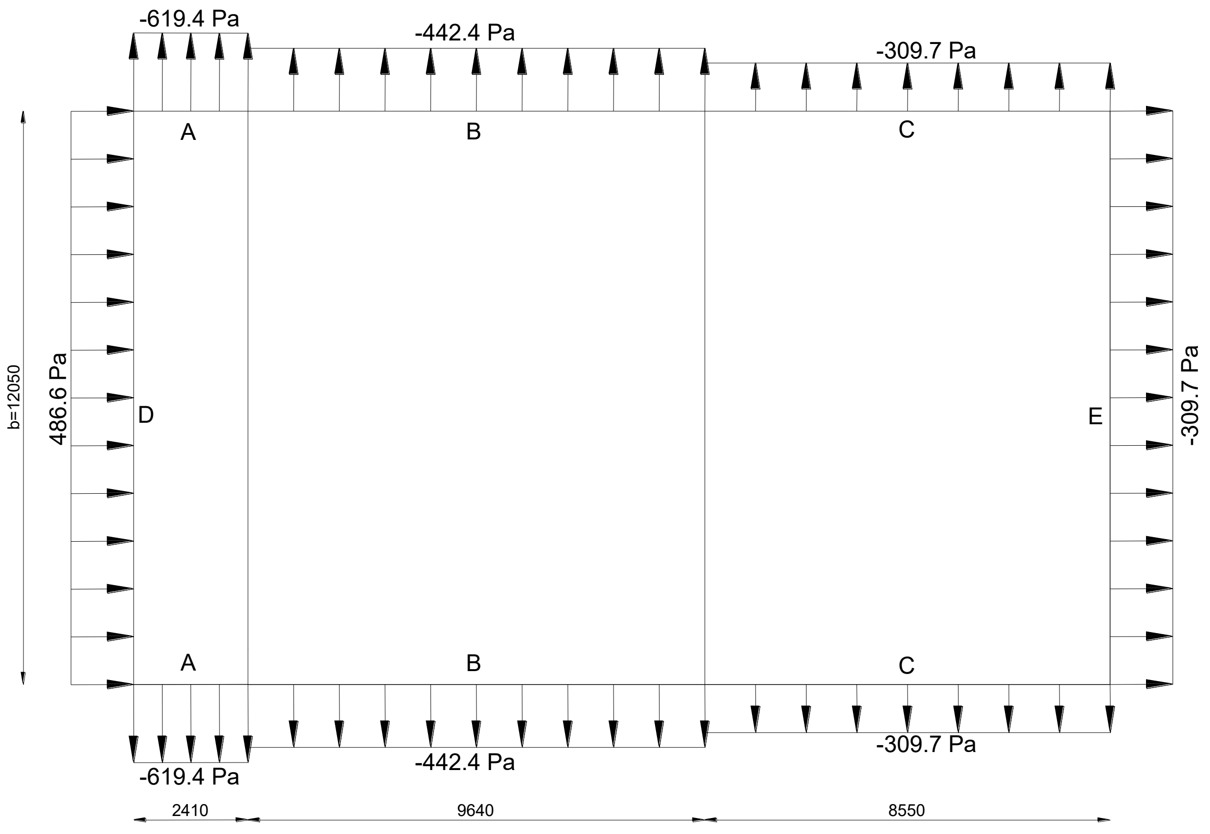
Suction. $W_A := C_{p.A} \cdot q_p = [-619.4]\ Pa$

Suction. $W_B := C_{p.B} \cdot q_p = [-442.4]\ Pa$

Suction. $W_C := C_{p.C} \cdot q_p = [-309.7]\ Pa$

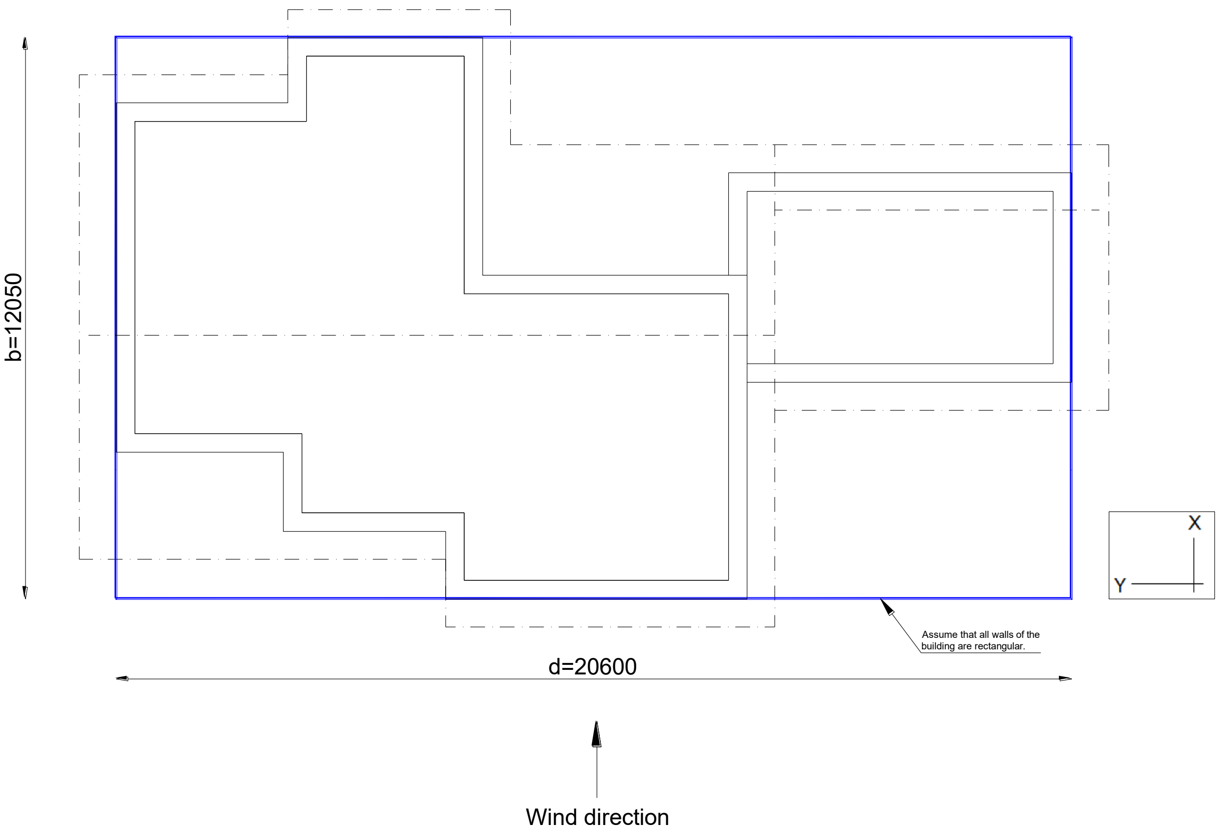
Compression. $W_D := C_{p.D} \cdot q_p = [486.6]\ Pa$

Suction. $W_E := C_{p.E} \cdot q_p = [-309.7]\ Pa$


-619.4 Pa
-442.4 Pa
-309.7 Pa
A
B
C
D
E
486.6 Pa
-309.7 Pa
b=12050
A
B
C
-619.4 Pa
-442.4 Pa
-309.7 Pa
2410
9640
8550

Appendix 6: Wind load on X-direction, exact method

TOP VIEW



BACK VIEW

Precursory height. $h_1 := 6113\ mm$

Plinth visible against the ground. Recommended values are between 300 mm to 500 mm. $h_2 := 500\ mm$

Height of the building. $h := h_1 + h_2 = 6613\ mm$

Width of the building perpendicular to the wind direction. $b := 20600\ mm$

Width of the building parallel to the wind direction. $d := 12050\ mm$

Terrain parameter, terrain category III, from table 4.1 EN-1991-1-4. $z_0 := 0.3\ m$

Terrain parameter, terrain category III, from table 4.1 EN-1991-1-4. $z_{min} := 5\ m$

Terrain parameter, terrain category III, from table 4.1 EN-1991-1-4. $z_{max} := 200\ m$

Terrain category II, from table 4.1 EN-1991-1-4. $z_{0.II} := 0.05\ m$

Condition. $z := \text{if } h < b \ \| \ h \ \text{else} \ \| \ \text{"check"} = 6613\ mm$

Turbulence factor, from EN 1991-1-4. $K_I := 1$

Orography factor explained in 4.3.3 EN-1991-1-4. $c_0 := 1$

Air density. $\rho := 1.25\ \dfrac{kg}{m^3}$

$z_{min} \le z \le z_{max}$ therefore:

Turbulence intensity, from 4.4 EN-1991-1-4. $I_v := \dfrac{K_I}{c_0 \cdot \ln\left(\dfrac{z}{z_0}\right)} = 0.323$

Terrain factor based on the roughness length z_0, from 4.3.2, (4.5) EN-1991-1-4. $K_r := 0.19 \cdot \left(\dfrac{z_0}{z_{0.II}}\right)^{0.07} = 0.215$

$z_{min} \le z \le z_{max}$ therefore:

Roughness factor, 4.3.2, (4.4), EN-1991-1-4. $c_r := K_r \cdot \ln\left(\dfrac{z}{z_0}\right) = 0.666$

Wind velocity

Directional factor. $c_{dir} := 1$

Seasonal factor. $c_{season} := 1$

Fundamental value of basic wind velocity, conservative value. $v_{b.0} := 22 \ \frac{m}{s}$

Basic wind velocity, from 4.2, (4.1), EN-1991-1-4. $v_b := c_{dir} \cdot c_{season} \cdot v_{b.0} = 22 \ \frac{m}{s}$

Mean wind

Mean wind velocity, from 4.3.1, (4.3), EN-1991-1-4. $v_m := v_b \cdot c_0 \cdot c_r = 14.656 \ \frac{m}{s}$

Peak velocity pressure, from 4.5, (4.8), EN-1991-1-4. $q_p := \left[1 + 7 \cdot I_v\right] \cdot \frac{1}{2} \cdot \rho \cdot {v_m}^2$

Result. $q_p = \left[438.103\right] \ \frac{N}{m^2}$

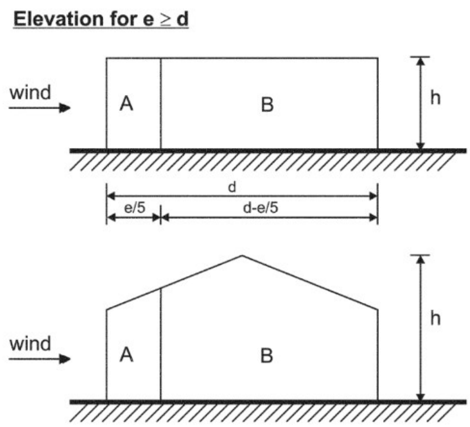
EN 1991-1-4, 7.2.2 (2). $e := min\left(b, 2 \cdot h\right) = 13.226 \ m$

Condition.

if $e \geq d$
‖ "true"
else
‖ "false"

= "true"

Follow the instruction according to the below figure.



Length of A face. $A := \frac{e}{5} = 2.645\ m$

Length of B face. $B := d - \frac{e}{5} = 9.405\ m$

Check. $A + B = 12.05\ m$

Area is bigger than $10\ m^2$. $A_{zone.A} := A \cdot h = 17.493\ m^2$

Area is bigger than $10\ m^2$. $A_{zone.B} := B \cdot h = 62.194\ m^2$

Area is bigger than $10\ m^2$. $A_{zone.D} := b \cdot h = 136.228\ m^2$

Area is bigger than $10\ m^2$. $A_{zone.E} := b \cdot h = 136.228\ m^2$

Height to length ratio, to select the correct external pressure coefficients. $Ratio := \frac{h}{d} = 0.549$

Condition.

$$\begin{array}{l} \text{if } 0.25 \le Ratio \le 1 \\ \quad \| \text{“true”} \\ \text{else} \\ \quad \| \text{“false”} \end{array} = \text{“true”}$$

Values are taken from the below table provided in EN-1991-1-4, 7.2.2(2).

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	

From table 7.1, EN-1991-1-4. $C_{p.A} := -1.2$
From table 7.1, EN-1991-1-4. $C_{p.B} := -0.8$
From table 7.1, EN-1991-1-4. $C_{p.D} := 0.8$
From table 7.1, EN-1991-1-4. $C_{p.E} := -0.5$

Internal pressure should be taken into account. Internal pressure coefficients are +0.2 and -0.3. When there is a negative value of Cp,10 negative internal pressure should be taken into account and subtracted. The same is true if the value of Cp,10 is positive, but in this case, positive internal pressure should be considered. Therefore the final values for the calculations will be as following:

Internal pressure coefficients.

$C_{i.A} := 0.2$
$C_{i.B} := 0.2$
$C_{i.D} := 0.2$
$C_{i.E} := 0.2$

External pressure coefficients.

$C_{e.A} := -0.3$
$C_{e.B} := -0.3$
$C_{e.D} := -0.3$
$C_{e.E} := -0.3$

Pressure coefficient for the external pressure.

$C_{p.A} := C_{p.A} - C_{i.A} = -1.4$
$C_{p.B} := C_{p.B} - C_{i.B} = -1$
$C_{p.D} := C_{p.D} - C_{e.D} = 1.1$
$C_{p.E} := C_{p.E} - C_{i.E} = -0.7$

Wind pressure on surfaces, from 5.2, (5.1), EN-1991-1-4.

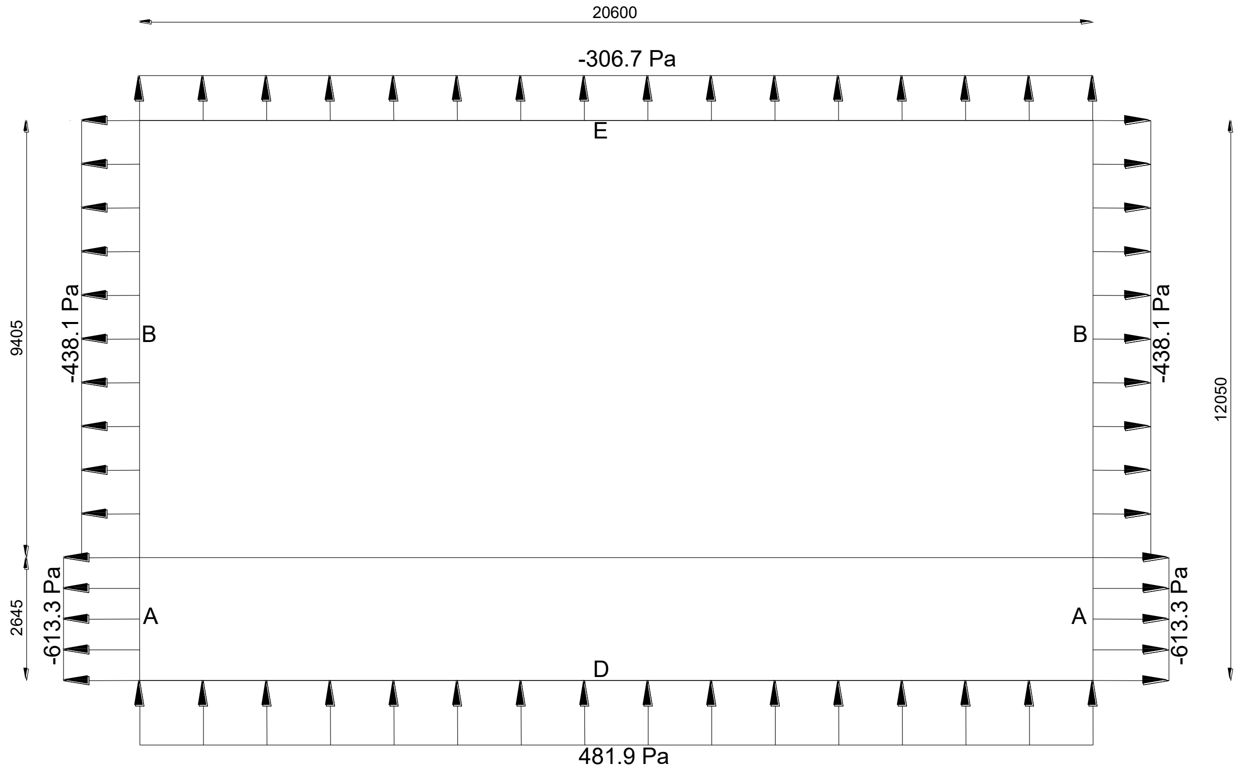
$W = q_p \cdot C_{pe}$ therefore:

Suction. $W_A := C_{p.A} \cdot q_p = \left[-613.3 \right] \ Pa$

Suction. $W_B := C_{p.B} \cdot q_p = \left[-438.1 \right] \ Pa$

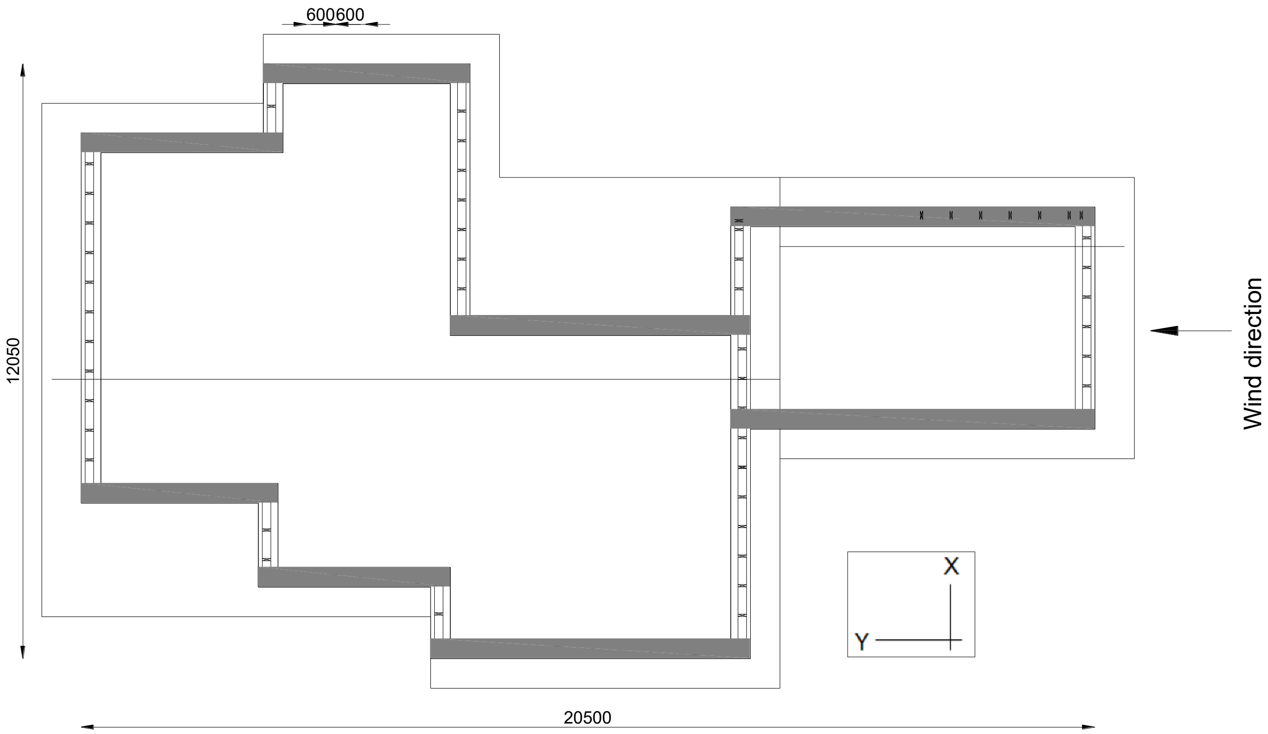
Compression. $W_D := C_{p.D} \cdot q_p = \left[481.9 \right] \ Pa$

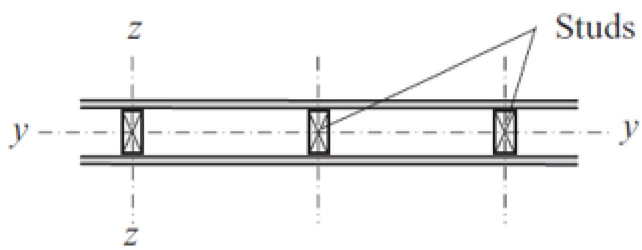
Suction. $W_E := C_{p.E} \cdot q_p = \left[-306.7 \right] \ Pa$


20600
-306.7 Pa
E
9405
-438.1 Pa
B
B
-438.1 Pa
12050
2645
-613.3 Pa
A
A
-613.3 Pa
D
481.9 Pa

Appendix 7: Stud on Y-direction for the critical load combination

All load combinations are checked, and the modification factor corresponds to the short-term load duration. In this appendix, only the critical load combination is shown. Load combination three will be shown as it is more critical for axial and combined bending.





Stud design.

Sheathing material provides enough lateral support about the weaker axis(z-z axis).

Timber properties

Material of the studs	C24 sawn timber
Bending strength.	$f_{m.k} := 24\ MPa$
Compression parallel to the grain.	$f_{c.0.k} := 21\ MPa$
Compression perpendicular to the grain.	$f_{c.90.k} := 2.5\ MPa$
5% modulus of elasticity parallel.	$E_{0.05} := 7.4\ GPa$
Service class.	Class 2

Initial data.

Width of studs.	$b := 48\ mm$
Length of joists.	$h := 173\ mm$
The longest span of the roof.	$R_1 := 12.05\ m$
Spacing of studs.	$k_1 := 600\ mm$
Tributary length.	$L_1 := \frac{R_1}{2} = 6.025\ m$
Design load in ULS.	$q_{Ed} = 4.615\ \frac{kN}{m^2}$
Height of the studs.	$L_2 := 2.5\ m$
Load coming in the center of the stud.	$N_{Ed} := q_{Ed} \cdot k_1 \cdot L_1 = 16.685\ kN$
Characteristic dead load.	$G_{k.3} = 0.883\ \frac{kN}{m^2}$
Characteristic snow load.	$Q_{k.1} = 2.4\ \frac{kN}{m^2}$
Characteristic wind load in X-direction.	$Q_{k.2} = 0.646\ \frac{kN}{m^2}$
Load coming in the center of the stud form dead load.	$N_D := G_{k.3} \cdot k_1 \cdot L_1 = 3.192\ kN$
Load coming in the center of the stud from snow load.	$N_S := Q_{k.1} \cdot k_1 \cdot L_1 = 8.676\ kN$
Load coming in the center of the stud from the wind load.	$N_W := 0\ kN$

Axial load, no bending moment occurs from the dead load. $M_D := 0\ kN \cdot m$

Axial load, no bending moment occurs from snow load. $M_S := 0\ kN \cdot m$

Bending moment from wind load. $M_W := \frac{Q_{k.2} \cdot k_1 \cdot {L_2}^2}{8} = 0.303\ kN \cdot m$

The coefficient for snow load. $\psi_0 := 0.7$

Ultimate Limit State combinations for normal force.

$$Combination1 := 1.35 \cdot K_{FI} \cdot N_D = 4.309\ kN$$

$$Combination2 := 1.15 \cdot K_{FI} \cdot N_D + 1.5 \cdot K_{FI} \cdot N_S = 16.685\ kN$$

$$Combination3 := 1.15 \cdot K_{FI} \cdot N_D + 1.5 \cdot K_{FI} \cdot N_S + 1.5 \cdot K_{FI} \cdot \psi_0 \cdot N_W = 16.685\ kN$$

$$Combination4 := 1.15 \cdot K_{FI} \cdot N_D + 1.5 \cdot K_{FI} \cdot N_W = 3.671\ kN$$

$$Combination5 := 1.15 \cdot K_{FI} \cdot N_D + 1.5 \cdot K_{FI} \cdot N_W + 1.5 \cdot K_{FI} \cdot \psi_0 \cdot N_S = 12.781\ kN$$

Ultimate Limit State combinations for bending moment.

$$Combination1 := 1.35 \cdot K_{FI} \cdot M_D = 0\ kN \cdot m$$

$$Combination2 := 1.15 \cdot K_{FI} \cdot M_D + 1.5 \cdot K_{FI} \cdot M_S = 0\ kN \cdot m$$

$$Combination3 := 1.15 \cdot K_{FI} \cdot M_D + 1.5 \cdot K_{FI} \cdot M_S + 1.5 \cdot K_{FI} \cdot \psi_0 \cdot M_W = 0.318\ m \cdot kN$$

$$Combination4 := 1.15 \cdot K_{FI} \cdot M_D + 1.5 \cdot K_{FI} \cdot M_W = 0.454\ m \cdot kN$$

$$Combination5 := 1.15 \cdot K_{FI} \cdot M_D + 1.5 \cdot K_{FI} \cdot M_W + 1.5 \cdot K_{FI} \cdot \psi_0 \cdot M_S = 0.454\ m \cdot kN$$

Action	Permanent	Permanent+medium	Permanent+medium+short	Permanent+short	Permanent+medium+short	Unit
	Combination 1	Combination 2	Combination 3	Combination 4	Combination 5	
N_Ed	4.309	16.685	16.685	3.671	12.781	kN
M_Ed	0	0	0.318	0.454	0.454	kN*m
k_mod	0.6	0.8	0.9	0.9	0.9	

Load combination 3; critical combination for compressive and combined

Design normal force for combination 3. $N_{Ed.C3} := 16.685\ kN$

Design bending moment for combination 3. $M_{Ed.C3} := 0.318\ kN \cdot m$

Modification factor, EN1995-1-1, 3.2, table 3.1, permanent action. $k_{mod.C3} := 0.9$

Buckling coefficient about the y axis. $k_{ey} := 1$

Buckling length about the y axis. $L_{ey} := L_2 \cdot k_{ey} = 2.5\ m$

Net area of the studs. $A_s := b \cdot h = 8304\ mm^2$

Moment of inertia about the y axis. $I_y := \frac{b \cdot h^3}{12} = \left(2.07 \cdot 10^7\right)\ mm^4$

The radius of gyration about the y axis. $r_y := \sqrt{\frac{I_y}{A_s}} = 49.94\ mm$

Slenderness ratio. $\lambda_y := \frac{L_{ey}}{r_y} = 50.059$

Slenderness ratio corresponding to bending about y-axis(deflection in the z-direction), EN1995-1-1, 6.3.2(6.21) $\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} = 0.849$

Slenderness ratio greater than 0.3. $\lambda_{rel.y} > 0.3$

Section modulus. $W_y := \frac{b \cdot h^2}{6} = \left(2.394 \cdot 10^5\right)\ mm^3$

Net area of the studs. $A_s := b \cdot h = 8304\ mm^2$

Compressive stress along the grain. $\sigma_{c.0.d} := \frac{N_{Ed.C3}}{A_s} = 2.009\ MPa$

EN1995-1-1, 6.1.6(2). $k_m := 0.7$

The factor for members within the straightness limits, EN1995-1-1, 6.3.2(6.29). $\beta_c := 0.2$

Bending stress about the y $\sigma_{m.y.d} := \frac{M_{Ed.C3}}{W_y} = 1.328\ MPa$

EN-1995-1-1, 6.3.2 (3), 6.27.

$$k_y := 0.5\left(1+\beta_c \cdot \left(\lambda_{rel.y}-0.3\right)+{\lambda_{rel.y}}^2\right)=0.915$$

To consider the buckling effect, Buckling deduction coefficients, EN1995-1-1, 6.3.2(6.25).

$$k_{c.y} := \frac{1}{k_y+\sqrt{{k_y}^2-{\lambda_{rel.y}}^2}}=0.795$$

Modification factor, EN1995-1-1, 3.2, table 3.1.

$$k_{mod.C3}=0.9$$

System strength factor EN1995-1-1, 6.6, Figure 6.12

$$k_{sys} := 1$$

Partial factor, EN1995-1-1, 2.4.1, Table 2.3.

$$\gamma_M := 1.3$$

Reference depth in bending, EN1995-1-1, 3.2(3.1).

$$k_h := min\left(1.3, \left(\frac{150\ mm}{h}\right)^{0.2}\right)=0.972$$

Design bending strength about the y axis.

$$f_{m.y.d} := k_{mod.C3} \cdot \frac{f_{m.k}}{\gamma_M} \cdot k_h \cdot k_{sys}=16.148\ MPa$$

Design compressive strength along the grain.

$$f_{c.0.d} := k_{mod.C3} \cdot k_{sys} \cdot \frac{f_{c.0.k}}{\gamma_M}=14.538\ MPa$$

Compression.

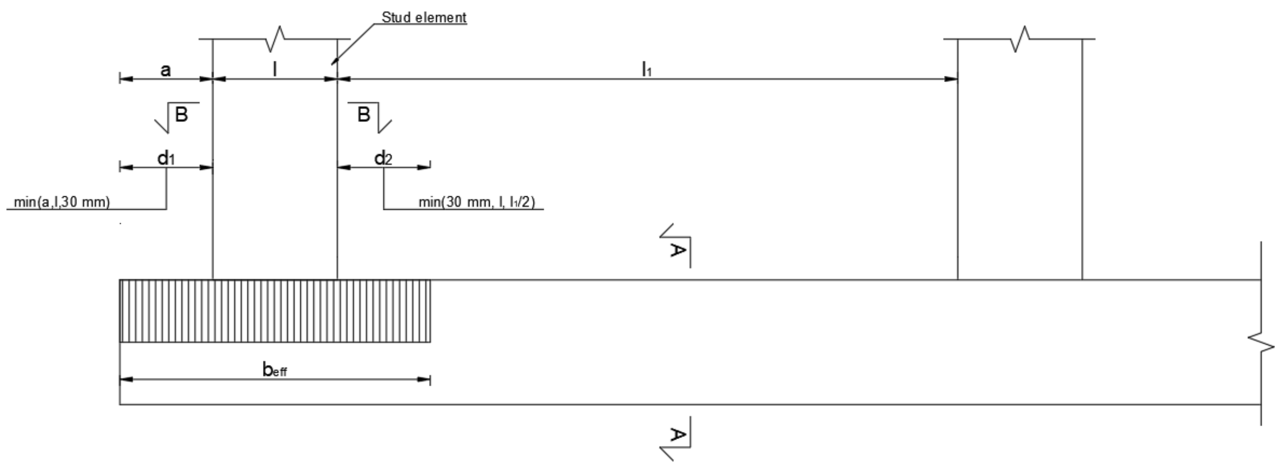
$$UR_{StudCompression} := \begin{cases} \dfrac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} & \text{if } \sigma_{c.0.d} \le k_{c.y} \cdot f_{c.0.d} \\ 0 & \text{else} \end{cases} = 17\%$$

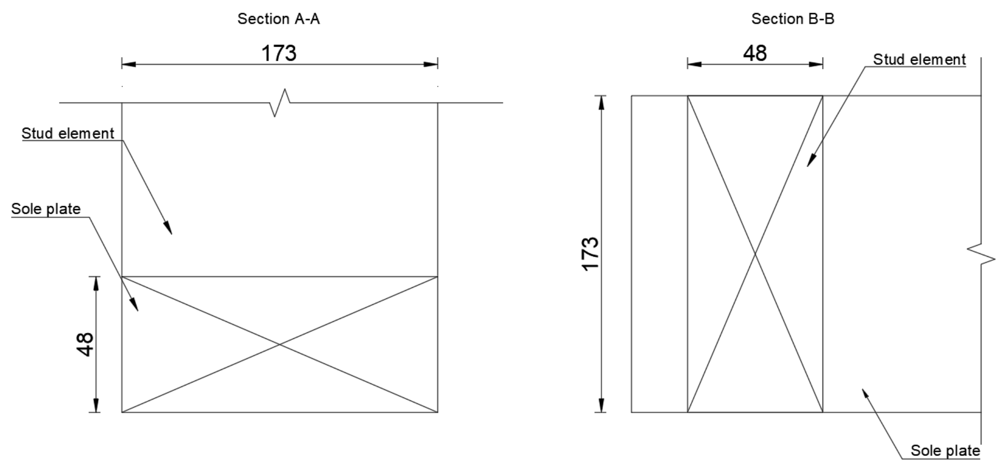
Combined compression and bending.

$$UR_{combined} := \text{if } \frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}}+\frac{\sigma_{m.y.d}}{f_{m.y.d}} \ \Vert\ \text{“okay”} \ \text{else} \ \Vert\ \text{“check calculations”} = \text{“okay”}$$

$$UR_{combined} := \frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}}+\frac{\sigma_{m.y.d}}{f_{m.y.d}}=26\%$$

Bearing Capacity of the soleplate (to foundation wall).





Stud width. $l := 48\ mm$

Studs are placed center to center, no side gap left. $a := 0\ mm$

Stud spacing. $l_1 := 600\ mm$

Distance from the left side. $d_1 := min\left(30\ mm, a, l\right) = 0\ mm$

Distance from the right side. $d_2 := min\left(30\ mm, l, \frac{l_1}{2}\right) = 30\ mm$

Effective length. $b_{eff} := b + d_1 + d_2 = 78\ mm$

The effective area of the stud. $A_{eff} := b_{eff} \cdot h = \left(1.349 \cdot 10^4\right)\ mm^2$

Design compressive stress along the grain. $\sigma_{c.90.d} := \frac{N_{Ed.C3}}{A_{eff}} = 1.236\ MPa$

Design compressive strength along the grain. $f_{c.90.d} := k_{mod.C3} \cdot k_{sys} \cdot \frac{f_{c.90.k}}{\gamma_M}$

Spacing between stud elements. $l_1 := 600\ mm$

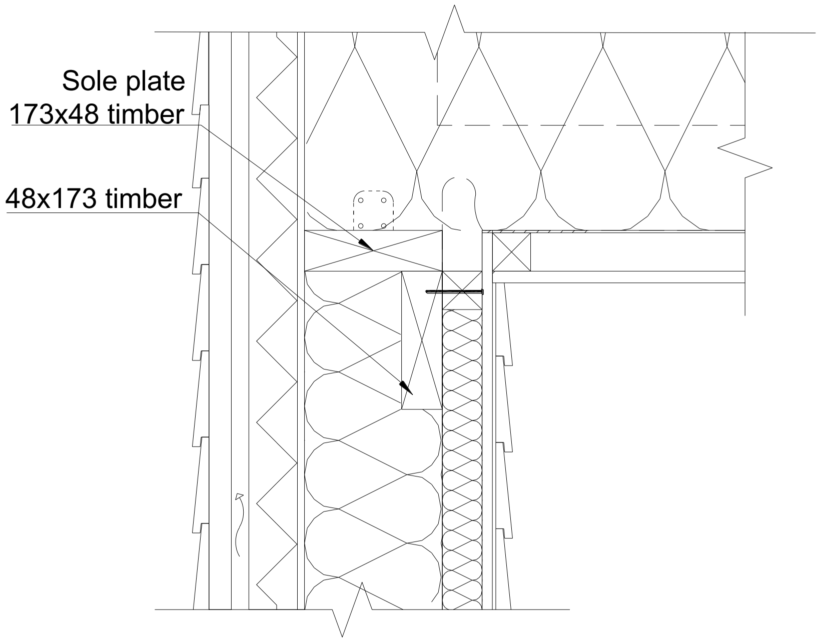
Bearing contact length $b_1 := 173\ mm$

Height of soleplate. $h_1 := 48\ mm$

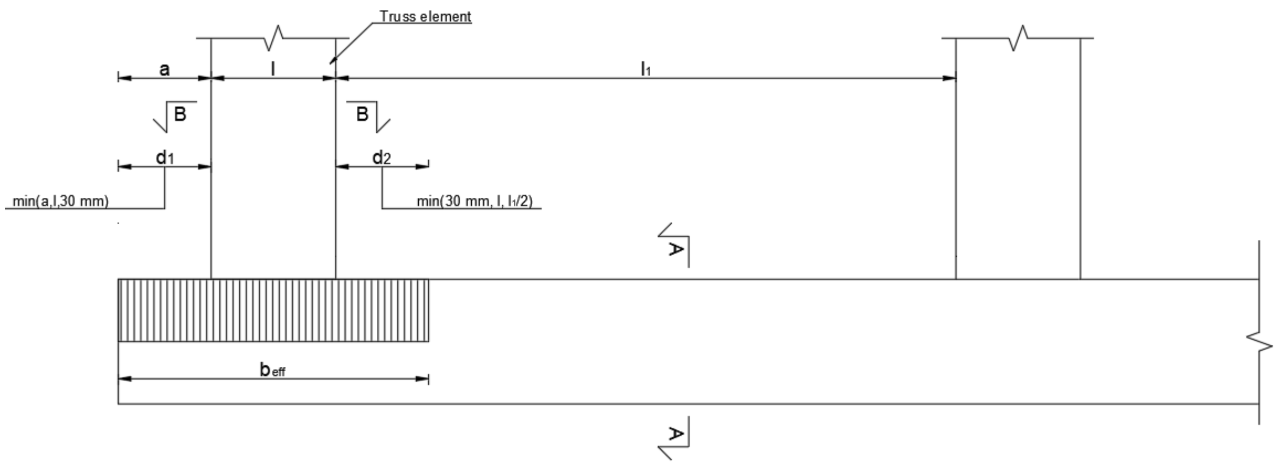
Factor taking into account the load configuration. $k_{c.90} := \text{if } l_1 \geq 2 \cdot h_1 \ \| \ 1.25 \ \text{else} \ \| \ 1 \ = 1.25$

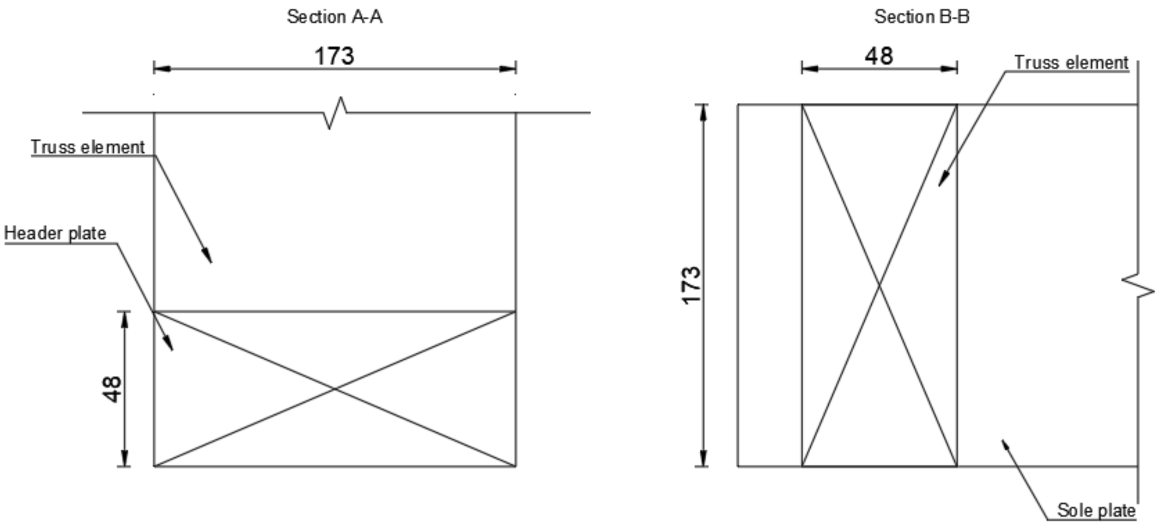
Utilization ratio for soleplate. $UR_{BearingCapacity1} := \frac{\sigma_{c.90.d}}{k_{c.90} \cdot f_{c.90.d}} = 57\%$

Bearing Capacity of the header plate (truss-stud connection).



For the header plate, on the bottom of it, there will be the installation of the same type of timber, but vertically as shown in the above figure. Therefore, this would be continuous support for the header plate, and the same procedure as before applies.





Truss width $l := 48\ mm$

Studs are placed center to center, no side gap left. $a := 0\ mm$

Truss spacing. $l_1 := 900\ mm$

Distance from the left side. $d_1 := min\left(30\ mm, a, l\right) = 0\ mm$

Distance from the right side. $d_2 := min\left(30\ mm, l, \frac{l_1}{2}\right) = 30\ mm$

Effective length. $b_{eff} := b + d_1 + d_2 = 78\ mm$

The effective area of the stud. $A_{eff} := b_{eff} \cdot h = \left(1.349 \cdot 10^4\right)\ mm^2$

Design compressive stress along the grain. $\sigma_{c.90.d} := \frac{N_{Ed.C3}}{A_{eff}} = 1.236\ MPa$

Design compressive strength along the grain. $f_{c.90.d} := k_{mod.C3} \cdot k_{sys} \cdot \frac{f_{c.90.k}}{\gamma_M}$

Spacing between truss elements. $l_1 := 900\ mm$

Bearing contact length $b_1 := 173\ mm$

Height of soleplate. $h_1 := 48\ mm$

Factor taking into account the load configuration.

$$k_{c.90} := \begin{array}{|l} \text{if } l_1 \ge 2 \cdot h_1 \\ \quad \| 1.25 \\ \text{else} \\ \quad \| 1 \end{array} = 1.25$$

Utilization ratio. $UR_{BearingCapacity2} := \dfrac{\sigma_{c.90.d}}{k_{c.90} \cdot f_{c.90.d}} = 57\%$

Conclusions for combination 3.

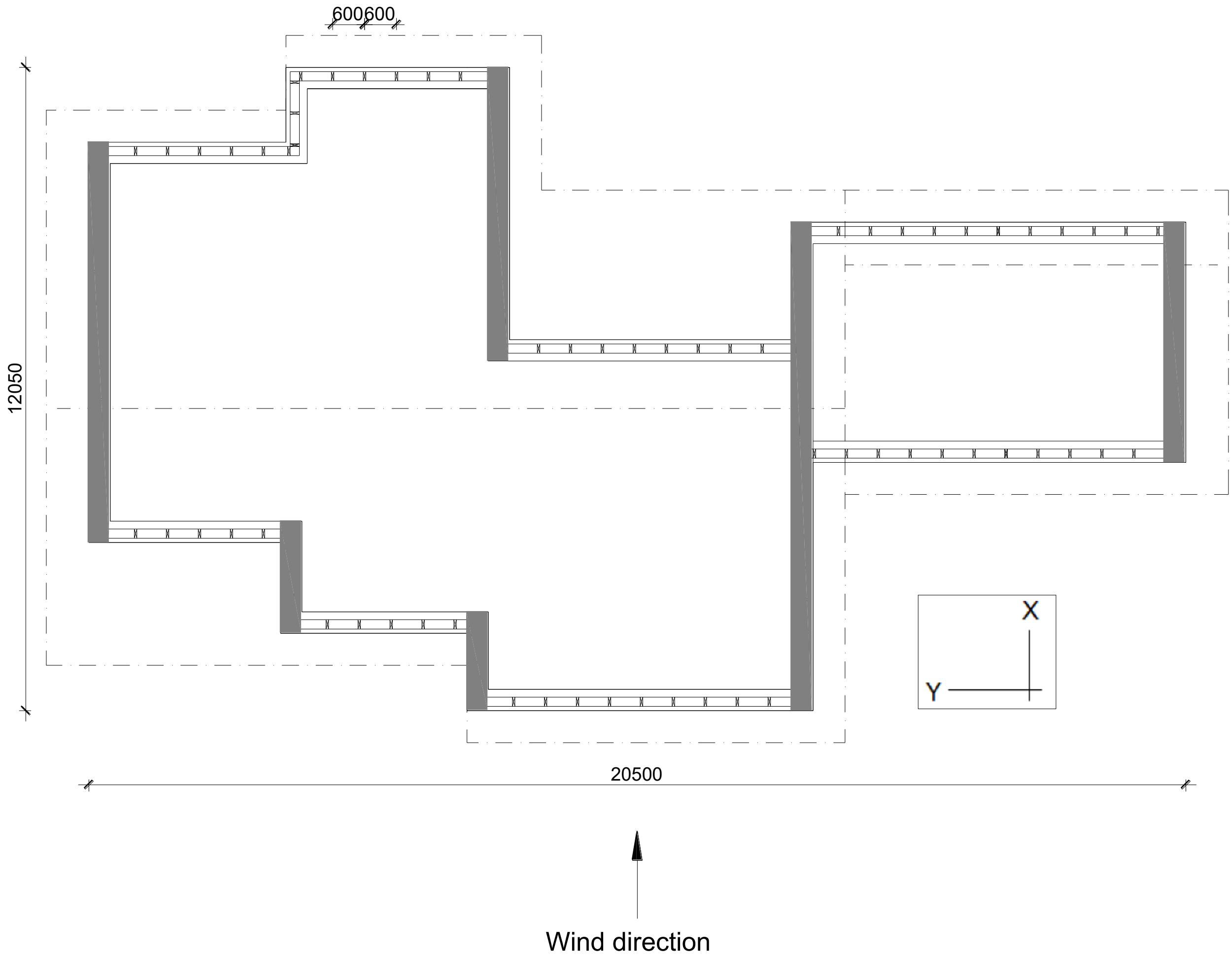
Compressive Utilization Ratio. $UR_{StudCompression} = 17\%$

Bearing Utilization Ratio. $UR_{BearingCapacity1} = 57\%$

Bearing Utilization Ratio. $UR_{BearingCapacity2} = 57\%$

Combined bending and axial compression. $UR_{combined} = 26\%$

Appendix 8: Stud on X-direction for the critical load combination.



Studs in this direction are not designed to take the load from the truss element and are placed parallel to X-axis. Therefore, studs on Y-direction would take the load coming from the roof. Walls in this direction will be under wind action more. For sure, they will take the load coming from the roof if a truss element is placed on top of the wall but compared to the walls in the other direction, the load would be smaller.

Therefore, checking for the maximum expected force is neglected as the studs in X-direction are not critical compared to those in Y-direction. It assumed that these walls do not take the vertical load, and checking can be neglected for compression, as the critical studs from the other direction were checked.

They can be considered as they only take wind load, and checking when wind action is valid can be done as follows.

Characteristic wind load in Y-direction. $Q_{k.3}=0.474\ \frac{kN}{m^2}$

Load coming in the center of the stud form dead load. $N_D:=0\ kN$

Load coming in the center of the stud from snow load. $N_S:=0\ kN$

Load coming in the center of the stud from the wind load. $N_W:=0\ kN$

Axial load, no bending moment occurs from the dead load. $M_D:=0\ kN\cdot m$

Axial load, no bending moment occurs from snow load. $M_S:=0\ kN\cdot m$

Bending moment from wind load. $M_W:=\frac{Q_{k.2}\cdot k_1\cdot L_2^{\ 2}}{8}=0.303\ kN\cdot m$

The coefficient for snow load. $\psi_0:=0.7$

Ultimate Limit State combination for normal force.

$$Combination1:=1.35\cdot K_{FI}\cdot N_D=0\ kN$$

$$Combination2:=1.15\cdot K_{FI}\cdot N_D+1.5\cdot K_{FI}\cdot N_S=0\ kN$$

$$Combination3:=1.15\cdot K_{FI}\cdot N_D+1.5\cdot K_{FI}\cdot N_S+1.5\cdot K_{FI}\cdot \psi_0\cdot N_W=0\ kN$$

$$Combination4:=1.15\cdot K_{FI}\cdot N_D+1.5\cdot K_{FI}\cdot N_W=0\ kN$$

$$Combination5:=1.15\cdot K_{FI}\cdot N_D+1.5\cdot K_{FI}\cdot N_W+1.5\cdot K_{FI}\cdot \psi_0\cdot N_S=0\ kN$$

Ultimate Limit State combination for bending moment.

$$Combination1:=1.35\cdot K_{FI}\cdot M_D=0\ kN\cdot m$$

$$Combination2:=1.15\cdot K_{FI}\cdot M_D+1.5\cdot K_{FI}\cdot M_S=0\ kN\cdot m$$

$$Combination3:=1.15\cdot K_{FI}\cdot M_D+1.5\cdot K_{FI}\cdot M_S+1.5\cdot K_{FI}\cdot \psi_0\cdot M_W=0.318\ m\cdot kN$$

$$Combination4:=1.15\cdot K_{FI}\cdot M_D+1.5\cdot K_{FI}\cdot M_W=0.454\ m\cdot kN$$

$$Combination5:=1.15\cdot K_{FI}\cdot M_D+1.5\cdot K_{FI}\cdot M_W+1.5\cdot K_{FI}\cdot \psi_0\cdot M_S=0.454\ m\cdot kN$$

			WIND LOAD			
Action	Permanent	Permanent+medium	Permanent+medium+short	Permanent+short	Permanent+medium+short	Unit
	Combination 1	Combination 2	Combination 3	Combination 4	Combination 5	
N_Ed	0	0	0	0	0	kN
M_Ed	0	0	0.318	0.454	0.454	kN*m
k_mod	0.6	0.8	0.9	0.9	0.9	

The above table shows that the first two combinations do not have loads or bending moments in the studs. From the table, it can be seen that the studs are taking only bending moments due to wind, which comes from the distribution of the load. Combinations 4 and 5 have the same bending moments, and the value of k_{mod} is the same; therefore, checking one of them is enough.

Combination 4 and 5.

Design normal force for combinations 4 and 5. $N_{Ed.C4} := 0\ kN$

Design bending moment for combinations 4 and 5. $M_{Ed.C4} := 0.454\ kN \cdot m$

Modification factor, EN1995-1-1, 3.2, table 3.1, permanent action. $k_{mod.C4} := 0.9$

Buckling coefficient about Y-axis. $k_{ey} := 1$

Buckling length about Y-axis. $L_{ey} := L_2 \cdot k_{ey} = 2.5\ m$

Net area of the studs. $A_s := b \cdot h = 8304\ mm^2$

Moment of inertia about Y-direction. $I_y := \dfrac{b \cdot h^3}{12} = \left(2.07 \cdot 10^7\right)\ mm^4$

The radius of gyration about the Y-direction. $r_y := \sqrt{\dfrac{I_y}{A_s}} = 49.94\ mm$

Slenderness ratio. $\lambda_y := \dfrac{L_{ey}}{r_y} = 50.059$

Slenderness ratio corresponding to bending about y-axis(deflection in the z-direction), EN1995-1-1, 6.3.2(6.21) $\lambda_{rel.y} := \dfrac{\lambda_y}{\pi} \cdot \sqrt{\dfrac{f_{c.0.k}}{E_{0.05}}} = 0.849$

Section modulus. $W_y := \dfrac{b \cdot h^2}{6} = \left(2.394 \cdot 10^5\right)\ mm^3$

Net area of the studs. $A_s := b \cdot h = 8304\ mm^2$

Compressive stress along the grain. $\sigma_{c.0.d} := \dfrac{N_{Ed.C4}}{A_s} = 0\ MPa$

Bending stress. $\sigma_{m.y.d} := \dfrac{M_{Ed.C4}}{W_y} = 1.896\ MPa$

EN1995-1-1, 6.1.6(2).

$$k_m := 0.7$$

The factor for members within the straightness limits, EN1995-1-1, 6.3.2(6.29).

$$\beta_c := 0.2$$

$$k_y := 0.5\ \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + {\lambda_{rel.y}}^2\right) = 0.915$$

Buckling deduction coefficients to consider the buckling effect.

$$k_{c.y} := \frac{1}{k_y + \sqrt{{k_y}^2 - {\lambda_{rel.y}}^2}} = 0.795$$

$$k_{mod.C4} = 0.9$$

Modification factor, EN1995-1-1, 3.2, table 3.1. Snow is a medium-term load.

System strength factor EN1995-1-1, 6.6, Figure 6.12

$$k_{sys} := 1$$

Partial factor, EN1995-1-1, 2.4.1, Table 2.3.

$$\gamma_M := 1.3$$

Reference depth in bending, EN1995-1-1, 3.2(3.1).

$$k_h := min\left(1.3, \left(\frac{150\ mm}{h}\right)^{0.2}\right) = 0.972$$

Design bending strength about the y-axis.

$$f_{m.y.d} := k_{mod.C4} \cdot \frac{f_{m.k}}{\gamma_M} \cdot k_h \cdot k_{sys} = 16.148\ MPa$$

Design compressive strength along the grain.

$$f_{c.0.d} := k_{mod.C4} \cdot k_{sys} \cdot \frac{f_{c.0.k}}{\gamma_M} = 14.538\ MPa$$

Compression check.

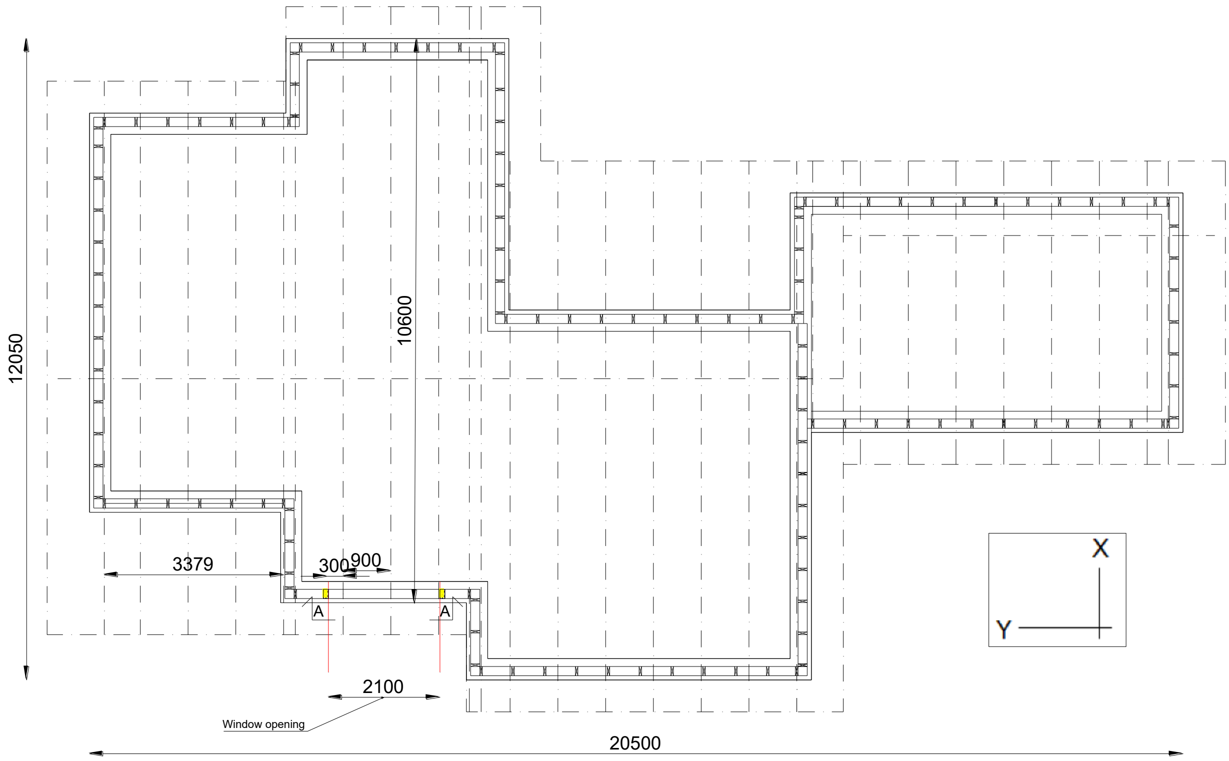
$$UR_{StudCompression} := \left\| \begin{array}{l} \text{if } \sigma_{c.0.d} \le k_{c.y} \cdot f_{c.0.d} \\ \quad \left\| \dfrac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} \right. \\ \text{else} \\ \quad \left\| \text{“check the calculations”} \right. \end{array} \right\| = 0$$

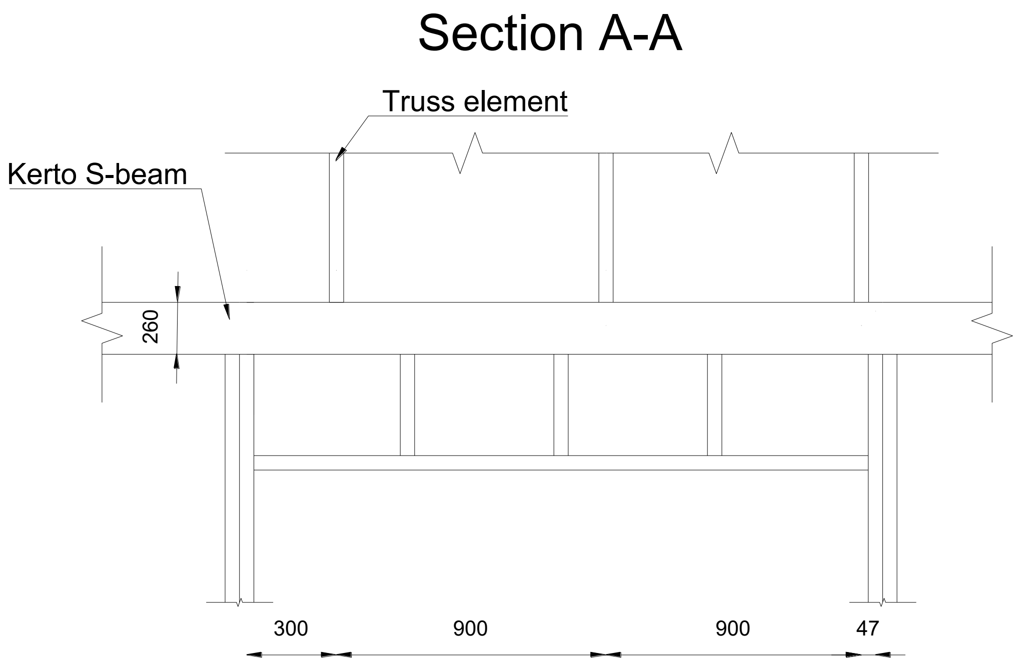
Combined compression and bending.

$$UR_{combined} := \left| \begin{array}{l} \text{if } \dfrac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + \dfrac{\sigma_{m.y.d}}{f_{m.y.d}} \\ \quad \left\| \text{“okay”} \right. \\ \text{else} \\ \quad \left\| \text{“check calculations”} \right. \end{array} \right. = \text{“okay”}$$

$$UR_{combined} := \frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + \frac{\sigma_{m.y.d}}{f_{m.y.d}} = 12\%$$

Appendix 9: Window opening check





Loads

Characteristic roof. $G_{k.1} := 0.683 \ \frac{kN}{m^2}$

Characteristic dead load at the upper eaves. $G_{k.2} := 0.2 \ \frac{kN}{m^2}$

Characteristic total dead load. $G_{k.3} := G_{k.1} + G_{k.2} = 0.88 \ \frac{kN}{m^2}$

Characteristic snow load. $Q_{k.1} := 2.4 \ \frac{kN}{m^2}$

Dimensions

Lattice span. $L := 12.05 \ m$

Width of opening. $L_1 := 2.1 \ m$

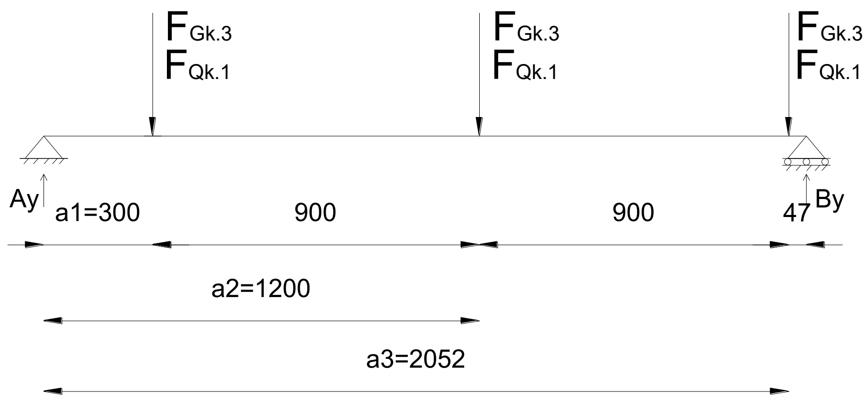
Truss element spacing. $s := 900 \ mm$

Point load at truss contact

Point load from the dead load. $F_{Gk.3} := G_{k.3} \cdot \frac{L}{2} \cdot s = 4.79 \ kN$

Point load from snow load. $F_{Qk.1} := Q_{k.1} \cdot \frac{L}{2} \cdot s = 13.01 \ kN$

The Finite Element Method will be used to solve the analytical model for practical and study reasons. This is because in case the dimensions of the window will change, only some variables are changed to get the result. Other methods can be used to solve the analytical model of the beam, such as different software, such as Dlubal RFEM, but if the window's dimensions will change, then through the window, the opening is designed once again from the beginning for found forces for the analytical model. For this reason, it is simpler to use the Finite Element Method, where the only things that need to be changed are the distances of forces from one of the supports.



Kerto S-beam made of laminated veneer lumber will be used for the beam over the window. The beam is selected from the Metsä Wood. Mechanical properties and other information are taken from the manuals provided by this company.

Dimensions

Kerto S-beam width. $b := 75\ mm$

Kerto S-beam height. $h := 260\ mm$

Timber properties

Beam material. Kerto LVL S-beam.

Bending capacity. $f_{m.k} := 44\ \frac{N}{mm^2}$

Shear capacity. $f_{v.k} := 4.2\ \frac{N}{mm^2}$

Compression perpendicular. $f_{c.90.k} := 6 \ \frac{N}{mm^2}$

Mean modulus of elasticity parallel. $E_{mean} := 13800 \ \frac{N}{mm^2}$

5% modulus of elasticity parallel. $E_{0.05} := 11600 \ \frac{N}{mm^2}$

Service class. Class 1

Finite Element Method for the dead load.

$a1 := 300 \ mm$
$a2 := 1200 \ mm$
$a3 := 2052 \ mm$

$$f(x,a,n) := \begin{cases} (x-a)^n & \text{if } x-a \geq 0 \\ 0 \ m^n & \text{else} \end{cases} \qquad df(x,a,n) := \begin{cases} n \cdot (x-a)^{n-1} & \text{if } x-a \geq 0 \\ 0 \ m^{n-1} & \text{else} \end{cases}$$

$$d2f(x,a,n) := \begin{cases} n \cdot (n-1) \cdot (x-a)^{n-2} & \text{if } x-a \geq 0 \\ 0 \ m^{n-2} & \text{else} \end{cases} \qquad d3f(x,a,n) := \begin{cases} n \cdot (n-1) \cdot (n-2) \cdot (x-a)^{n-3} & \text{if } x-a \geq 0 \\ 0 \ m^{n-3} & \text{else} \end{cases}$$

Elastic modulus. $E := E_{mean}$

Moment of inertia about the y-y axis. $I := \frac{b \cdot h^3}{12} = \left(1.099 \cdot 10^8\right) \ mm^4$

Equations

$$v\left(x,A_y,\alpha_0\right):=\frac{A_y\cdot f\left(x,0\ m,3\right)}{3!\cdot E\cdot I}+-\alpha_0\cdot f\left(x,0\ m,1\right)+\frac{-F_{Gk.3}\cdot f\left(x,a_1,3\right)}{3!\cdot E\cdot I}-\frac{F_{Gk.3}\cdot f\left(x,a_2,3\right)}{3!\cdot E\cdot I}-\frac{F_{Gk.3}\cdot f\left(x,a_3,3\right)}{3!\cdot E\cdot I}$$

$$dv\left(x,A_y,\alpha_0\right):=\frac{A_y\cdot df\left(x,0\ m,3\right)}{3!\cdot E\cdot I}+-\alpha_0\cdot df\left(x,0\ m,1\right)+\frac{-F_{Gk.3}\cdot df\left(x,a_1,3\right)}{3!\cdot E\cdot I}-\frac{F_{Gk.3}\cdot df\left(x,a_2,3\right)}{3!\cdot E\cdot I}-\frac{F_{Gk.3}\cdot df\left(x,a_3,3\right)}{3!\cdot E\cdot I}$$

$$d2v\left(x,A_y,\alpha_0\right):=\frac{A_y\cdot d2f\left(x,0\ m,3\right)}{3!\cdot E\cdot I}+-\alpha_0\cdot d2f\left(x,0\ m,1\right)+\frac{-F_{Gk.3}\cdot d2f\left(x,a_1,3\right)}{3!\cdot E\cdot I}-\frac{F_{Gk.3}\cdot d2f\left(x,a_2,3\right)}{3!\cdot E\cdot I}-\frac{F_{Gk.3}\cdot d2f\left(x,a_3,3\right)}{3!\cdot E\cdot I}$$

$$d3v\left(x,A_y,\alpha_0\right):=\frac{A_y\cdot d3f\left(x,0\ m,3\right)}{3!\cdot E\cdot I}+-\alpha_0\cdot d3f\left(x,0\ m,1\right)+\frac{-F_{Gk.3}\cdot d3f\left(x,a_1,3\right)}{3!\cdot E\cdot I}-\frac{F_{Gk.3}\cdot d3f\left(x,a_2,3\right)}{3!\cdot E\cdot I}-\frac{F_{Gk.3}\cdot d3f\left(x,a_3,3\right)}{3!\cdot E\cdot I}$$

Applying boundary conditions

Guess Values

$A_y:=1\ kN \qquad \alpha_0:=1\ deg$

Constraints

$v\left(L_1,A_y,\alpha_0\right)=0 \qquad E\cdot I\cdot d2v\left(L_1,A_y,\alpha_0\right)=0$

Solver

$$\begin{bmatrix}A_y\\ \alpha_0\end{bmatrix}:=\mathbf{find}\left(A_y,\alpha_0\right)=\begin{bmatrix}\left(6.266\cdot 10^{3}\right)\ N\\ 0.001\end{bmatrix}$$

$A_y=6.266\ kN$

$x := 0, 1\ cm .. L_1$

Where:

L_1 – the length of the beam.

The equation to draw the graph accordingly.

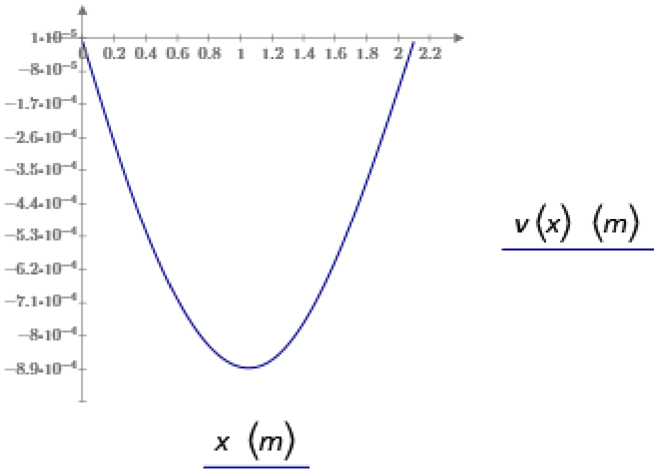
Deformation. $v(x) := v(x, A_y, \alpha_0)$

Slope. $dv(x) := dv(x, A_y, \alpha_0)$

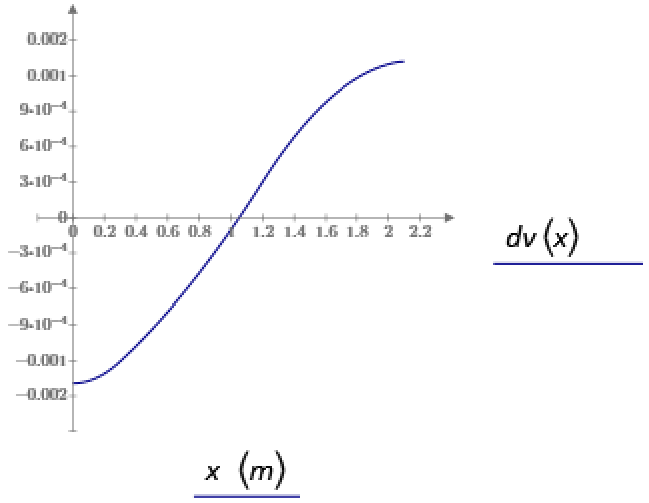
Bending moment. $M(x) := E \cdot I \cdot d2v(x, A_y, \alpha_0)$

Shear force. $V(x) := E \cdot I \cdot d3v(x, A_y, \alpha_0)$

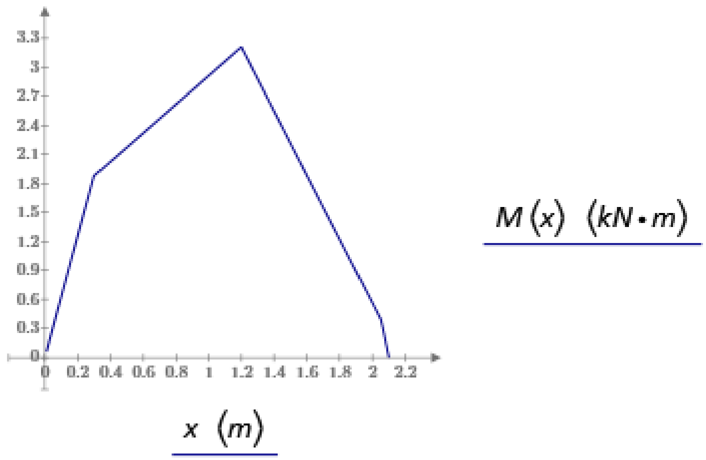
Deformation.



Slope.



Bending moment.



The position where the maximum bending moment occurs.

Guess Values

$x := 2\ m$

Constraints

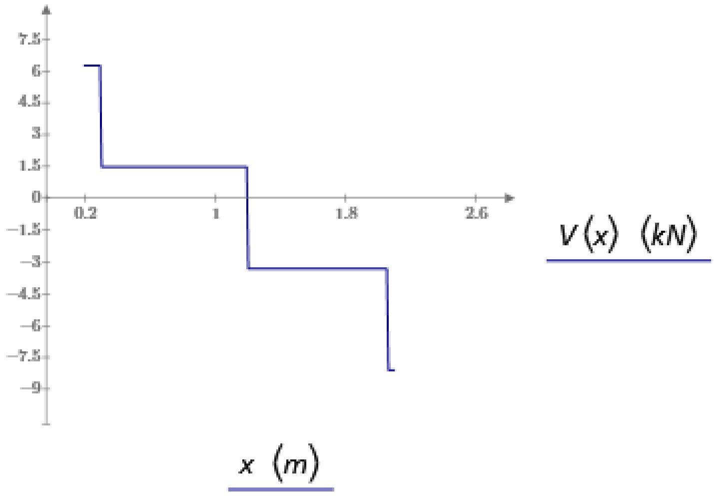
Solver

$X_{max} := \mathbf{maximize}\,(M, x) = 1.2\ m$

Value of maximum bending moment for the dead load.

$M_{max} := M\left(X_{max}\right) = 3.209\ kN \cdot m$

Shear force.



Support reaction at the end for the dead load. $B_y := -V(L_1) = 8.099\ kN$

Summary of actions for the dead load.

Left support reaction. $A_y = 6.27\ kN$

Right support reaction. $B_y = 8.1\ kN$

Maximum bending moment. $M_{max} = 3.21\ kN \cdot m$

The same procedure applies to the snow load. Therefore, the result obtained from applying the Finite Element Method for the snow load would follow.

Summary of action for snow load.

Left support reaction. $A_y = 17.03\ kN$

Right support reaction. $B_y = 22.01\ kN$

Maximum bending moment. $M_{max} = 8.72\ kN \cdot m$

Ultimate Limit State combination for shear force

$Combination1 := 1.35 \cdot V_{Gk.3} = 10.94\ kN$
$Combination2 := 1.15 \cdot V_{Gk.3} + 1.5 \cdot V_{Qk.1} = 42.33\ kN$

Ultimate Limit State combination for bending moment

$Combination1.1 := 1.35 \cdot M_{Gk.3} = 4.33\ kN \cdot m$
$Combination1.2 := 1.15 \cdot M_{Gk.3} + 1.5 \cdot M_{Qk.1} = 16.77\ kN \cdot m$

The same procedure applies here for defining the modification factor when verifying the studs. In each load combination, the shortest load term duration should be taken to define the modification factor accordingly, and the check must follow. As the critical combination here will be the one with the higher values, the check is applied for this. The check for other combinations was applied too, but only the critical one is explained in this section.

Bending check for combination1.2

Design bending moment. $M_{Ed} := Combination1.2 = 16.77\ kN \cdot m$

Snow load, medium-term action. $k_{mod} := 0.8$

Beam material. Kerto LVL S-beam.

Bending capacity. $f_{m.k} := 44\ \frac{N}{mm^2}$

Shear capacity. $f_{v.k} := 4.2\ \frac{N}{mm^2}$

Compression perpendicular. $f_{c.90.k} := 6\ \frac{N}{mm^2}$

Mean modulus of elasticity parallel. $E_{mean} := 13800\ \frac{N}{mm^2}$

5% modulus of elasticity parallel. $E_{0.05} := 11600\ \frac{N}{mm^2}$

Service class. Class 1

Kerto S-beam width. $b := 75\ mm$

Kerto S-beam height. $h := 260\ mm$

Bending check

Section modulus. $W_y := \frac{b \cdot h^2}{6} = \left(8.45 \cdot 10^5\right)\ mm^3$

Bending stress. $\sigma_{m.y.d} := \frac{M_{Ed}}{W_y} = 19.85\ MPa$

Bending strength, size effect parameter, Metsä Wood manual. $s := 0.12\ \frac{N}{mm^2}$

Taken as unitless. $s := 0.12$

Reference depth in bending, EN1995-1-1, 3.4(3.3), for LVL members. $k_h := min\left(\left(\frac{300}{\frac{h}{mm}}\right)^s, 1.2\right) = 1.017$

System strength factor EN1995-1-1, 6.6, Figure 6.12 $k_{sys} := 1$

Design bending strength about the y-axis. $f_{m.y.d} := k_{mod} \cdot \frac{f_{m.k}}{\gamma_M} \cdot k_h \cdot k_{sys} = 27.55\ MPa$

Condition

$$\begin{array}{l} \text{if } \sigma_{m.y.d} \le f_{m.y.d} \\ \quad \Vert \text{“okay”} \\ \text{else} \\ \quad \Vert \text{“fail”} \end{array} = \text{“okay”}$$

$$UR := \frac{\sigma_{m.y.d}}{k_{crit} \cdot f_{m.y.d}} = 72.05\%$$

Shear check

Design shear force. $V_{Ed} := Combination2 = 42.33\ kN$

EN1995-1-1, 6.1.7 (2). $k_{cr} := 1$

Effective width, EN1995-1-1, 6.1.7 (2), 6.13a. $b_{ef} := b \cdot k_{cr} = 75\ mm$

Design shear stress, EN1995-1-1, 6.1.7. $\tau_d := \frac{3}{2} \cdot \frac{V_{Ed}}{2 \cdot k_{cr} \cdot b \cdot h} = 1.63\ MPa$

Design shear strength, EN1995-1-1, 6.1.7. $f_{v.d} := k_{mod} \cdot \frac{f_{v.k}}{\gamma_M} \cdot k_{sys} = 2.58\ MPa$

Condition

$$\left.\begin{array}{l}\textbf{if } \tau_d \le f_{v.d} \\ \quad \Vert \text{“okay”} \\ \textbf{else} \\ \quad \Vert \text{“fail”}\end{array}\right| = \text{“okay”}$$

$$UR := \frac{\tau_d}{f_{v.d}} = 62.99\%$$

Compression check

Characteristic support reaction from the dead load. $R_{Gk.3} := 8.1\ kN$

Characteristic support reaction from the snow load. $R_{Qk1} := 22.01\ kN$

Truss width. $l := 48\ mm$

Studs are placed center to center, no side gap left. $a := 0\ mm$

Stud spacing, windows opening. $l_1 := 2100\ mm$

Distance from the left side. $d_1 := min\left(30\ mm, a, l\right) = 0\ mm$

Distance from the right side. $d_2 := min\left(30\ mm, l, \frac{l_1}{2}\right) = 30\ mm$

Effective length. $b_{eff} := b + d_1 + d_2 = 105\ mm$

The effective area of the stud. $A_{eff} := b_{eff} \cdot h = \left(2.73 \cdot 10^4\right)\ mm^2$

Design normal force. $N_{Ed} := 1.15 \cdot R_{Gk.3} + 1.5 \cdot R_{Qk1} = 42.33\ kN$

Design compressive stress along the grain.

$$\sigma_{c.90.d} := \frac{N_{Ed}}{A_{eff}} = 1.551\ MPa$$

Design compressive strength along the grain.

$$f_{c.90.d} := k_{mod} \cdot k_{sys} \cdot \frac{f_{c.90.k}}{\gamma_M} = 3.692\ MPa$$

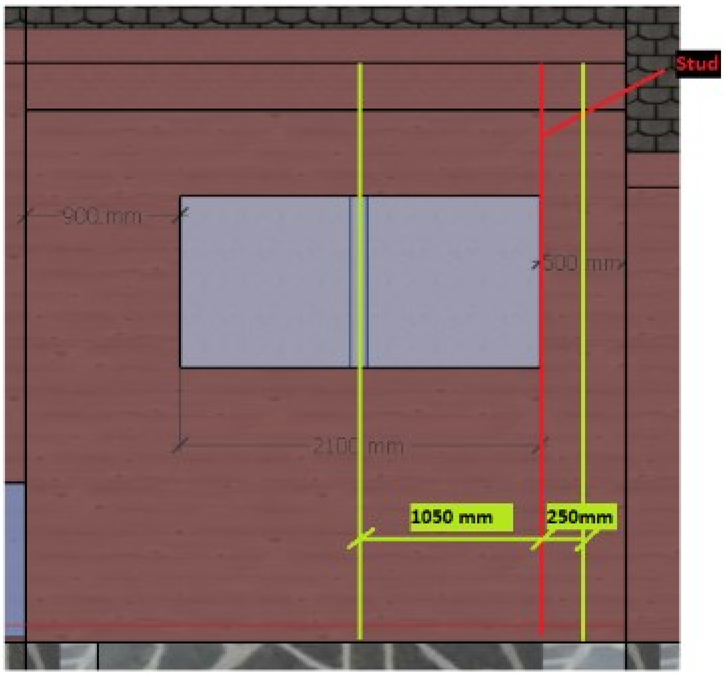
Factor taking into account the load configuration, RIL 205, 1.9.2009.

$$k_{c.90} := 1.4$$

Bearing capacity.

$$UR_{BearingCapacity1} := \frac{\sigma_{c.90.d}}{k_{c.90} \cdot f_{c.90.d}} = 30\%$$

Combined action, normal force plus bending moment from the wind.



Characteristic wind load in X-direction.

$$Q_{k.2} := 0.646\ \frac{kN}{m^2}$$

Tributary length.

$$L_2 := 1050\ mm$$

Tributary length.

$$L_3 := 250\ mm$$

Final tributary length.

$$L_4 := L_2 + L_3 = 1300\ mm$$

Height of the stud

$$L_5 := 3100\ mm$$

Bending moment from wind load.

$$M_W := \frac{Q_{k.2} \cdot L_4 \cdot L_5^{\,2}}{8} = 1.009\ kN \cdot m$$

To find the N_{Ed} the support reaction for the beam is taken, and the right side, the tributary area, is added to find the regular forces. This would result in the total normal force acting on the stud.

The normal force from the dead load. $N_D := G_{k.3} \cdot \frac{L}{2} \cdot L_4 = 6.916\ kN$

The normal force from snow load. $N_S := Q_{k.1} \cdot \frac{L}{2} \cdot L_4 = 18.798\ kN$

Ultimate Limit State combination for normal force

$$N_{Ed} := 1.15 \cdot N_D + 1.5 \cdot N_S = 36.151\ kN$$

Ultimate Limit State combination for bending moment

$$M_{Ed} := 1.5 \cdot M_W = 1.513\ m \cdot kN$$

Critical combination

Design normal force for combination 1. $N_{Ed.C1} := N_{Ed} = 36.151\ kN$

Design bending moment for combination 1. $M_{Ed.C1} := M_{Ed} = 1.513\ m \cdot kN$

Modification factor, EN1995-1-1, 3.2, table 3.1, short-term action. $k_{mod.C1} := 0.9$

Bending strength. $f_{m.k} := 24\ MPa$

Compression parallel to the grain. $f_{c.0.k} := 21\ MPa$

Compression perpendicular to the grain. $f_{c.90.k} := 2.5\ MPa$

5% modulus of elasticity parallel. $E_{0.05} := 7.4\ GPa$

Buckling coefficient about Y-axis. $k_{ey} := 1$

Buckling length about Y-axis. $L_{ey} := L_5 \cdot k_{ey} = 3.1\ m$

Width of the stud. $b := 48\ mm$

Height of the stud. $h := 173\ mm$

Net area of the studs. $A_s := b \cdot h = 8304\ mm^2$

Moment of inertia about Y-direction. $I_y := \dfrac{b \cdot h^3}{12} = \left(2.07 \cdot 10^7\right)\ mm^4$

The radius of gyration about the Y-direction. $r_y := \sqrt{\dfrac{I_y}{A_s}} = 49.94\ mm$

Slenderness ratio. $\lambda_y := \dfrac{L_{ey}}{r_y} = 62.073$

Slenderness ratio corresponding to bending about y-axis(deflection in the z-direction), EN1995-1-1, 6.3.2(6.21) $\lambda_{rel.y} := \dfrac{\lambda_y}{\pi} \cdot \sqrt{\dfrac{f_{c.0.k}}{E_{0.05}}} = 1.053$

Section modulus. $W_y := \dfrac{b \cdot h^2}{6} = \left(2.394 \cdot 10^5\right)\ mm^3$

Net area of the studs. $A_s := b \cdot h = 8304\ mm^2$

Compressive stress along the grain. $\sigma_{c.0.d} := \dfrac{N_{Ed.C1}}{A_s} = 4.353\ MPa$

Bending stress. $\sigma_{m.y.d} := \dfrac{M_{Ed.C1}}{W_y} = 6.32\ MPa$

EN1995-1-1, 6.1.6(2). $k_m := 0.7$

The factor for members within the straightness limits, EN1995-1-1, 6.3.2(6.29). $\beta_c := 0.2$

$$k_y := 0.5\ \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + {\lambda_{rel.y}}^2\right) = 1.129$$

Buckling deduction coefficients to consider the buckling effect. $k_{c.y} := \dfrac{1}{k_y + \sqrt{{k_y}^2 - {\lambda_{rel.y}}^2}} = 0.65$

Modification factor, EN1995-1-1, 3.2, table 3.1. Snow is a medium-term load. $k_{mod.C1} = 0.9$

System strength factor EN1995-1-1, 6.6, Figure 6.12 $k_{sys} := 1$

Partial factor, EN1995-1-1, 2.4.1, Table 2.3. $\gamma_M := 1.3$

Reference depth in bending, EN1995-1-1, 3.2(3.1). $k_h := min\left(1.3, \left(\dfrac{150\ mm}{h}\right)^{0.2}\right) = 0.972$

Design bending strength about the y-axis. $f_{m.y.d} := k_{mod.C1} \cdot \dfrac{f_{m.k}}{\gamma_M} \cdot k_h \cdot k_{sys} = 16.148\ MPa$

Design compressive strength along the grain.

$$f_{c.0.d} := k_{mod.C1} \cdot k_{sys} \cdot \frac{f_{c.0.k}}{\gamma_M} = 14.538\ MPa$$

Condition

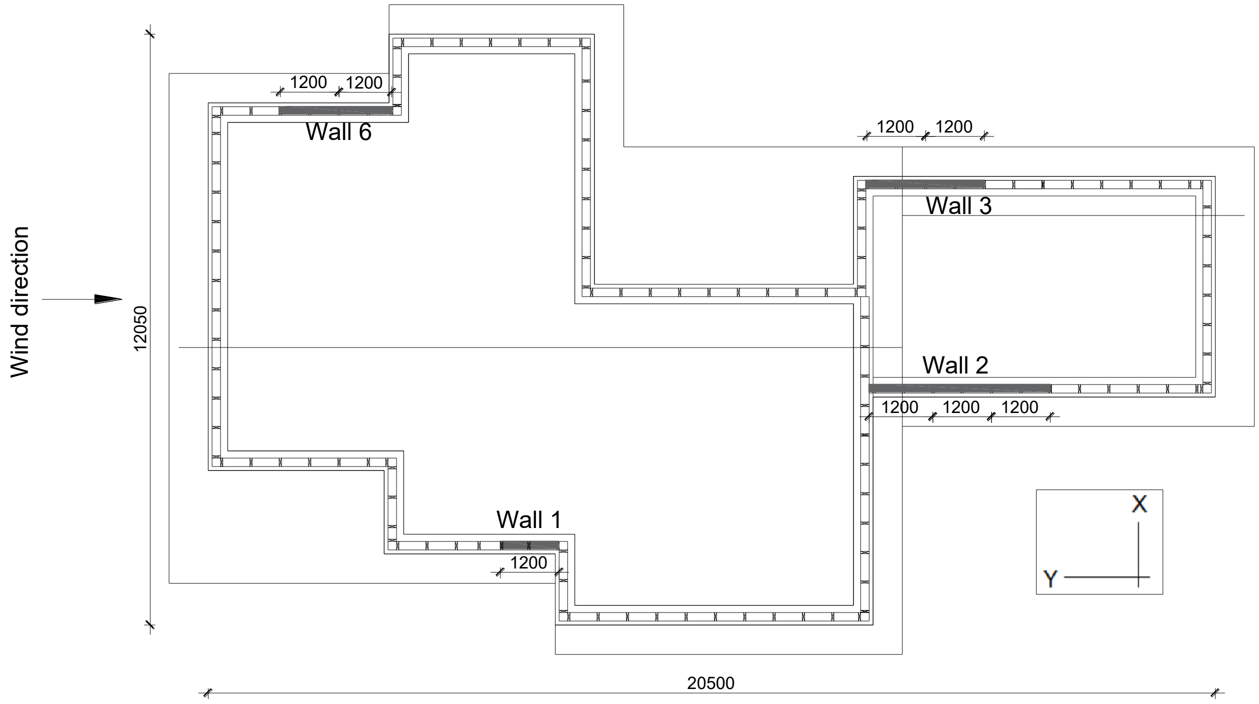
$$UR_{StudCompression} := \left\| \begin{array}{l} \text{if } \sigma_{c.0.d} \le k_{c.y} \cdot f_{c.0.d} \\ \quad \left\| \dfrac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} \right. \\ \text{else} \\ \quad \| 0 \end{array} \right\| = 46\%$$

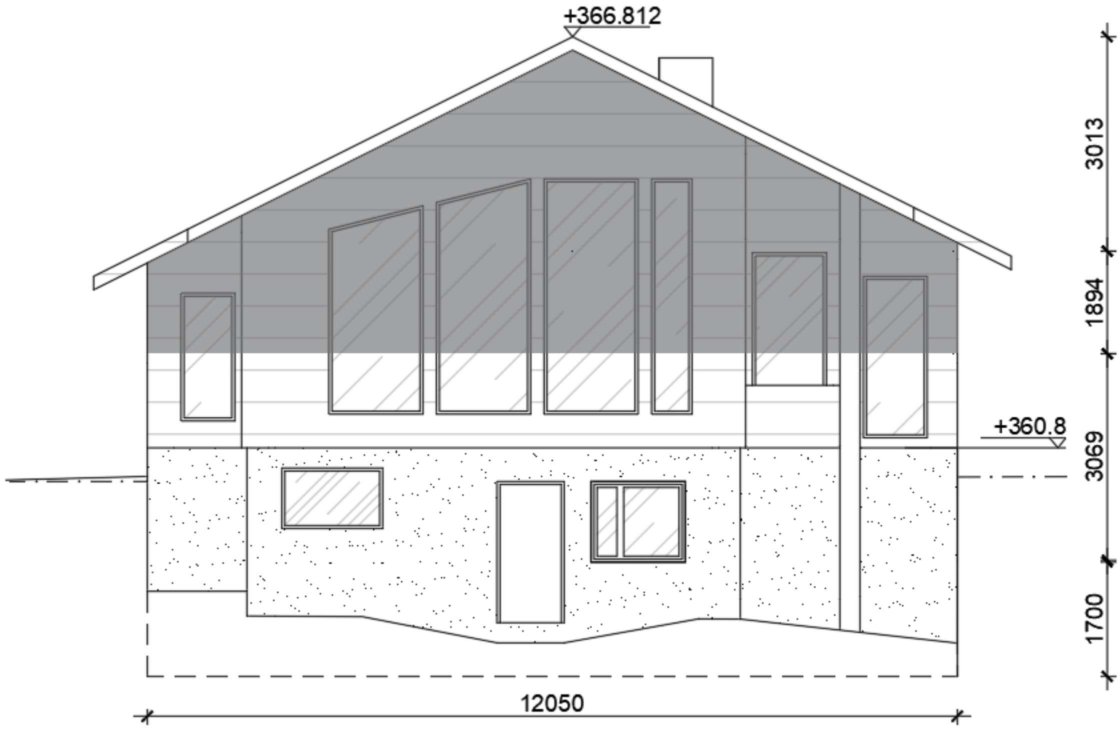
Combined compression and bending.

$$UR_{combined} := \left. \begin{array}{l} \text{if } \dfrac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + \dfrac{\sigma_{m.y.d}}{f_{m.y.d}} \\ \quad \| \text{“okay”} \\ \text{else} \\ \quad \| \text{“check calculations”} \end{array} \right| = \text{“okay”}$$

$$UR_{combined} := \frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + \frac{\sigma_{m.y.d}}{f_{m.y.d}} = 85\%$$

Appendix 10: Stiffening walls in Y-direction





Height of shorter wall. $h_1 := 3200\ mm$

Height of taller wall. $h_2 := 4375\ mm$

Average height. $h_3 := \frac{h_1 + h_2}{2} \cdot \frac{1}{2} = 1.894\ m$

Height from the ridge of the roof to the base of the wall. See the above picture. $h_4 := 3013\ mm$

Rectangle area; see above picture. $A_1 := R_1 \cdot h_3 = 22.82\ m^2$

Triangle area. $A_2 := \frac{R_1 \cdot h_4}{2} = 18.153\ m^2$

Total wind area face. $A := A_1 + A_2 = 40.973\ m^2$

Characteristic wind load in Y-direction. $Q_{k.3} = 0.474\ \frac{kN}{m^2}$

Design distributed load. $q_{Ed} := 1.5\ K_{FI} \cdot Q_{k.3} = 0.711\ \frac{kN}{m^2}$

Design force. $F_{Ed} := q_{Ed} \cdot A = 29.132\ kN$

The outer walls in this direction are considered in service class 2, as they may be exposed to outside weather conditions.

Resistance of stiffening walls

Wall 1

Type of fastener QU 32.

The characteristic lateral design capacity of an individual fastener EN1991-1-4(9.21). $F_{f.Rk1} := 450\ N$

Partial safety factor. $\gamma_n := 1.3$

Design lateral design capacity of an individual fastener EN1991-1-4(9.21). $F_{f.Rd1} := \frac{F_{f.Rk1}}{\gamma_n} = 346.154\ N$

Fastener spacing EN1991-1-4(9.21). $s_1 = 70\ mm$

Wall panel width EN1991-1-4(9.22). $b_i := 1200\ mm$

Height of the considered wall. $h := 3.1\ m$

The ratio of height, EN1991-1-4, 9.2.4.2, (4). $b_0 := \frac{h}{2} = 1.55\ m$

EN1991-1-4, 9.2.4.2, (4), (9.22).

$$c_i := \begin{cases} 1 & \text{if } b_i \geq b_0 \\ \dfrac{b_i}{b_0} & \text{else} \end{cases} = 0.8$$

The design racking load-carrying capacity of each wall panel EN1991-1-4, 9.2.4.2, (4), (9.21).

$$F_{1.Rd} := \frac{F_{f.Rd1} \cdot b_i \cdot c_i}{s_1} = 4.594\ kN$$

Design racking load-carrying capacity for the top chord.

$$F_{Top.Rd1} := F_{1.Rd} = 4.594\ kN$$

Wall 2

Type of fastener QU 32.

The characteristic lateral design capacity of an individual fastener EN1991-1-4(9.21).

$$F_{f.Rk2} := 450\ N$$

Partial safety factor.

$$\gamma_n := 1.3$$

Design lateral design capacity of an individual fastener EN1991-1-4(9.21).

$$F_{f.Rd2} := \frac{F_{f.Rk2}}{\gamma_n} = 346.154\ N$$

Fastener spacing EN1991-1-4(9.21).

$$s_1 = 70\ mm$$

Wall panel width EN1991-1-4(9.22).

$$b_i := 1200\ mm$$

Height of the considered wall.

$$h := 2.5\ m$$

The ratio of height, EN1991-1-4, 9.2.4.2, (4).

$$b_0 := \frac{h}{2} = 1.25\ m$$

EN1991-1-4, 9.2.4.2, (4), (9.22).

$$c_i := \begin{cases} 1 & \text{if } b_i \geq b_0 \\ \dfrac{b_i}{b_0} & \text{else} \end{cases} = 1$$

The design racking load-carrying capacity of each wall panel EN1991-1-4(9.21).

$$F_{2.Rd} := \frac{F_{f.Rd2} \cdot b_i \cdot c_i}{s_1} = 5.697\ kN$$

Three wall panels with a width of 1200 mm.

$$F_{2.Rd} := 3 \cdot F_{2.Rd} = 17.09\ kN$$

Wall 3

Type of fastener	QU 32.
The characteristic lateral design capacity of an individual fastener EN1991-1-4(9.21).	$F_{f.Rk3} := 450\ N$
Partial safety factor.	$\gamma_n := 1.3$
Design lateral design capacity of an individual fastener EN1991-1-4(9.21).	$F_{f.Rd3} := \frac{F_{f.Rk3}}{\gamma_n} = 346.154\ N$
Fastener spacing EN1991-1-4(9.21).	$s_1 = 70\ mm$
Wall panel width EN1991-1-4(9.22).	$b_i := 1200\ mm$
Height of the considered wall.	$h := 2.5\ m$
The ratio of height, EN1991-1-4, 9.2.4.2, (4).	$b_0 := \frac{h}{2} = 1.25\ m$
EN1991-1-4, 9.2.4.2, (4), (9.22).	$c_i := \begin{cases} 1 & \text{if } b_i \geq b_0 \\ \frac{b_i}{b_0} & \text{else} \end{cases} = 1$
The design racking load-carrying capacity of each wall panel EN1991-1-4(9.21).	$F_{3.Rd} := \frac{F_{f.Rd3} \cdot b_i \cdot c_i}{s_1} = 5.697\ kN$
Two wall panels with a width of 1200 mm.	$F_{3.Rd} := 2 \cdot F_{3.Rd} = 11.393\ kN$

Wall 4

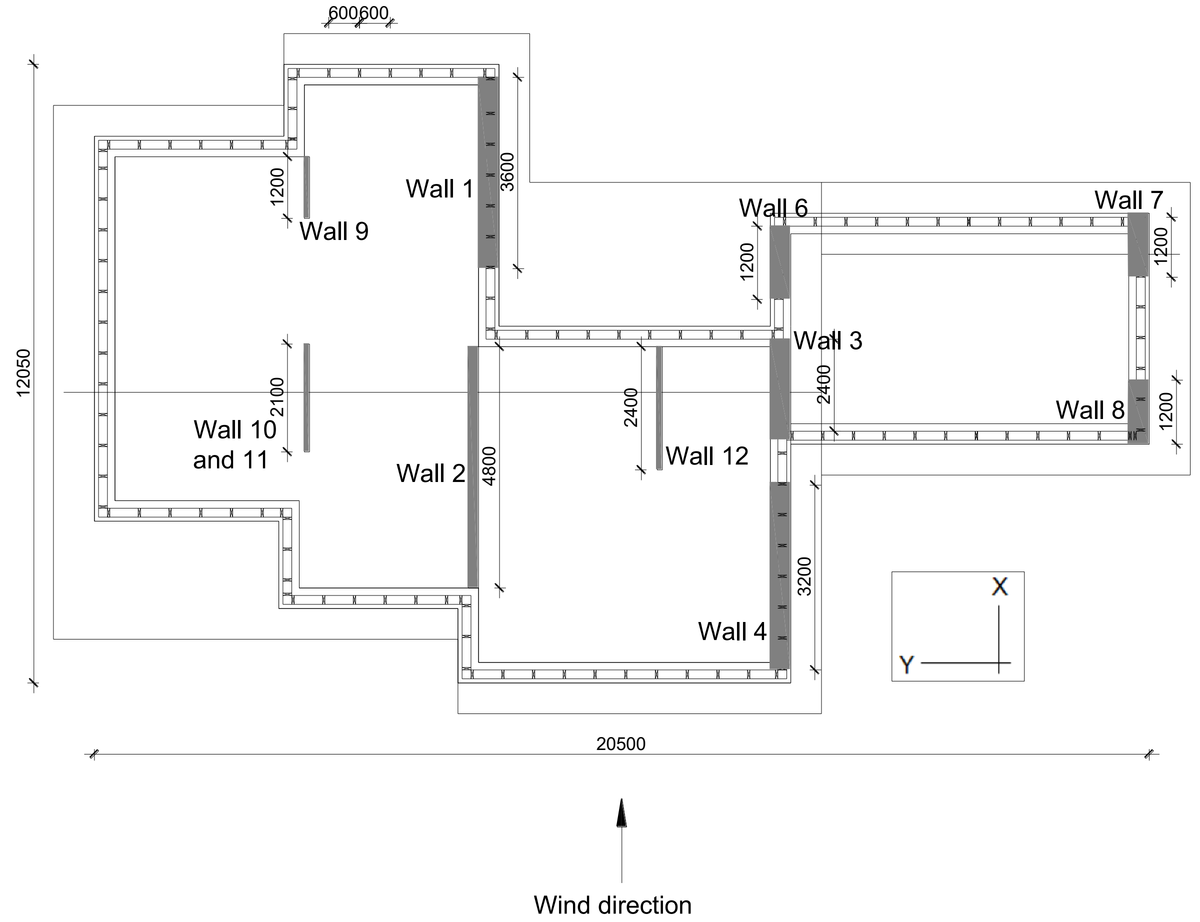
Type of fastener	QU 32.
The characteristic lateral design capacity of an individual fastener EN1991-1-4(9.21).	$F_{f.Rk4} := 450\ N$
Partial safety factor.	$\gamma_n := 1.3$
Design lateral design capacity of an individual fastener EN1991-1-4(9.21).	$F_{f.Rd4} := \frac{F_{f.Rk4}}{\gamma_n} = 346.154\ N$
Fastener spacing EN1991-1-4(9.21).	$s_1 = 70\ mm$
Wall panel width EN1991-1-4(9.22).	$b_i := 1200\ mm$
Height of the considered wall.	$h := 3.2\ m$
The ratio of height, EN1991-1-4, 9.2.4.2, (4).	$b_0 := \frac{h}{2} = 1.6\ m$
EN1991-1-4, 9.2.4.2, (4), (9.22).	$c_i := \begin{cases} 1 & \text{if } b_i \geq b_0 \\ \frac{b_i}{b_0} & \text{else} \end{cases} = 0.8$
The design racking load-carrying capacity of each wall panel EN1991-1-4(9.21).	$F_{4.Rd} := \frac{F_{f.Rd4} \cdot b_i \cdot c_i}{s_1} = 4.451\ kN$
Two wall panels with a width of 1200 mm.	$F_{4.Rd} := 2 \cdot F_{4.Rd} = 8.901\ kN$
Summary of wall resistances	$F_{Rd} := F_{1.Rd} + F_{2.Rd} + F_{3.Rd} + F_{4.Rd} = 41.979\ kN$

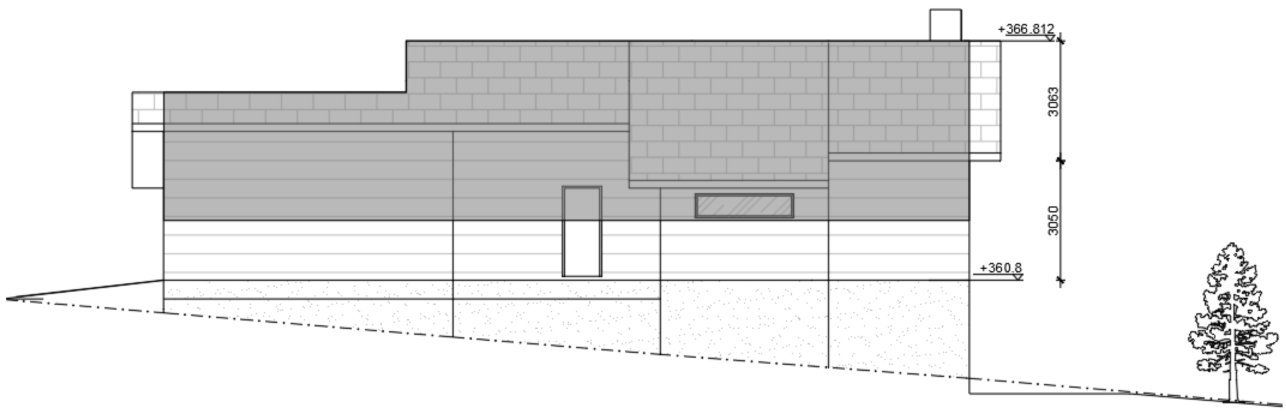
Condition to be checked.

$$\begin{cases} \text{“Walls do not have enough resistance”} & \text{if } F_{Rd} < F_{Ed} \\ \text{“Walls have enough resistance”} & \text{else} \end{cases} = \text{“Walls have enough resistance”}$$

$$UR := \frac{F_{Ed}}{F_{Rd}} = 69.397\%$$

Appendix 11: Stiffening walls in X-direction





The procedure will be the same as in the case of the Y-direction. The only difference to be taken into account would be that the service class would affect the type of fastener for the inner stiffening walls. Below it shows the calculation of one of the inner stiffening walls, while the same procedure applies to others. The utilization ratio can be seen that it is 95%, which would not be a concern in this case as, besides the gypsum boards, the walls will also have wooden boards, which would increase the stiffness of the wall. Since there is no direct method of calculating the stiffness of the gypsum boards or wooden boards, it is safe to assume that the walls will have more stiffness, decreasing the utilization ratio.

To find the wind force coming to this face of the wall, we need to multiply the load in X-direction by the area marked on the above drawing with red lines.

First-floor wall height. $h_5 := 3050\ mm$

Height of roof apex. $h_6 := 3062\ mm$

Considered height for calculation. $h_7 := \frac{h_5}{2} + h_6 = 4587\ mm$

Length of the building. $R_2 := 20500\ mm$

Design wind pressure load in ULS. $A_3 := 88.8\ m^2$

Design load coming from wind pressure. $q_{Ed} := 1.5 \cdot Q_{k.2} = 0.969\ \frac{kN}{m^2}$

Design force coming from wind pressure. $F_{Ed} := q_{Ed} \cdot A_3 = 86.047\ kN$

Wall 2

Wall 2 will be exposed to inner conditions; therefore, the service class would be 1, and the proper fastener should be taken.

Type of fastener QM-ST 32.

The characteristic lateral design capacity of an individual fastener EN1991-1-4(9.21). $F_{f.Rk2} := 400\ N$

Partial safety factor. $\gamma_n := 1.3$

Design lateral design capacity of an individual fastener EN1991-1-4(9.21). $F_{f.Rd2} := \frac{F_{f.Rk2}}{\gamma_n} = 307.692\ N$

$s_1 = 70\ mm$ Fastener spacing EN1991-1-4(9.21).

$b_i := 1200\ mm$ Wall panel width EN1991-1-4(9.22).

$h := 2.4\ m$ Height of the considered wall.

$b_0 := \frac{h}{2} = 1.2\ m$

$$c_i := \left\| \begin{array}{l} \text{if } b_i \geq b_0 \\ \quad \| 1 \\ \text{else} \\ \quad \left\| \frac{b_i}{b_0} \right. \end{array} \right| = 1$$

$F_{2.Rd} := \frac{F_{f.Rd2} \cdot b_i \cdot c_i}{s_1} = 5.275\ kN$ The design racking load-carrying capacity of each wall panel EN1991-1-4(9.21).

$F_{2.Rd} := 4 \cdot F_{2.Rd} = 21.099\ kN$ Three wall panels with a width of 1200 mm.

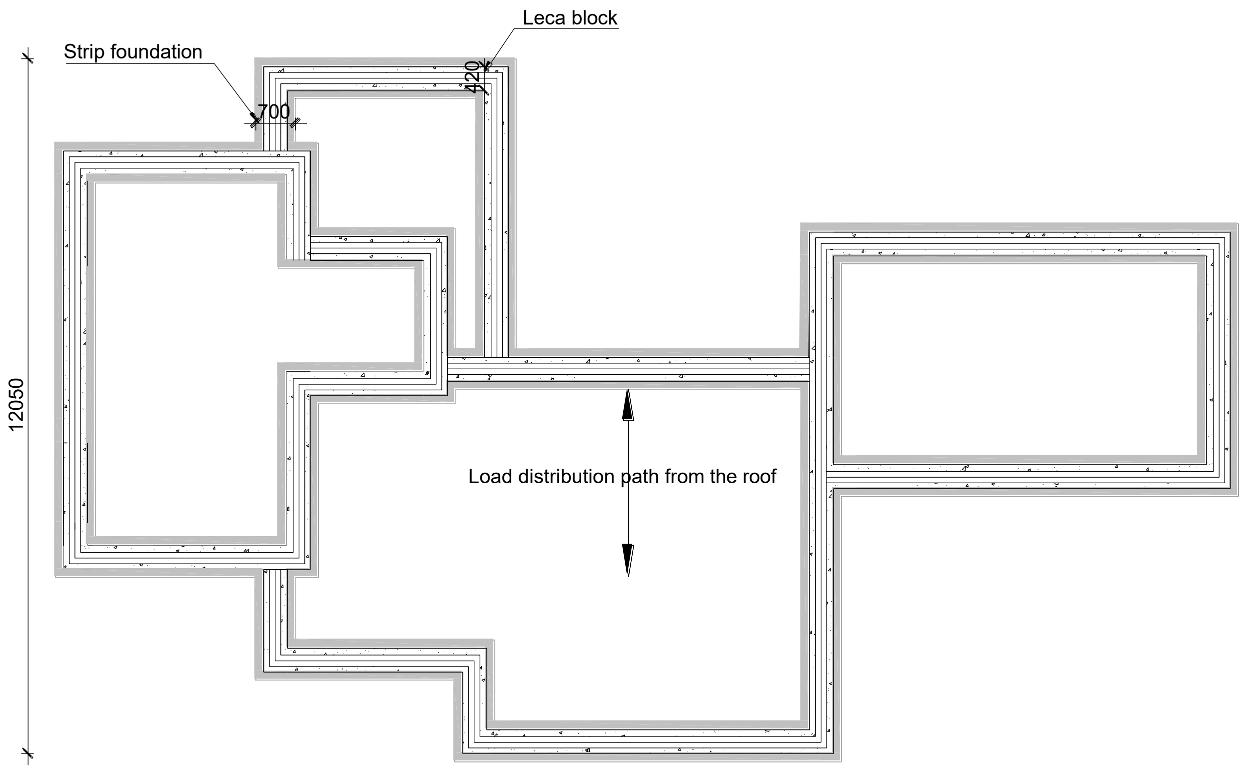
Summary of wall resistances

$$F_{Rd} := F_{1.Rd} + F_{2.Rd} + F_{3.Rd} + F_{4.Rd} + F_{6.Rd} + F_{7.Rd} + F_{8.Rd} + F_{9.Rd} + F_{10.Rd} + F_{11.Rd} + F_{12.Rd} = 90.138\ kN$$

$$\left. \begin{array}{l} \text{if } F_{Rd} < F_{Ed} \\ \quad \| \text{“Walls do not have enough resistance”} \\ \text{else} \\ \quad \| \text{“Have enough resistance”} \end{array} \right| = \text{“Have enough resistance”}$$

$UR := \frac{F_{Ed}}{F_{Rd}} = 95\%$

Appendix 12: Foundation design



Concrete class. C25/30

Type of soil. Sand moraine

Ground pressure. $P_g := 250 \ \frac{kN}{m^2}$

Soil capacity was assumed to be around 250 kN/m2 conservatively; there was no information regarding the soil capacity.

Dead load

The longest span of the building. $b_{span} := 12.05 \ m$

Tributary length. $T_{length} := \frac{b_{span}}{2} = 6.03 \ m$

Characteristic roof. $g_{D1.k} := 0.683 \ \frac{kN}{m^2}$

Characteristic dead load at the upper eaves. $g_{D2.k} := 0.2 \ \frac{kN}{m^2}$

Characteristic dead load. $g_{D.k} := g_{D1.k} + g_{D2.k} = 0.88 \ \frac{kN}{m^2}$

Characteristic dead load. $g_{D.k} := \left(g_{D1.k} + g_{D2.k}\right) \cdot T_{length} = 5.32 \ \frac{kN}{m}$

Snow load

Eaves length.	$e_{length} := 800\ mm$
Characteristic snow load.	$q_{S.k} := 2.4\ \frac{kN}{m^2}$
Characteristic dead load.	$q_{S.k} := q_{S.k} \cdot \left(T_{length} + e_{length}\right) = 16.38\ \frac{kN}{m}$

Self-weight of the floor.

The thickness of the wall.	$t_{wall} := 420\ mm$
Thickens of the concrete	$t_{floor} := 200\ mm$
The density of concrete.	$\gamma_c := 25\ \frac{kN}{m^3}$
Characteristic self-weight of the floor.	$g_{f.k} := t_{floor} \cdot \gamma_c = 5\ \frac{kN}{m^2}$
Characteristic self-weight of the floor.	$g_{f.k} := g_{f.k} \cdot \left(T_{length} - \frac{2 \cdot t_{wall}}{2}\right) = 28.03\ \frac{kN}{m}$

Live load

Characteristic live load.	$q_{L.k} := 2\ \frac{kN}{m^2}$
Characteristic live load.	$q_{L.k} := q_{L.k} \cdot T_{length} = 12.05\ \frac{kN}{m}$

Inner wall weight, it is assumed that partition walls will be made of concrete. The partition wall will be from timber material, but this calculation is safe.

The thickness of the partition wall; a conservative value.	$t_{pwall} := 130\ mm$
Height of partition wall.	$h_{pwall} := 2.5\ m$
Characteristic load of partition walls.	$q_{iw.k} := \gamma_c \cdot t_{pwall} \cdot h_{pwall} = 8.13\ \frac{kN}{m}$

Inner wall weight

Grade of timber material. C24

The density of the wood. $\gamma_{wood} := 4.2 \frac{kN}{m^3}$

The thickness of the partition $t_{upperwall} := 173\ mm$

Height of partition wall. $h_{upperwall} := 2.5\ m$

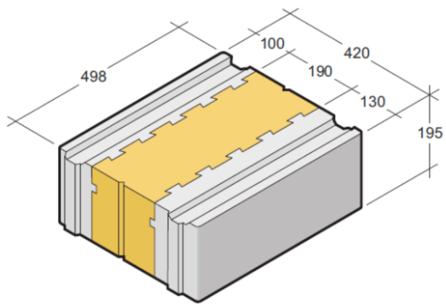
Characteristic load of partition walls. $n_{upperwall} := \gamma_{wood} \cdot t_{upperwall} \cdot h_{upperwall} = 1.82 \frac{kN}{m}$

The total load in the internal load-bearing part of the wall.

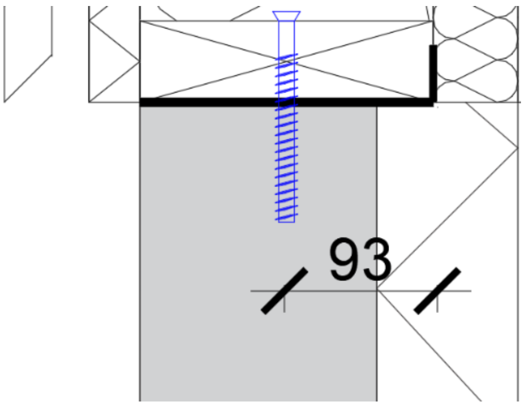
$$n_{ik} := g_{D.k} + q_{S.k} + 2 \cdot g_{f.k} + 2 \cdot q_{L.k} + q_{iw.k} = 109.98 \frac{kN}{m}$$

External wall

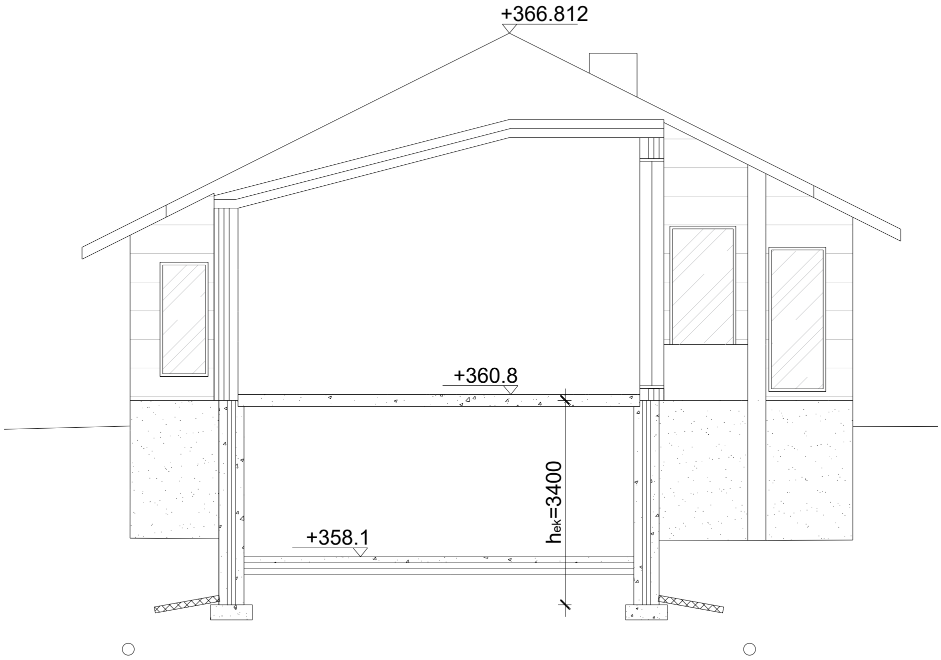
The load-bearing wall will consist of Leca blocks, LTH-420-6, below.



LTH-420
LTH-420-6 MPa



Stud element on top of the load bearing part of wall. The distance from center of the stud to the center fo the Leca block.

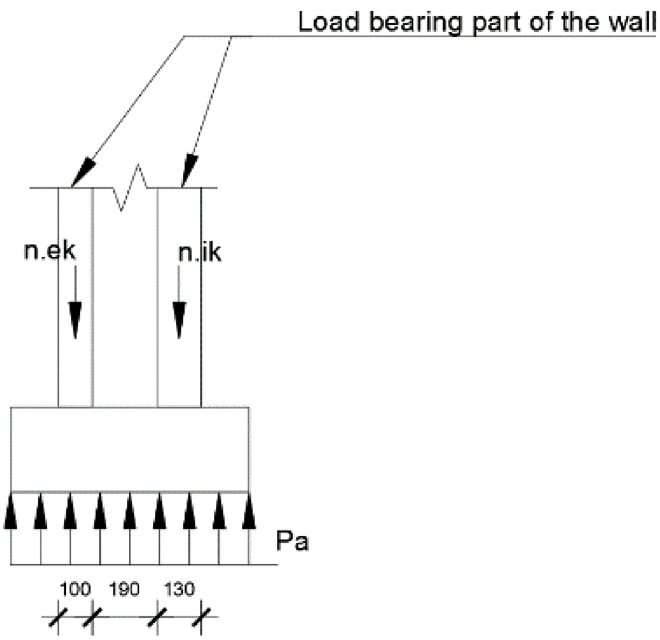


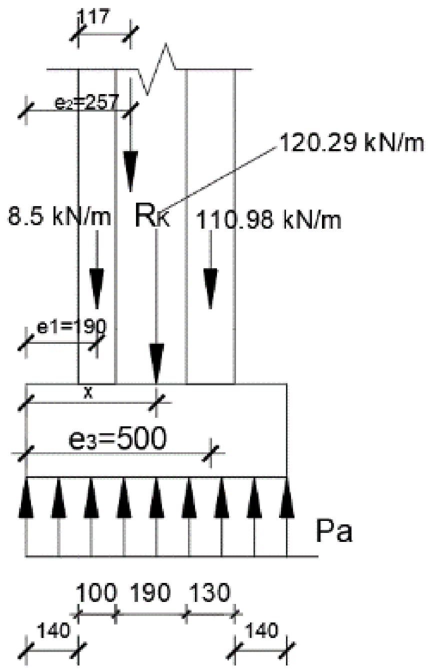
The thickness of the external wall. $t_{ek} := 100\ mm$

Height of the wall. $h_{ek} := 3.4\ m$

Value of outer part of the wall. $n_{ek} := \gamma_c \cdot t_{ek} \cdot h_{ek} = 8.5\ \frac{kN}{m}$

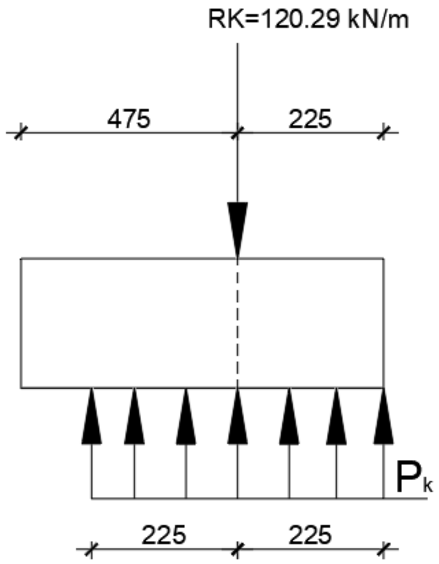
Assume preliminary width of 700 mm.





The eccentricity of the force.	$e_1 := 140\ mm + 50\ mm = 190\ mm$
The eccentricity of the force.	$e_2 := 117\ mm + 140\ mm = 257\ mm$
The eccentricity of the force.	$e_3 := 700\ mm - 140\ mm - 120\ \dfrac{mm}{2} = 500\ mm$
Distance of force from edge of the strip foundation.	$x := \dfrac{n_{ek} \cdot e_1 + n_{upperwall} \cdot e_2 + n_{ik} \cdot e_3}{R_k} = 474.43\ mm$

Ground pressure



Half-length of pressure part. $l := 225\ mm$

The effective length of ground pressure. $p_k := 2 \cdot l = 450\ mm$

Effective pressure. $P_k := \frac{R_k}{p_k} = 267.31\ \frac{kN}{m^2}$

$$\begin{array}{l} \textbf{if } P_k > P_g \\ \quad \| \text{“move footing adequately to the left”} \\ \textbf{else} \\ \quad \| \text{“okay!”} \end{array} = \text{“move footing adequately to the left”}$$

Defining new x distance

Distance to be moved. $e := 60\ mm$

The new eccentricity of the force. $e_{1n} := e_1 - e = 130\ mm$

The new eccentricity of the force. $e_{2n} := e_2 - e = 197\ mm$

The new eccentricity of the force. $e_{3n} := e_3 - e = 440\ mm$

Distance of center force to the edge of strip foundation. $x := \frac{n_{ek} \cdot e_{1n} + n_{upperwall} \cdot e_{2n} + n_{ik} \cdot e_{3n}}{R_k} = 414.43\ mm$

The x value is taken by personal choice. $x := 390\ mm$

Ground pressure

Half-length of pressure part. $l := 225\ mm + e = 285\ mm$

The effective length of ground pressure. $p_k := 2 \cdot l = 570\ mm$

Effective pressure. $P_k := \frac{R_k}{p_k} = 211.04\ \frac{kN}{m^2}$

$$\begin{array}{l} \textbf{if } P_k > P_g \\ \quad \| \text{“move footing to the right”} \\ \textbf{else} \\ \quad \| \frac{P_k}{P_g} \end{array} = 84.42\%$$

Okay!

Height of strip foundation.

Design normal force. $N_{Ed} := 1.15 \cdot (n_{ek} + n_{upperwall}) = 11.86 \ \frac{kN}{m}$

Design force from the inner part of the wall. $N_{id} := 1.15 \cdot (g_{D.k} + 2 \cdot g_{f.k} + q_{iw.k}) + 1.5 \cdot (q_{L.k} + 0.7 \cdot q_{L.k} + 0.7 \cdot q_{S.k}) = 127.85 \ \frac{kN}{m}$

Designed final normal force. $R_d := N_{id} + N_{Ed} = 139.71 \ \frac{kN}{m}$

Ground pressure

Effective pressure length. $P_k := 2 \cdot l = 570 \ mm$

The design value of ground pressure. $\delta_{gd} := \frac{R_d}{P_k} = 245.1 \ \frac{kN}{m^2}$

National Annex of Finland. $\alpha_{ct} := 0.85$

5% fractile tensile strength $f_{c.t.k} := 1.8 \ MPa$

Partial safety factor of concrete. $\psi_c := 1.5$

Design tensile strength of concrete. $f_{ctd} := \alpha_{ct} \cdot \frac{f_{c.t.k}}{\psi_c} = 1.02 \ MPa$

Adding the shift of foundation. $a := 140 \ mm + e = 200 \ mm$

Mathcad solver block is used below.

Solve Guess Values

$h_f := 250 \ m$ Guess value.

Constraints

$$\frac{0.85 \cdot h_f}{a} - \sqrt{\frac{3 \cdot \delta_{gd}}{f_{ctd}}} = 0$$

$h_{feff} := \text{find}(h_f) = 199.78 \ mm$

Designed the height of the foundation. $h_f := 250 \ mm$

Utilization ratio. $UR := \frac{h_{feff}}{h_f} = 79.91\%$

Conclusion.

Strip foundation with dimension 700x250 (mm).

Reinforcement design.

Concrete class. C20/25

Mean tensile strength. $f_{ctm} := 2.9\ MPa$

Designing for 1-meter strip footing. $b := 1000\ mm$

Depth of footing. $d := 250\ mm$

The thickness of the wall. $t := 420\ mm$

Steel characteristic yield strength. $f_{yk} := 500\ MPa$

Minimum reinforcement area.

$$A_{s.min} := min\left(\frac{0.26 \cdot f_{ctm} \cdot b \cdot t \cdot d}{f_{yk} \cdot m}, \frac{0.0013 \cdot d \cdot t \cdot d}{m}\right)$$

$$A_{s.min} = 34.13\ mm^2$$

Required reinforcement area.

$$A_{s.required} := \frac{A_{s.min}}{m} = 34.13\ mm^2$$

Maximum reinforcement area.

$$A_c := b \cdot h_f = 250000\ mm^2$$

$$A_{s.max} := 0.04 \cdot A_c = 10000\ mm^2$$

As it can be seen, the minimum required reinforcement area is very; therefore, a mesh can be selected as the reinforcement for the strip foundation to take care of any casualties about serviceability, such for example cracking.

Selected mesh. #5-150

Diameter of mesh reinforcement. $\varphi_s := 5\ mm$

The cross-section area of one 5 mm rebar. $A_{s0} := \pi \cdot \frac{{\varphi_s}^2}{4} = 19.63\ mm^2$

The width of the strip foundation will be 700; therefore, a mesh consisting of 4 rebars with a distance of 150mm would be enough for the design.

The number of 5mm rebars. $n := 4$

Provided area of reinforcement. $A_{s.provided} := n \cdot A_{s0} = 78.54\ mm^2$

Utilization ratio. $UR := \frac{A_{s.required}}{A_{s.provided}} = 43.4\%$

Condition.

$$\left.\begin{array}{l} \text{if } A_{s.provided} \le A_{s.max} \\ \quad \| \text{“Okay”} \\ \text{else} \\ \quad \| \text{“Check”} \end{array}\right| = \text{“Okay”}$$

Appendix 13: Floor design, Ruukki's solution

ruukki

COMSLAB
Two-apartement house

v. 1.0.11
2022-05-14 15:07 (GMT)
RESULTS ARE VALID ONLY FOR RUUKKI COMPOSITE SHEETS

Project: Two-apartement house

Updated: 2022-04-28 18:53 (GMT)

Created: 2022-04-20 14:22 (GMT)

Customer:

National annex: Finnish NA

Contact person: Ledion Shqefni

Engineer's contact info: Visamäentie 25 B 14 b

Email: ledion1800@student.hamk.fi

Telephone number: 0414987738

ID	Structural part	Updated	Created
1	Floor Slab	2022-04-28 18:53 (GMT)	2022-04-22 18:40 (GMT)

v. 1.0.11

COMSLAB

Two-apartement house

2022-05-14 15:07 (GMT)

RESULTS ARE VALID ONLY FOR RUUKKI COMPOSITE SHEETS

Structural part: Floor Slab

Updated: 2022-04-28 18:53 (GMT) **Version:** 1.0.10 (2022-04-20)

Created: 2022-04-22 18:40 (GMT)

Reliability class: RC2

Deflection limit: L/100 (according to Eurocode)

Exposure class: XC3, **Intended service life:** L50, **Allowed crack width:** 0.3 mm

Concrete strength class: C25/30

Reinforcement grade: A500HW / B500B

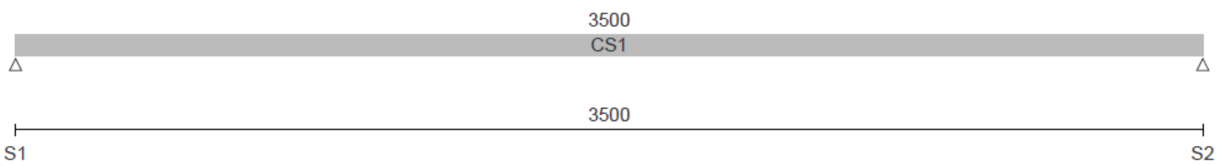
Concrete density: 25 kN/m^3,

Fire resistance class: R30

Concrete top cover: 30 mm, **Concrete bottom cover:** 47 mm

No fire design is conducted for the given fire resistance class R30. According to 4.3.2(5) of EN 1994-1-2, the fire resistance of composite slabs under the load bearing criterion "R" is at least 30 minutes.

Structural model



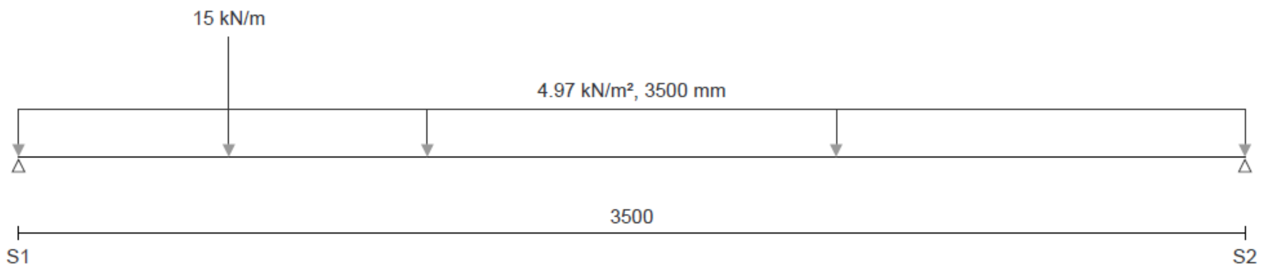
Spans

Span	Length	Slab thickness	Sheet	Sheet thickness
	[mm]	[mm]		[mm]
CS1	3500	200	CS48-36-750 Zn	0.7

Supports

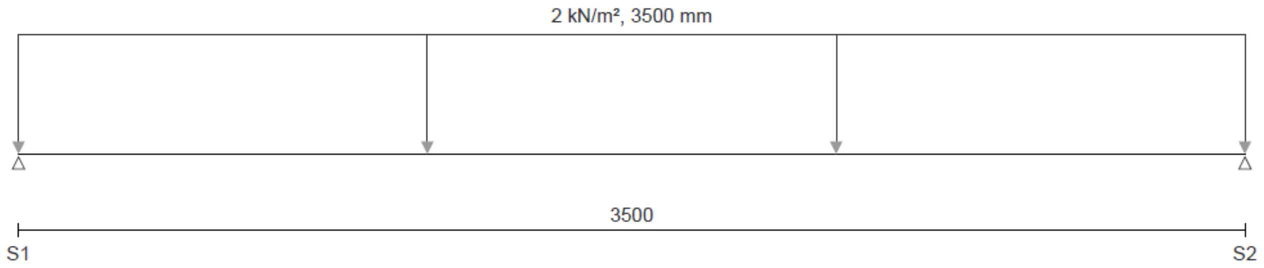
Support	Width
	[mm]
S1	100
S2	100

Dead load



Live load

Load category: A: areas in residental buildings



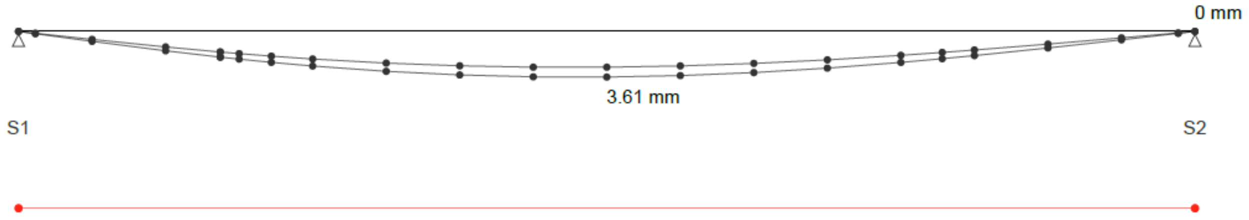
Span utilization ratios

	M_{pos} [kNm/m]	M_{neg} [kNm/m]	V_V [kN/m]	D_{ST} [mm]	D_{LT} [mm]	W_{ST} [mm]	W_{LT} [mm]
CS1	16.1 / 52.0 31.0 %	0.0 / 0.0 0.0 %	29.0 / 79.3 37.0 %	3.6 / 14.0 26.0 %	3.8 / 14.0 27.0 %	0.0 / 0.0 0.0 %	0.0 / 0.0 0.0 %

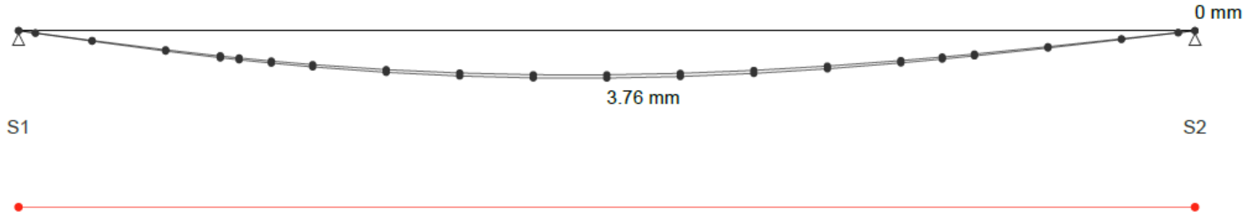
COMSLAB
Two-apartement house
FLOOR SLAB

v. 1.0.11
2022-05-14 15:07 (GMT)
RESULTS ARE VALID ONLY FOR RUUKKI COMPOSITE SHEETS

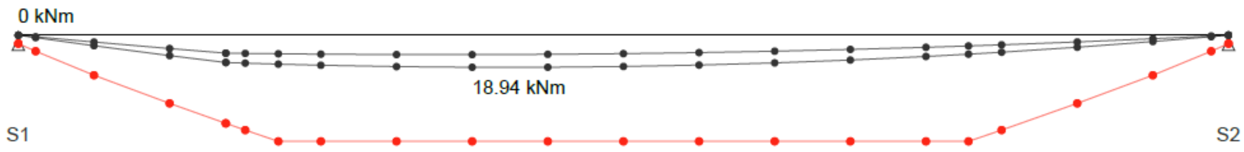
Deflection, short term



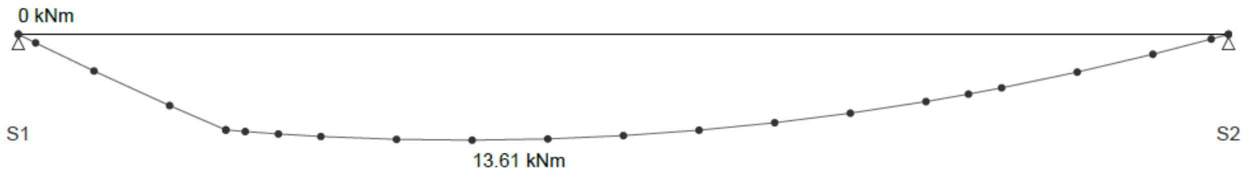
Deflection, long term



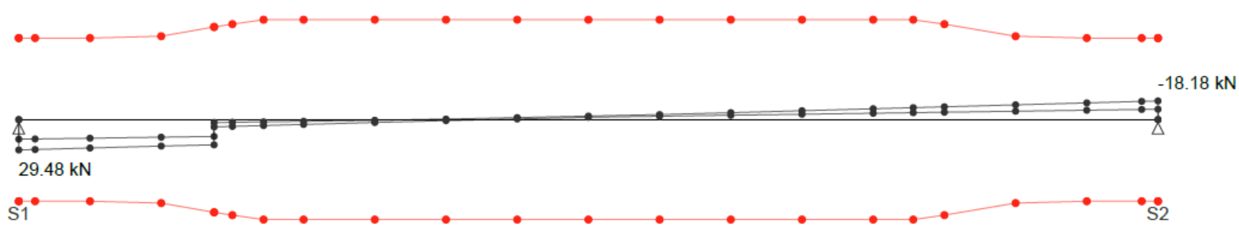
Bending moment



Fire situation



Shear force



Support reactions

Support	Min	Max
	[kN]	[kN]
S1	19.02	29.48
S2	10.14	18.18

Reinforcements

Span reinforcements

-

Support reinforcements

-

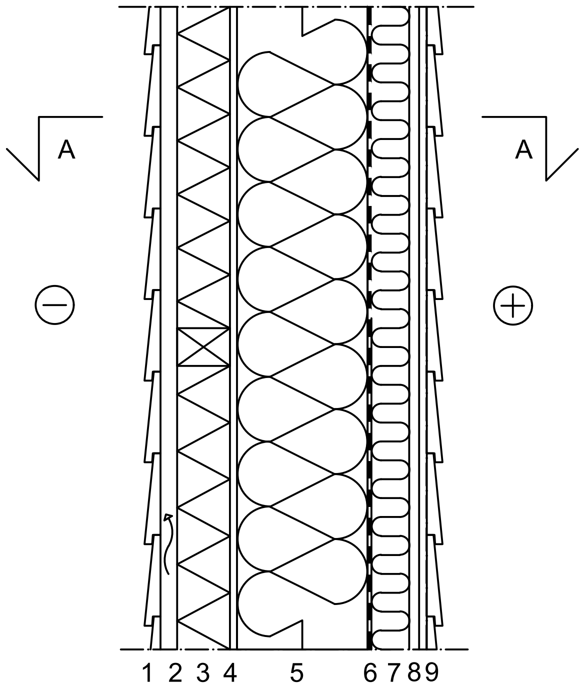
Transverse reinforcement

Type	Required area	Selected area
	[mm^2/m]	[mm^2/m]
Ø4-150	80	83

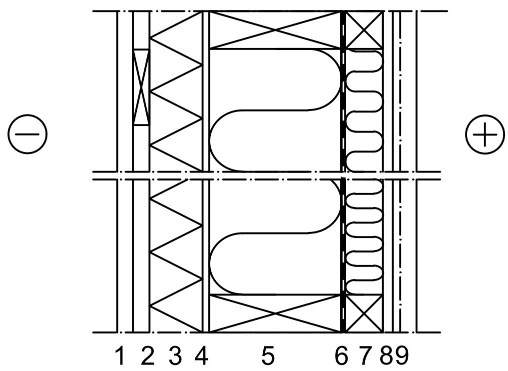
Propping

	Distance from structure end [mm]	Distance from support [mm]
CS1		
1	1183	S1 + 1183
2	2317	S1 + 2317

Two apartment house	Timber wall	
Ledion Shqefni	1	US-1
	07.05.2022	



Section A-A

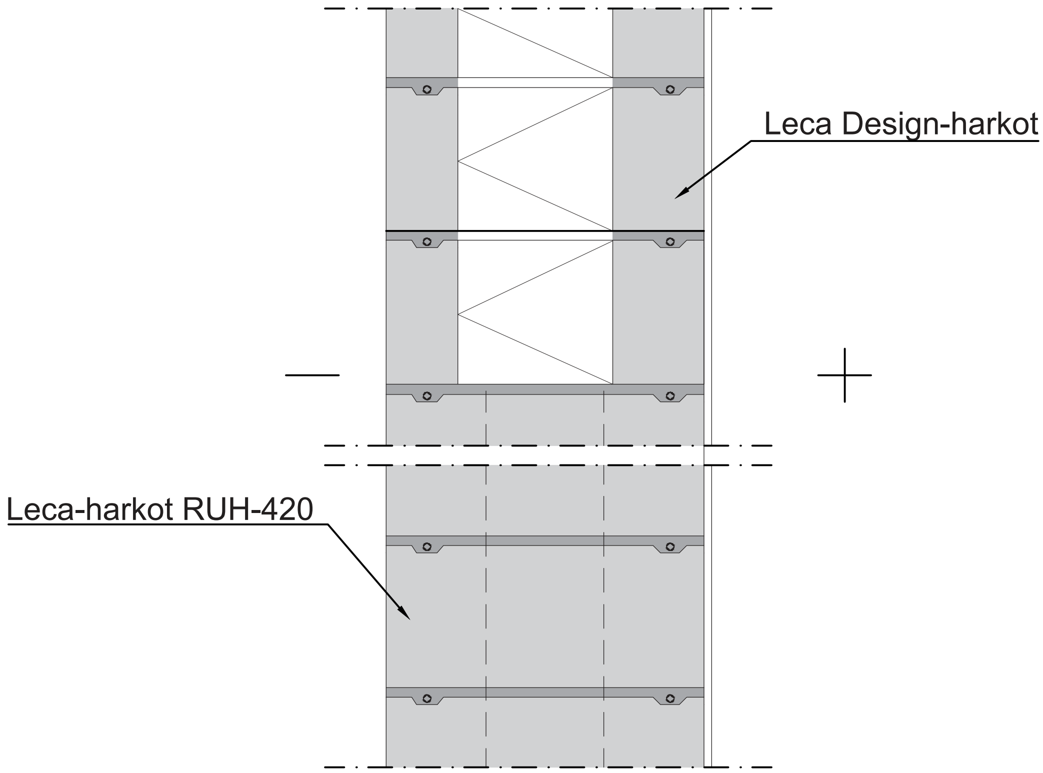


1 Timber cladding 28x220
22 mm 2 VENTILATION SPACE + board 22x100mm, k600
68 mm 3 WINDSCREEN INSULATION, stone wool PAROC Cortex pro (1200x1800) and collar 68mm, collar spacing 1200mm
9 mm 4 WINDSCREEN PLASTERBOARD or equivalent (can be used to stiffen the structure if necessary)
175 mm 5 THERMAL INSULATION, PAROC eXtra stone wool and PORTABLE WOOD FRAME C24 175x50mm, k600
6 STEAM BARRIER between body and collar
50 mm 7 THERMAL INSULATION, stone wool PAROC eXtra and collar 48x48mm, k600
13 mm 8 BUILDING BOARD, gypsum board
9 Timber cladding 28x220

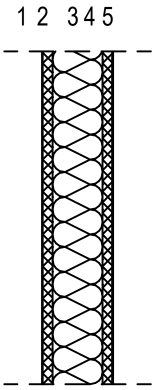
U-value 0.13 W / m²K
Fire class REI 60
Airborne sound insulation value R / w 57 dB, R / w + C 50 dB, R / w + C / tr 42 dB

FIRE LOAD <20 MJ / m² INSULATED INSULATION IN THE STRUCTURE

Two apartment house	Basement wall	
Ledion Shqefni	3 07.05.2022	US-2



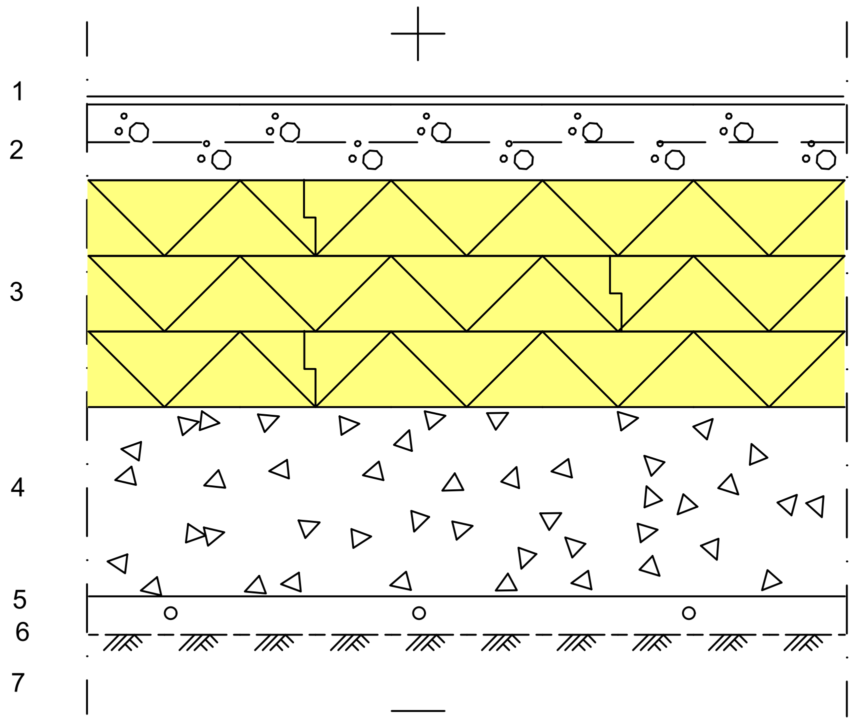
Two apartment house	Timber wall	
Ledion Shqefni	4 07.05.2022	VS



1. Surface Layer, 1 [mm]
2. Wood boards 28x220
3. Wooden frame min. 45 x 66 mm, k600mm + ISOVER ACOUSTIC
4. Wood boards 28x220
5. Surface Layer, 1 [mm]

Total thickness = 124 [mm]
Fire class = EI 30
Airborne sound insulation = Rw 40 dB, Rw+C 37 dB, Rw+Ctr 35 dB

Two apartment house	Concrete floor structure	
Ledion Shqefni	5 07.05.2022	AP



1 Floor covering the room as explained
2 Reinforced concrete slab 80 ... 100 mm, = 1.7 W / mK, reinforcement according to the construction plan
3 Finnfoam FL-300 insulation 300 mm
- Thermal conductivity 0.037 W / mK to 0.038 W / mK
- Short-term compressive strength CS (10) 250 kPa
- Water absorption by immersion WL (T) 0.7
- Water vapor permeability = 150
- Dimensional stability DS (70,90)
- Load deflection CC (2 / 1,5 / 50) 130 kPa (used as design basis)

4 Crushed stone 8 ... 16 mm> 200 mm
5 Sand layer with passive heat / cold pipes
6 Filter cloth
7 Base, tilt to the secret 1:50

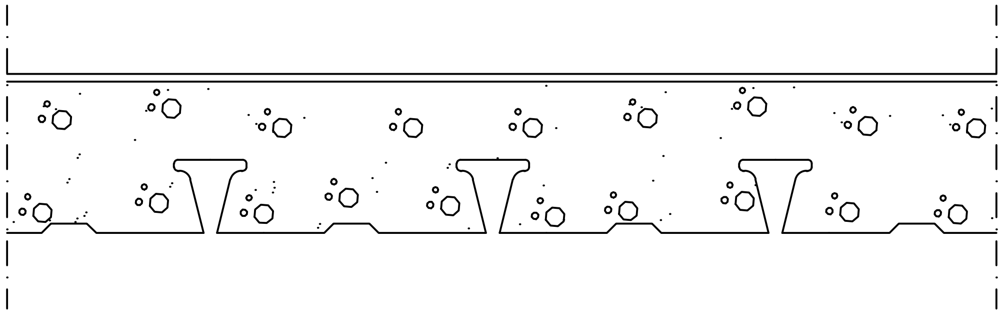
U-VALUE taking into account the geometry of the building (RakMk C4 2012): 0,10 W/m K ^{2}

YmA 1010/2017 §33 level

NOTE!

- On underground floors, Finnfoam XPS insulation is dimensioned for thermal conductivity
  $\leq$ 70 mm:n plate thickness λ_U=0,036 W/mK
  $\geq$ 80 mm:n plate thickness λ_U=0,038 W/mK
  Dimensioning The values of thermal conductivity λU have been corrected by the addition of the moisture conversion factor SFS-EN 10456 according to the instructions in the draft RIL 225.

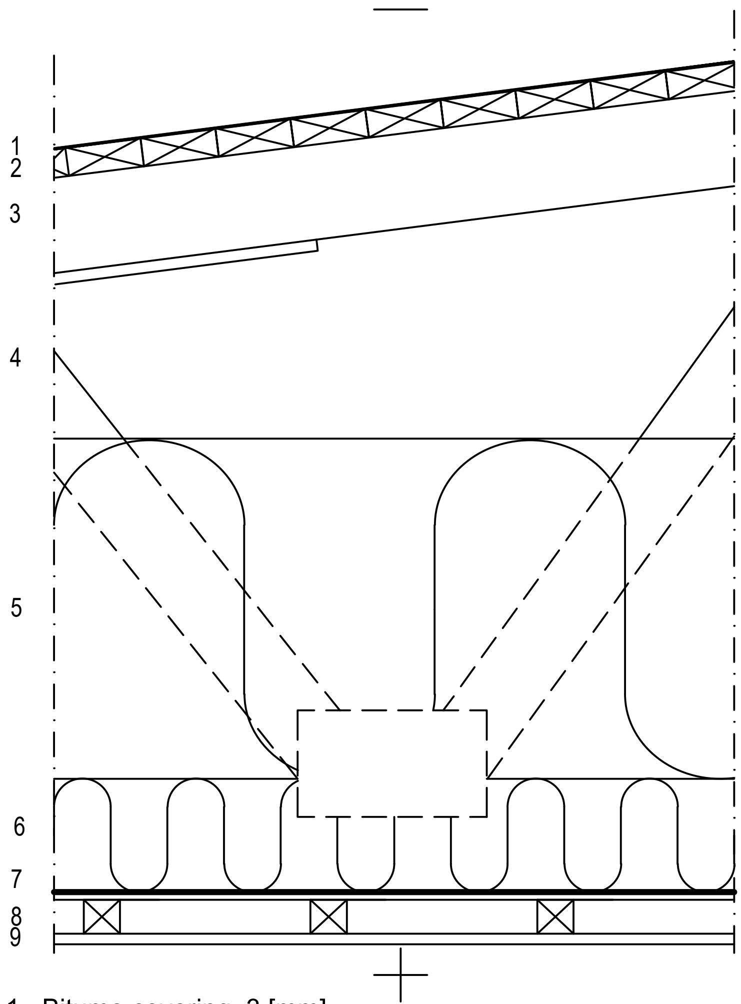
Two apartment house	Composite concrete slab	
Ledion Shqefni	6 07.05.2022	VP



LAYERS

1 SURFACE MATERIAL ACCORDING TO DESIGNER SPECIFICATION
2 REINFORCED CONCRETE SLAB ACCORDING TO CONSTRUCTION DRAWING
3 LOAD-BEARING PROFILED SHEET MOULD
ACCORDING TO CONSTRUCTION DRAWING
REINFORCEMENTS ACCORDING TO CONSTRUCTION DRAWING

Two apartment house	Roof structure	
Ledion Shqefni	2 07.05.2022	YP



1. Bitume covering, 3 [mm]
2. Wood board, 95x23 [mm]
3. Truss element.
4. Cold ventilated space
5. PAROC XST 013, 450 [mm]
6. PAROC soft thermal insulation, 150 [mm]
7. Vapor barrier, 1 [mm]
8. Wood frame, 48x48, k300 [mm]
9. Wood board, 14 [mm]

Total thickness = 344 [mm]
Fire class = P2/REI 60

REI 60, load 5.5 kN / frame post / 9.2 kN / m
(EUFI29-19003518-T1)
(insulation: PAROC eXtra 150 mm + PAROC Cortex 30 mm)

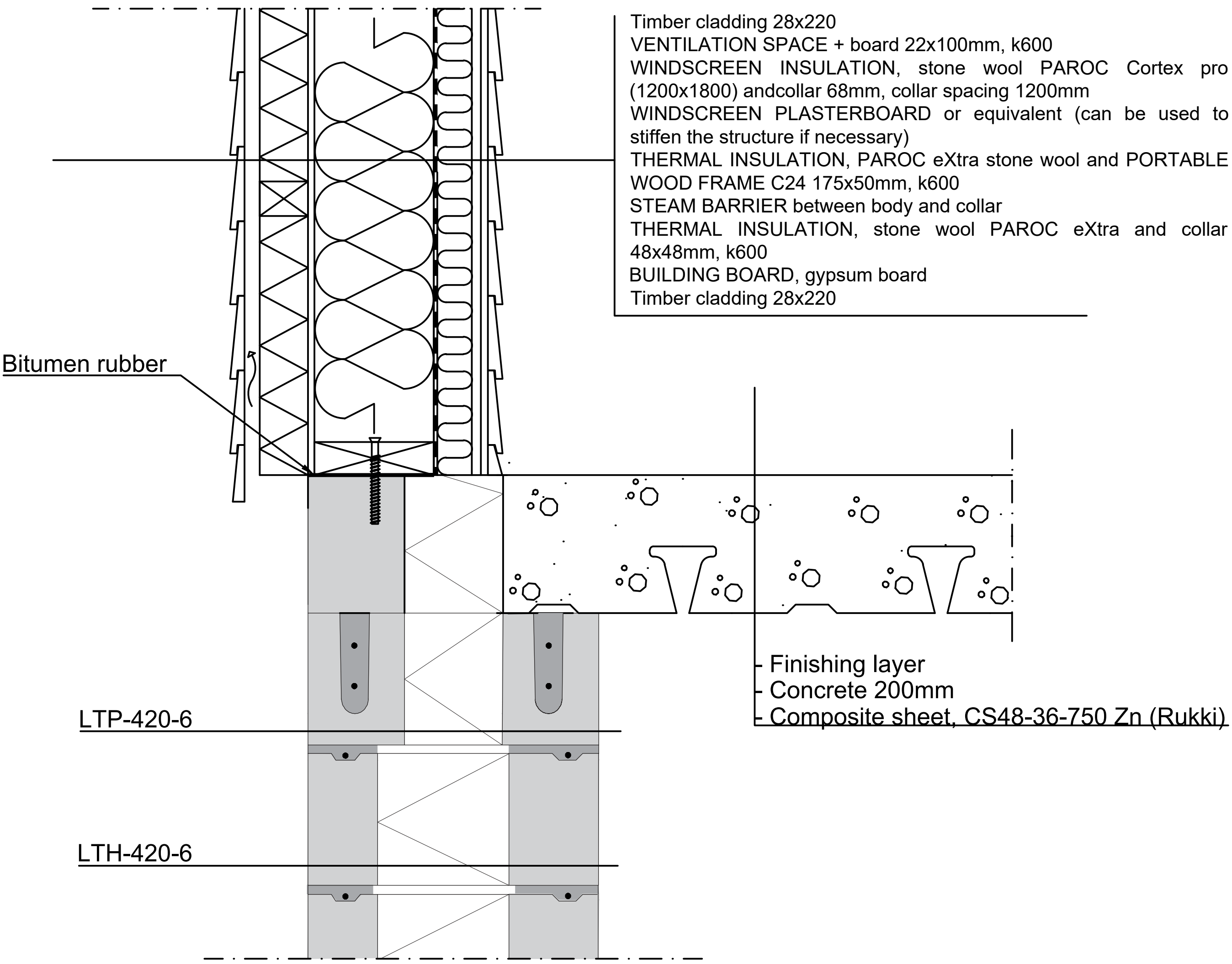
Rw 42 dB / Rw + C 40 dB / Rw + Ctr 37 dB
(insulation: PAROC eXtra 50 + 125 mm + PAROC Cortex pro 40 mm)

U-value = 0.12 [W/(m²·K)]

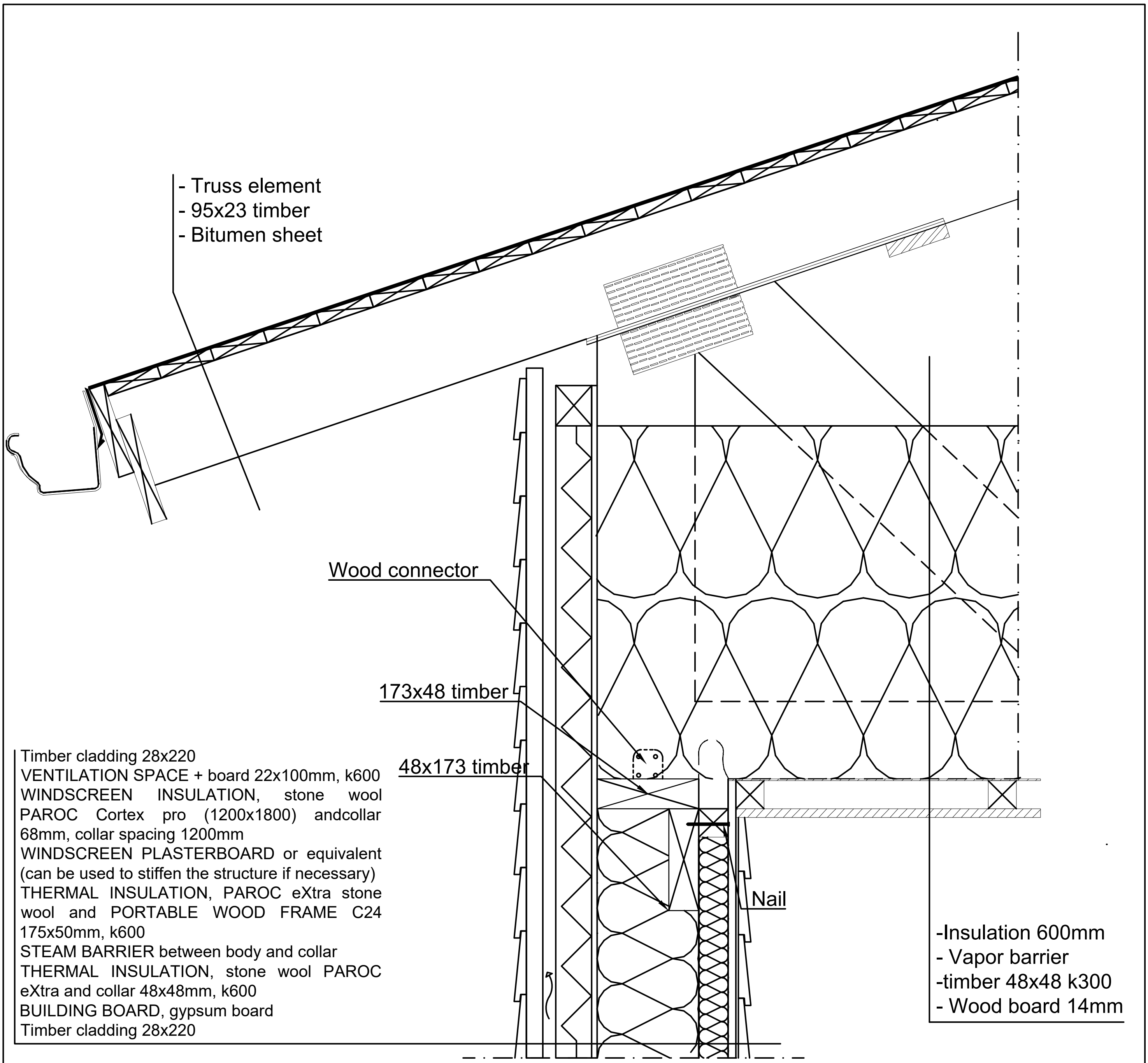
U-value correction term ΔU = 0.000 W / m2 / K.

Appendix 15: Detail connection

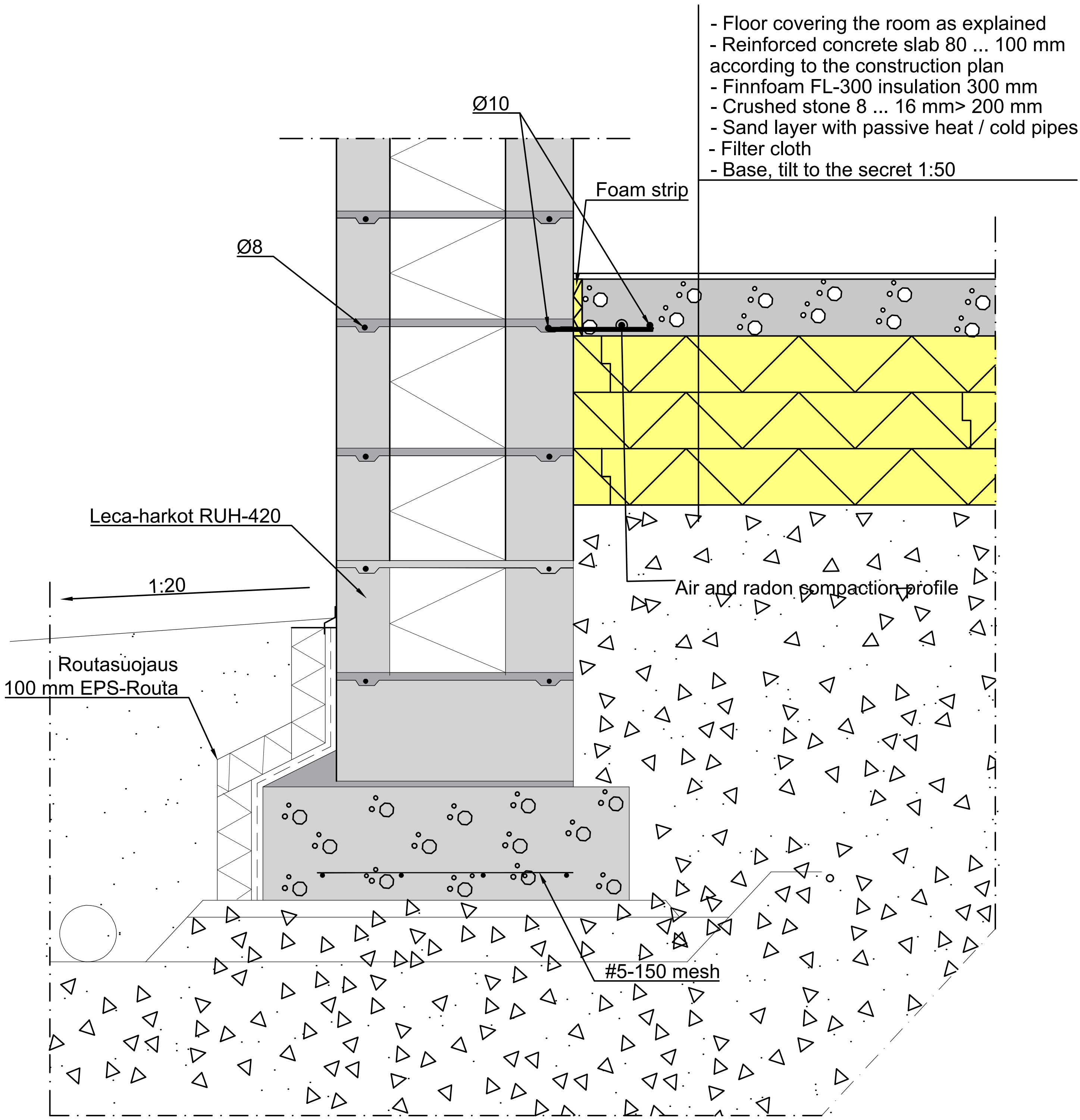
Two apartment house	Detail connection 1	
Ledion Shqefni	7 14.05.2022	US1-VP



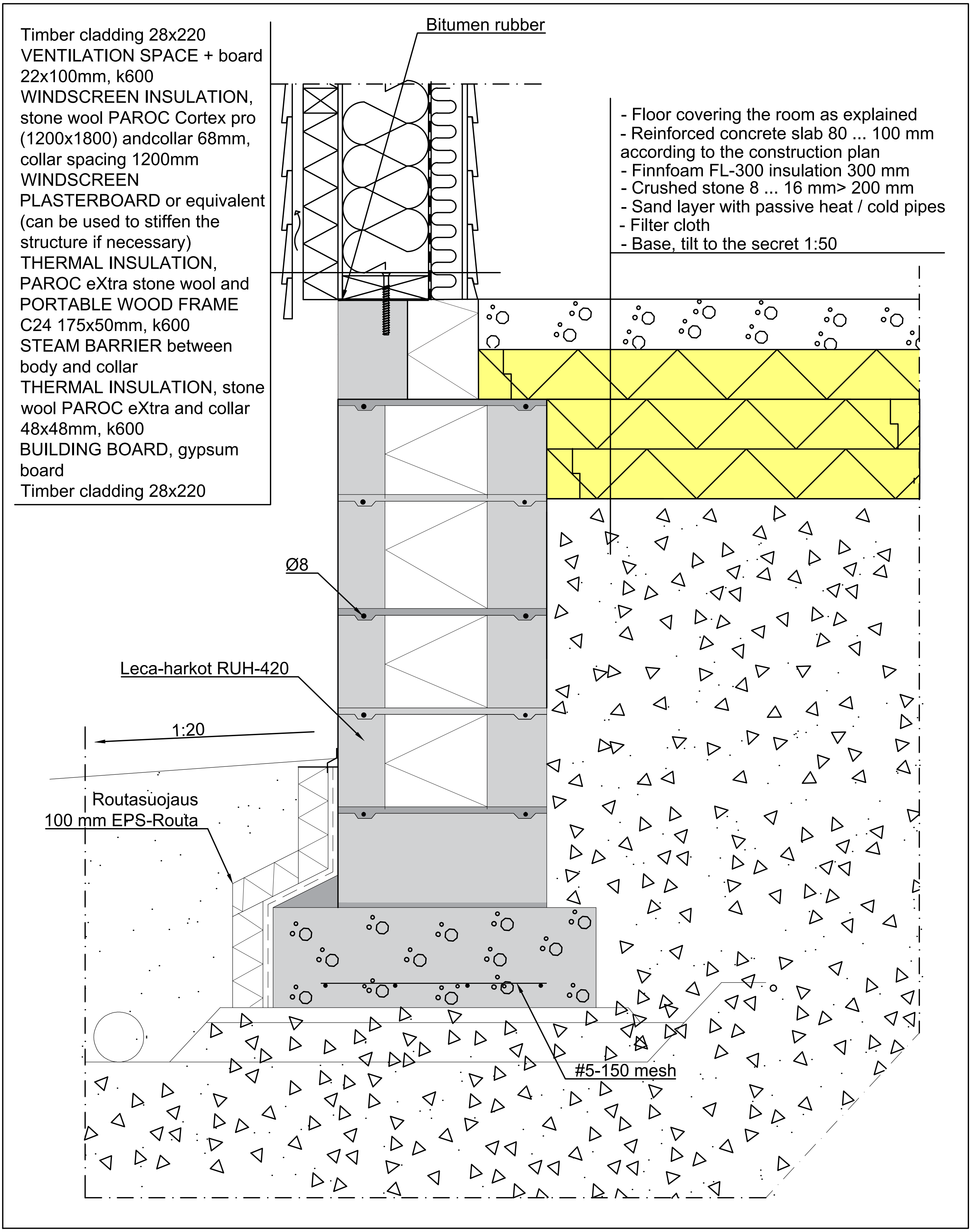
Two apartment house	Detail connection 2	
Ledion Shqefni	8 14.05.2022	YP-US



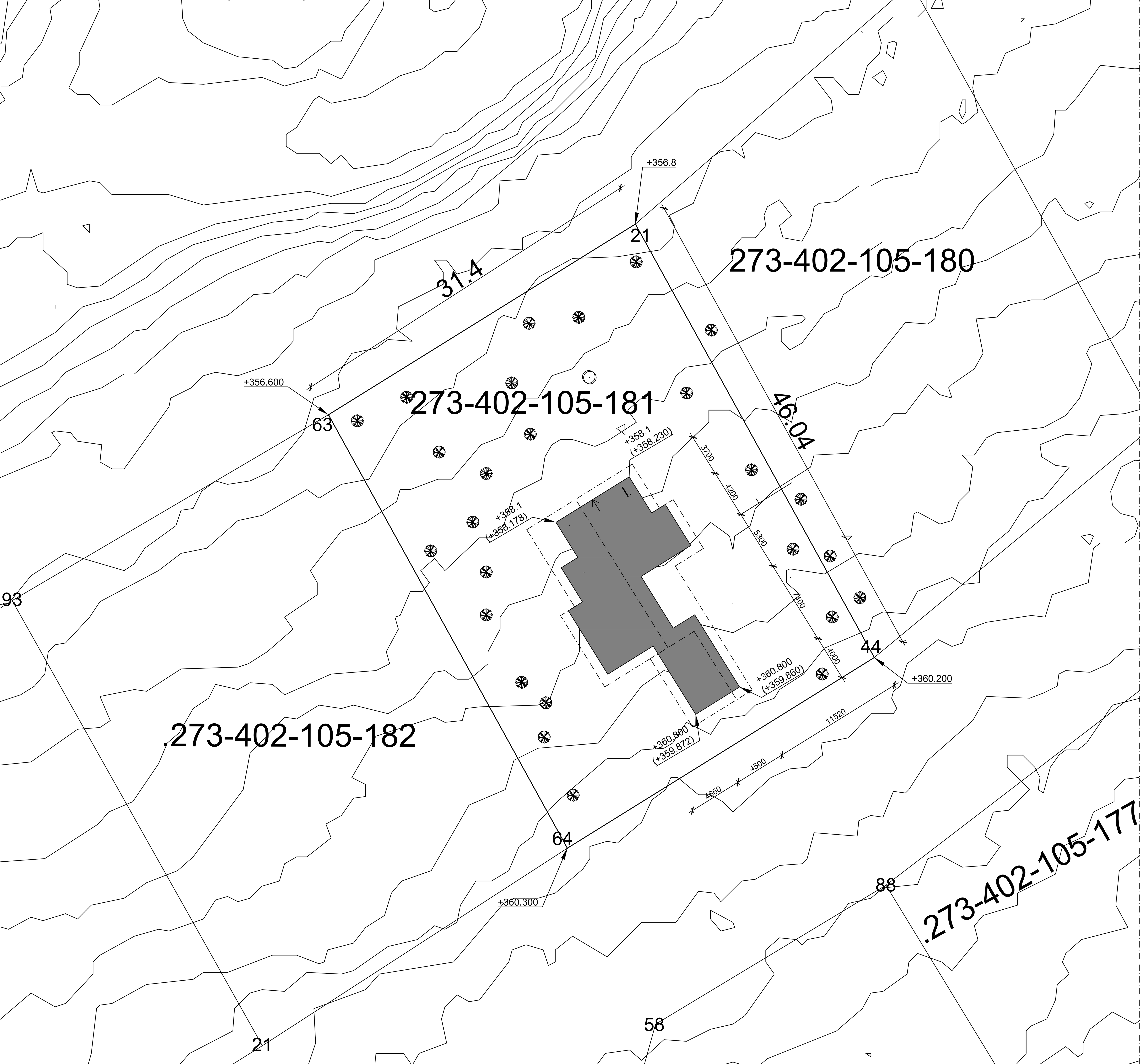
Two apartment house	Detail connection 2	
Ledion Shqefni	9 / 14.05.2022	US1-AP



Two apartment house	Detail connection 4	
Ledion Shqefni	10 14.05.2022	US1-US2-AP



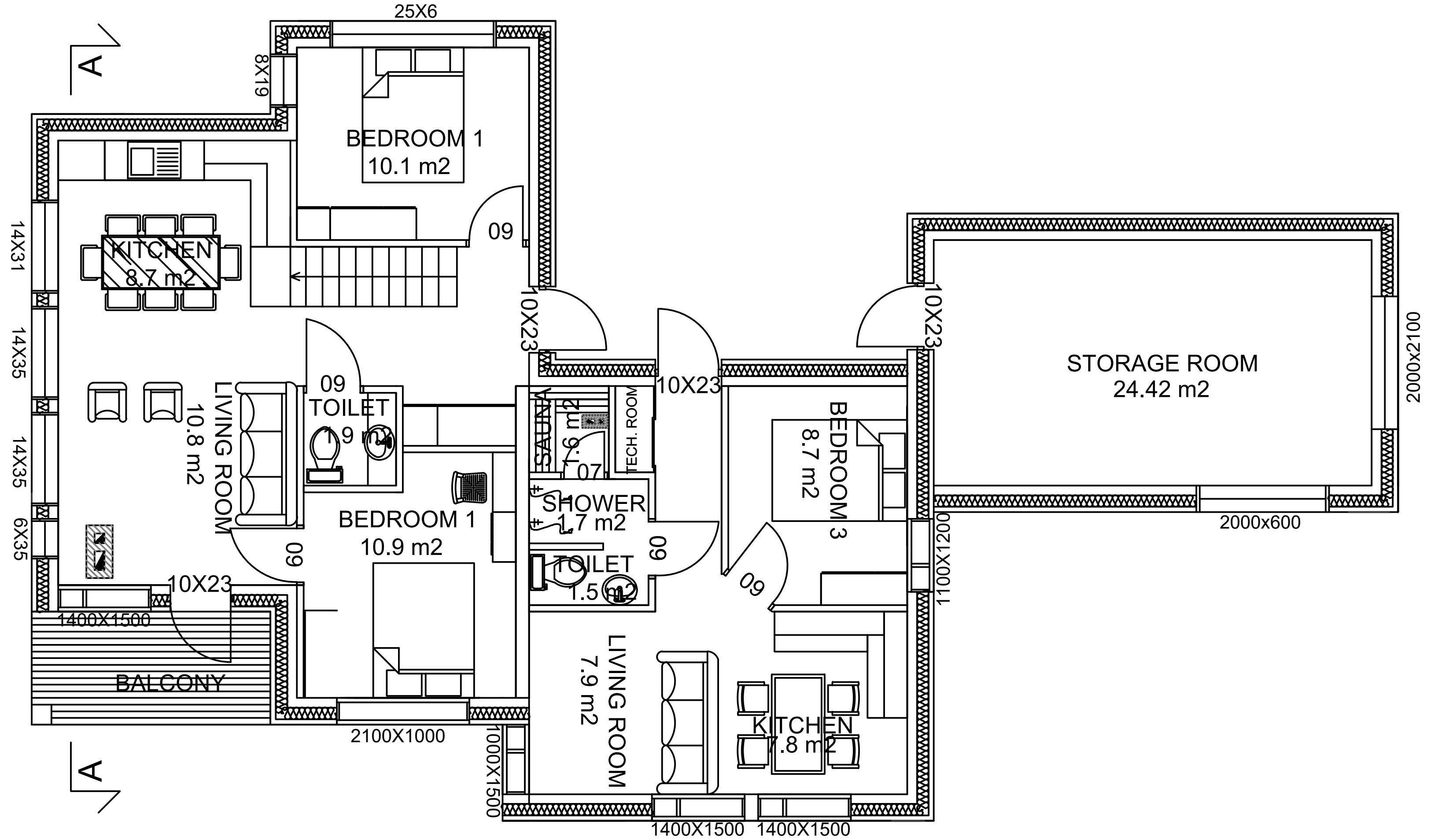
Appendix 16: Building plot drawing



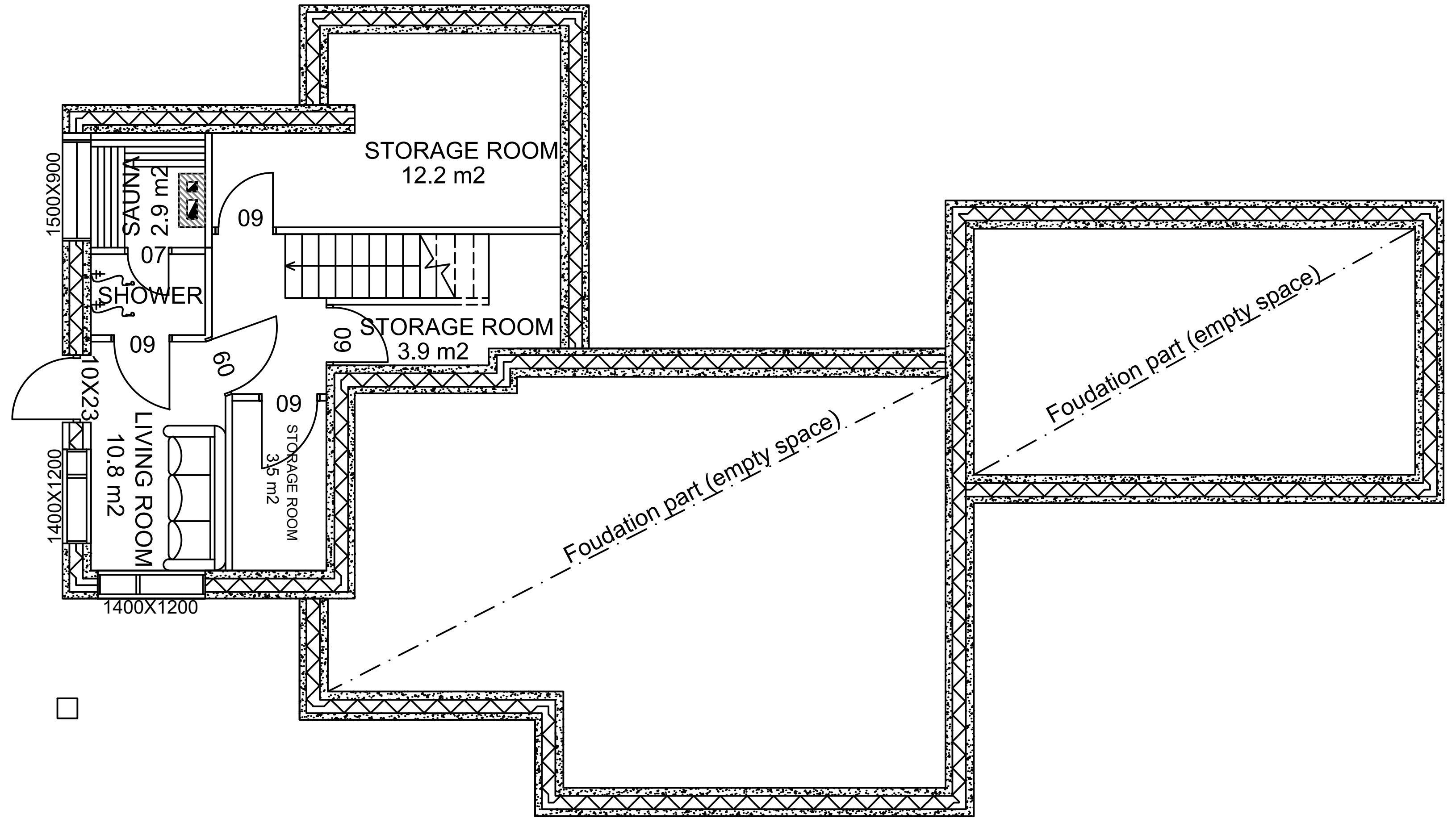
Kolari

Building plot 1350 m2
Building area 152 m2
Floor area 120 m2
Volume 514 m3

K.OSA	KORTTELI/TILA	TONTTI/RNo	RAKENNUSLUVAN TUNNUS			
1	116	4				
RAKENNUSTOIMENPIDE			PIIRUSTUSLAJI			JUOKS.No
NEW BUILDING			ARCHITECTURAL DRAWING			4
RAKENNUSKOHTEEN NIMI JA OSOITE			PIIRUSTUKSEN SISÄLTÖ			MITTAKAAVA
TWO APARTMENT HOUSE Kolari			Building plot			1:200
			SUUNN.ALA	TYÖ No	PIIR.No	MUUTOS
			PLOT DRAWING			
			PÄIVÄYS	YHT.HENK.	TIEDOSTO	PIIRTÄJÄ
LEDION SHQEFNI			14.05.2022			



1ST FLOOR



BASEMENT

BUILDING AREA 152 m2
BUILDING VOLUME 514 m3

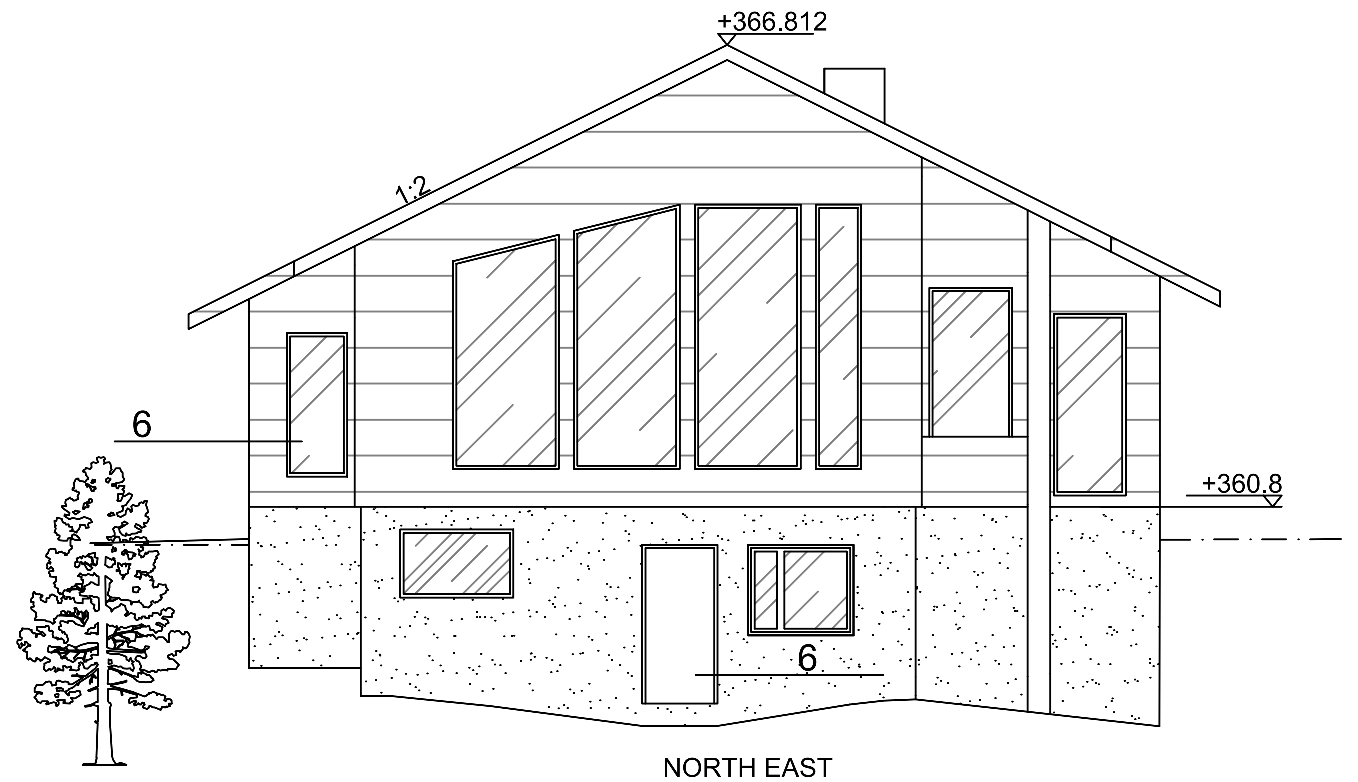
GEOTHERMAL HEATING PUMP+
FLOOR HEATING

AIR VENTILATION WITH HR (>65%)

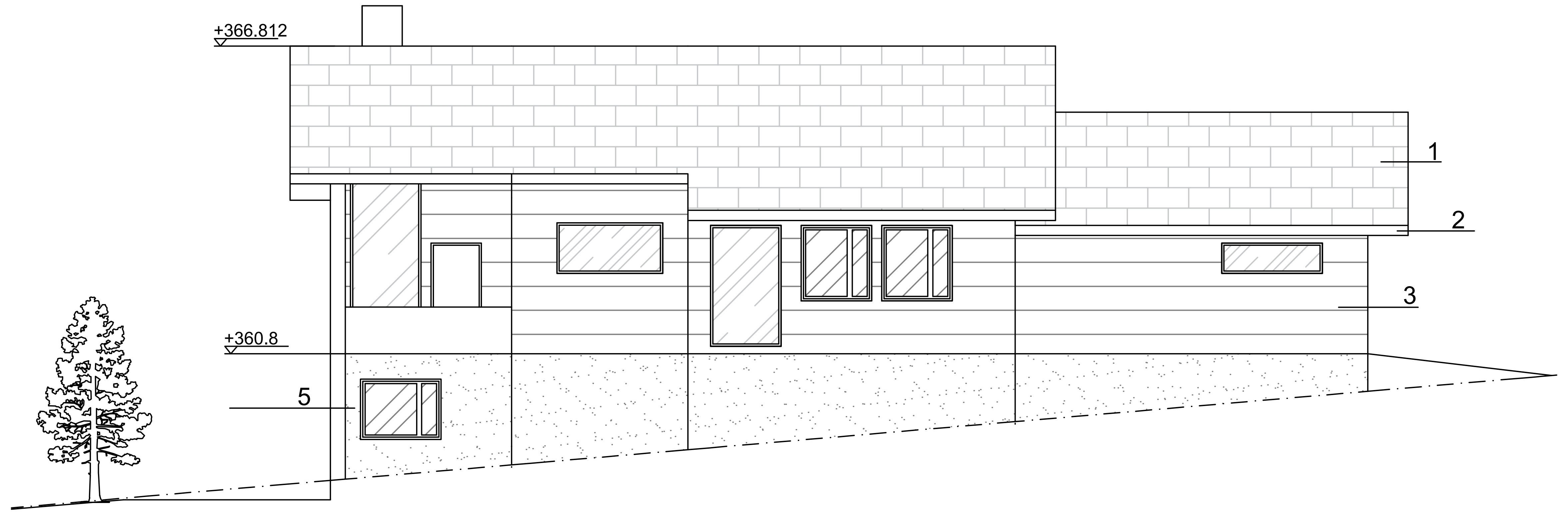
ALL WINDOWS WHICH GLASS <700mm FROM FLOOR LEVEL SAFETY GLASS

FIRE CLASS OF BUILDING IS P3

K.OSA	KORTTELI/TILA	TONTTI/RNo	RAKENNUSLUVAN TUNNUS	
1	116	4		
RAKENNUSTOIMENPIDE			PIIRUSTUSLAJI	JUOKS.No
NEW BUILDING			ARCHITECTURAL DRAWING	1
RAKENNUSKOHTEEN NIMI JA OSOITE			PIIRUSTUKSEN SISÄLTÖ	MITTAKAAVA
TWO APARTMENT HOUSE Kolari			FLOOR LAYOUT	1:100
			SUUNN.ALA / TYÖ No / PIIR.No / MUUTOS FLOOR PLANS	
LEDION SHQEFNI			PÄIVÄYS 07.05.2022 / YHT.HENK. / TIEDOSTO / PIIRTÄJÄ	

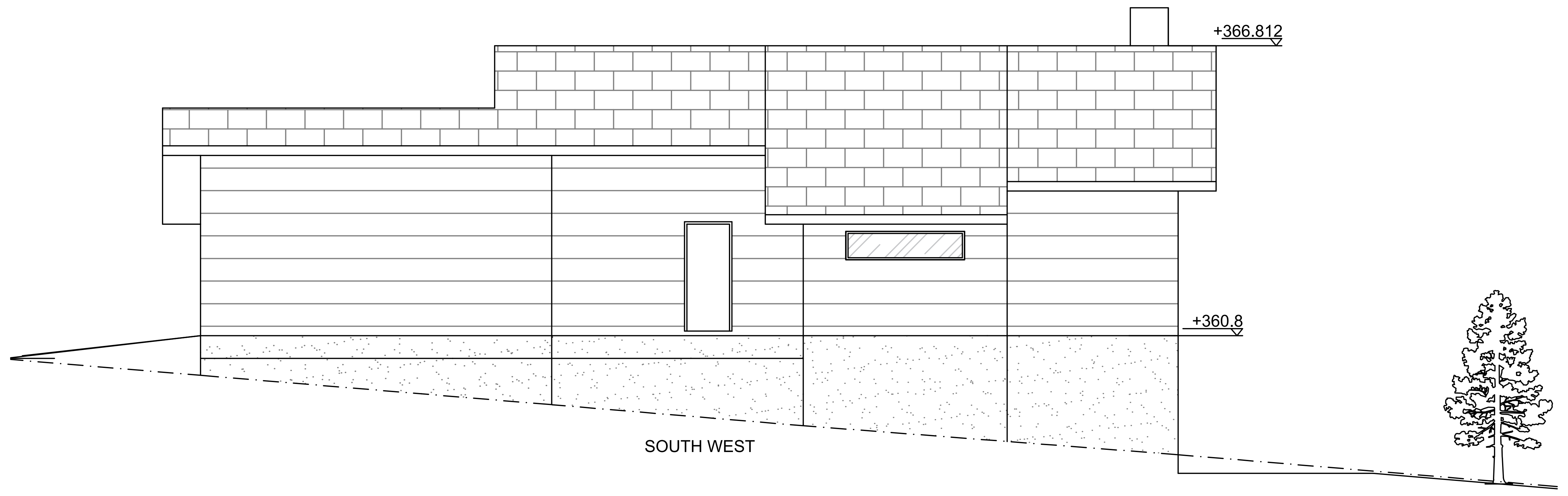


NORTH EAST



NORTH EAST

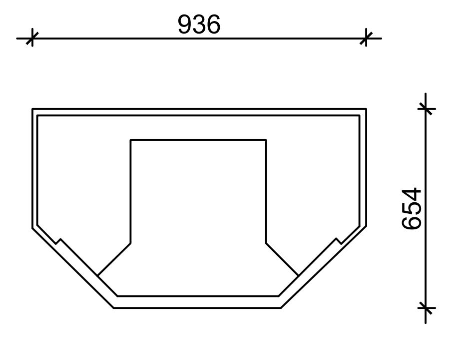
SOUTH EAST



SOUTH WEST

1. ROOF,BITUMEN SHEET
2. EAVES,WOOD,WHITE619X
3. HORIZONTAL PANEL,WOOD,S1005-Y50R
4. WINDOWS DECORATIONS,WOOD,6030-Y5030
5. BASEMENT WALL,PLASTERING,GRAY
6. WINDOW AND DOOR,WHITE

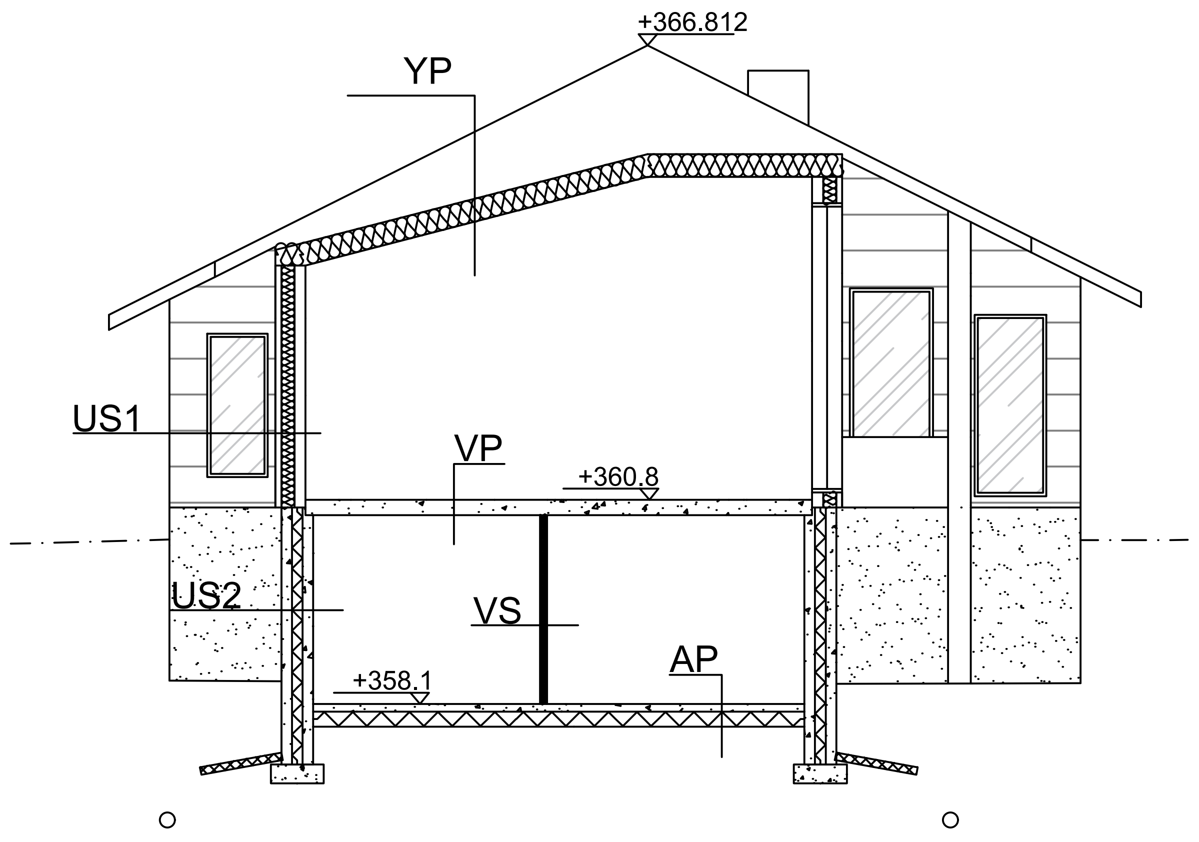
ROOF REQUIREMENTS MUST FOLLOW REGULATION 1007/2017



CHIMNEY 1:20

K.OSA 1	KORTTELI/TILA 116	TONTTI/RNo 4	RAKENNUSLUVAN TUNNUS
RAKENNUSTOIMENPIDE NEW BUILDING			PIIRUSTUSLAJI ARCHITECTURAL DRAWING — JUOKS.No 2
RAKENNUSKOHTEEN NIMI JA OSOITE TWO APARTMENT HOUSE Kolari			PIIRUSTUKSEN SISÄLTÖ FLOOR LAYOUT CHIMNEY 1:20 — MITTAKAAVA 1:100
LEDION SHQEFNI			SUUNN.ALA AR — TYÖ No — PIIR.No — MUUTOS; PÄIVÄYS 07.05.2022 — YHT.HENK. — TIEDOSTO — PIIRTÄJÄ

Appendix 19: Section drawing



YP
Bitume covering, 3 [mm]
Wood board, 95x23 [mm]
Truss element
Cold ventilated space
PAROC XST 013, 450 [mm]
PAROC soft thermal insulation, 150 [mm]
Vapor barrier, 1 [mm]
Wood frame, 48x48, k300 [mm]
Wood board, 14 [mm]

US 1
Wood cladding, 28 [mm]
Ventilation gap with vertical bending, 22 [mm]
Wind barrier, PAROC Cortex pro, 30 [mm]
Wooden frame (C24), k600 / PAROC soft thermal insulation, 48x173 [mm]
PAROC Vapor / air barrier, joint taping: PAROC XST 013
Enclosure, k600 / PAROC soft thermal insulation, 50 [mm]
Gypsum board, 13 [mm]
Surface layer, 1 [mm]

US 2
Leca block 420 [mm]

AP
Floor covering the room as explained
Reinforced concrete slab 80 ... 100 mm, = 1.7 W / mK, reinforcement according to the construction plan
Finnfoam FL-300 insulation 300 mm
Crushed stone 8 ... 16 mm> 200 mm
Sand layer with passive heat / cold pipes
Filter cloth
Base, tilt to the secret 1:50

VS
Surface Layer, 1 [mm]
Wood boards 28x220
Wooden frame min. 45 x 66 mm, k600mm + ISOVER ACOUSTIC
Wood boards 28x220
Surface Layer, 1 [mm]

VP
Surface material according to designer specification
Reinforced concrete slab according to construction drawing
Load-bearing profiled sheet mould according to construction drawing reinforcements according to construction drawing

Windows (U): 0.7 W/(m ² K)
Door (U): 0.7 W/(m ² K)

K.OSA 1	KORTTELI/TILA 116	TONTTI/RNo 4	RAKENNUSLUVAN TUNNUS				
RAKENNUSTOIMENPIDE NEW BUILDING			PIIRUSTUSLAJI ARCHITECTURAL DRAWING				JUOKS.No 3
RAKENNUSKOHTEEN NIMI JA OSOITE TWO APARTMENT HOUSE Kolari			PIIRUSTUKSEN SISÄLTÖ Section layout				MITTAKAAVA 1:100
LEDION SHQEFNI			SUUNN.ALA SECTION DRAWING	TYÖ No	PIIR.No		MUUTOS
			PÄIVÄYS 11.05.2022	YHT.HENK.	TIEDOSTO		PIIRTÄJÄ