# DESIGN OF A TWO APARTMENT HOUSE

Bachelor's thesis

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Abstract
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The purpose of this Bachelor's thesis is to demonstrate how to design and perform a structural analysis of a two apartment house. This thesis also aimed at providing a manual for designing a timber structure and calculating the stability of structural elements based on the principal loads that affect the structural integrity of these elements and the structure as a whole.

The case study timber building consists of two main systems. In one direction, the building is presented as a one-story apartment, with a basement that will be used as a living space, while the other direction is a two-story apartment, where the basement has in principle its main function, but not as a living space. This thesis will address the second design of the building, where the practical part involves the design of joists, beams, columns, and problem solving encountered in the project. Further, the practical part corresponds to the use of software such as AutoCAD, Mathcad, and RFEM.

As a result, this thesis will serve as an evaluation process of the overall covered processes, where at the end, a document with all the calculations will be attached in the form of an appendix.

Keywords Structural integrity, capacity, loads.
Pages 59 pages and appendices 82 pages

Contents

Appendices

# 1 Introduction

Structural design involves selecting the dimensions of structural members and modeling the support structure in respect to material resistance, performance, and durability requirements during the useful life of the structure. It is based on verification, and the purpose of this verification is to show that the actual requirements for the selected materials, dimensions, and construction system are met. With the implementation of Eurocodes, changes in the design criteria and the calculation of the structural capacity of the facilities are brought up.

## 1.1 Objectives

The purpose of this thesis is to compile information from Eurocodes and design guidelines that are essentially related to the design of structural components in buildings and to develop useful and consistent sizing guidelines for such structural components. The goal is to start with the boundary conditions, load calculations, combinations, and proceed with the calculations consistently. The aim is to give the reader a clear view of the practice calculation routines while the theory is going through and make it easier to find and obtain additional information based on the source references. Further, this thesis tries to present the structural elements and floor plan layouts to create an overall vision of the building.

## 1.2 Research questions

The main analysis is related to the question "How can the structural applications of the specific elements treated to be used and developed in a similar project?" It is here divided into separate questions, which are essentially related to each other:

- Which concepts of timber limit state are essential for similar projects?
- What are the stages and processes to be followed to achieve structural integrity?

## 1.3 Scope and limitations

The scope of this thesis consists of three development sections. The first section is the input that presents information about the object, structural materials, and information on the software used. The second section is more about concepts for the limit state design of timber and the forces applied to the structure. The third stage interprets the steps to be followed on the various decisions taken along with the addressing solutions to problems encountered. The connection of the steel rod with the beam is not presented in the thesis, as it is not included within the scope. Analyzing this connection would require more in-depth research on steel structures.

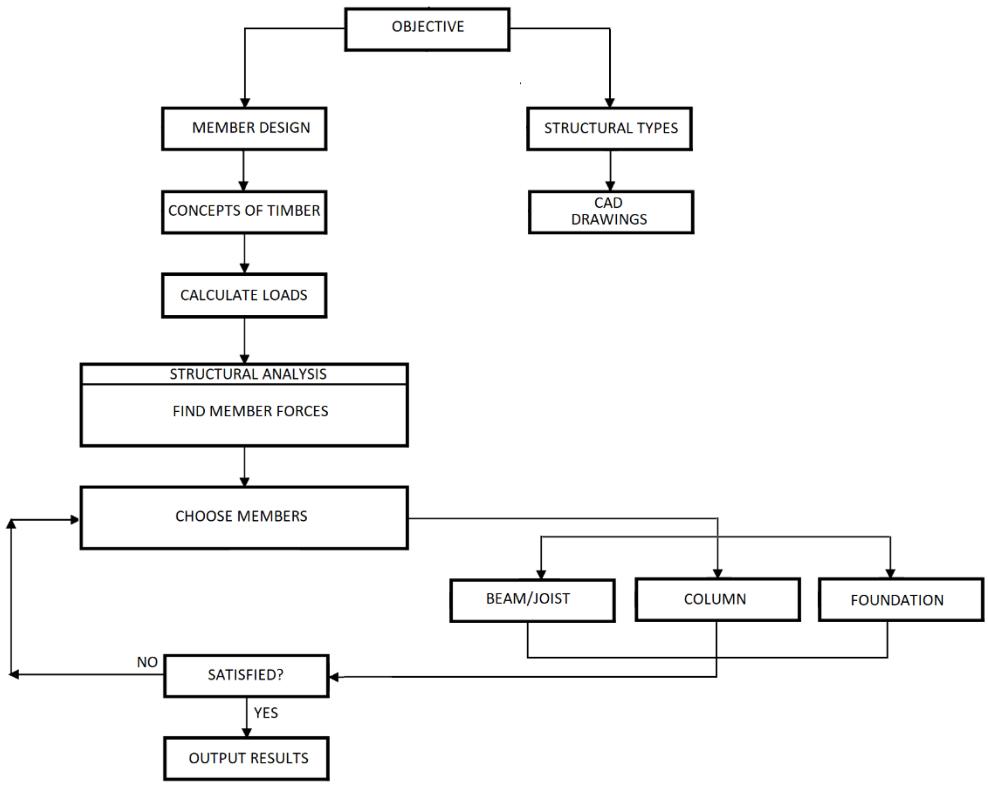
# 2 Description of the project

This thesis involves the analysis of a project which is still in the design phase. A thorough background study was performed to understand the wood design process and how parametric workflows can be useful in similar projects.

## 2.1 Structural design

Structural design is the methodical investigation of the stability, strength and rigidity of structures. The basic objective in structural analysis and design is to produce a structure capable of resisting all applied loads without failure during its intended life. The primary purpose of a structure is to transmit or support loads. If the structure is improperly designed or fabricated, or if the actual applied loads exceed the design specifications, the device will probably fail to perform its intended function, with possible serious consequences. (Lang, 2016) To minimize the possibility of structure failure, the main steps followed during the design phase are presented (Figure 1).

Figure 1 Flow-chart of the design process



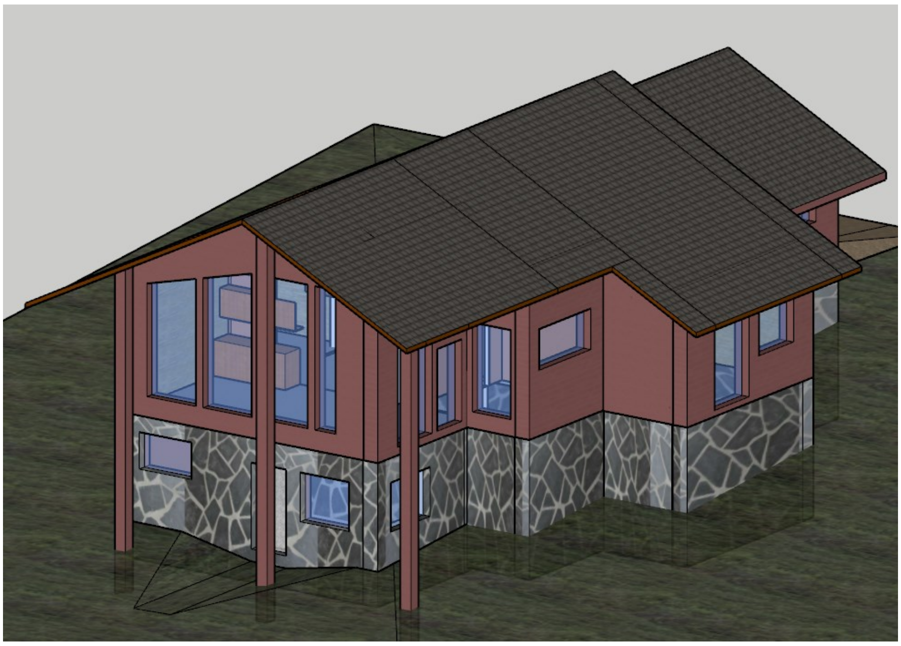
The design project consisted of the following steps:

- Presentation of structural types and their drawing using CAD software.
- Introduction to timber concepts.
- Definition of loads applied to the structure.
- Selection of elements based on applied forces.

## 2.2 General information about the structure

Referring to the information presented in the introduction section, this two-apartment house is a concept that is based on the calculations presented in the appendices. The location of the building is Louhikkotie, with a living area of 250m$^2$. The direction addressed in this thesis represents the object as a two-story building, with a basement where its function is to be as storage (Figure 2).

Figure 2 3D model of the building



The usage of timber as a primary construction material for projects of this scale is extremely prevalent in Finland. For this building, the material used to design the structural elements is softwood timber which belongs to class C24. The only timber material that does not belong to the softwood category is the joist. They belong to the category of Laminated Veneer

Lumber. The characteristic value of bending strength for both of these timber classes is 24MPa. The concrete used for foundation design belongs to class C25/30. The characteristic compressive strength of this concrete class is 25MPa.

Since the building is still in the design phase, decisions on the location of columns and beams are made based on achieving structural integrity. Follow appendix 12 for floor layout plans.

## 2.3 Methodology

This chapter explains the process of this research. Section 2.3.1 presents the source and how the data for this project is collected, in terms of structural calculations. Section 2.3.2 presents the software used in the project as well as their role to obtain the final results. The design of the first option which includes studs and shear walls is covered by Ledion Shqefni. (Shqefni, 2022) Calculations for snow load, wind load, and strip foundation are common.

### 2.3.1 Codes and standards

Structural calculations are based on the following standards:

- Basis of structural design EUROCODE 0 (SFS EN 1990 + Finnish NA)
- Loadings EUROCODE 1 (SFS EN 1991 + Finnish NA)
- Timber Structures EUROCODE 5 (SFS EN 1995 + Finnish NA)
- Foundation Structure EUROCODE 7 (SFS EN1997 + Finnish NA)

### 2.3.2 Software used during the project

All software used during the project is provided by HAMK University of Applied sciences.

Mathcad

Mathcad is a software that allows performing mathematical calculations in a format similar to how it would be done for manual hand calculations. It maintains a typical registration form, special mathematical and trigonometric functions, series functions, and matrix algebra. The flexibility that the application offers regarding variables and SI units can be used to answer almost any engineering problem. With a well-structured worksheet, design calculations can be performed to allow parameters to be changed and results to be viewed in a matter of seconds. Referring to the information mentioned above, the main use relates to the writing of equations and the calculation of forces such as dead, live, snow, and wind loads. Additionally, through the software, it is possible to analyze the elements in terms of structural stability.

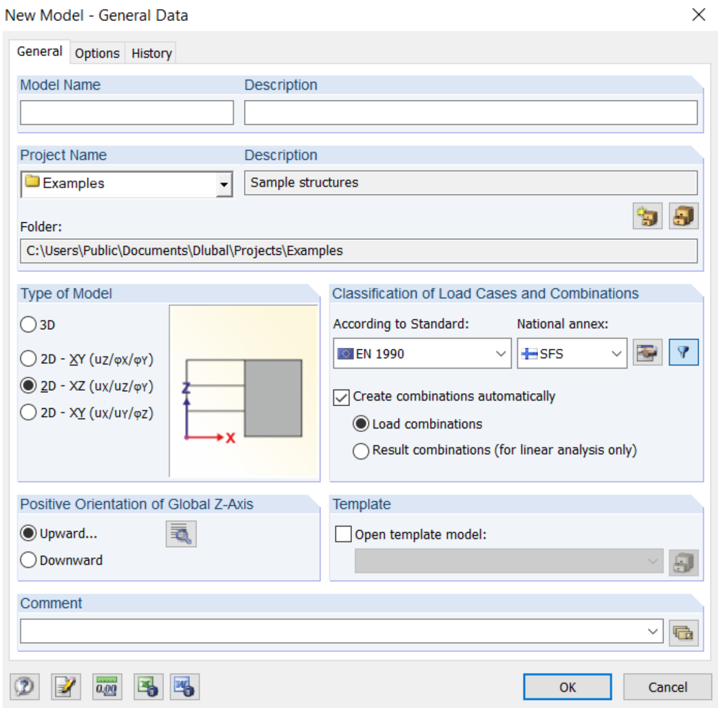
AutoCAD

In recent years, generation has stepped forward the lives and studies regions of humans in all disciplines. One of the biggest developments in the field of engineering has to do with computer-aided drafting. AutoCAD is CAD software that has as its primary function the introduction and generation of 2D and 3D drawings. Through AutoCAD is possible to modify geometric patterns, through various options, to develop all kinds of structures and objects. Through such options it is very easy to make changes, and create multiple drafts, so that the problems encountered are constantly adjusted, to get the desired final product. The main purpose of using this software is to generate cross-section drawings of the structural elements.

Dlubal RFEM

Dlubal RFEM is a finite 3D element analysis system used for structural analysis and construction of various materials from metal, wood, and concrete to complex and composite materials. The system offers a deep integration from BIM to various CAD software and various connections to all building models. It is a flexible and reliable F.E.M system for developers to build and manage all data processes throughout the project. RFEM calculates deformations, internal forces, pressures, and support forces, and also offers an option on which it generates load combinations. Considering the partial safety factors, the load combination combines the load of the included load cases into a large load case, which is then calculated. For this project, Dlubal RFEM is used to analyze statically indeterminate structures, specifically continuous beams, and joists, that cannot be analyzed using statics, or equations of equilibrium only. The connections were modeled as pinned, and the loads calculated with Mathcad were applied to the elements to check the maximum bending moment and support reactions. Classification of load cases and combinations are followed according to EN1990 and the Finnish National Annex (Figure 3).

Figure 3 Dlubal RFEM Version 5.26




## ComSlab

ComSlab is software designed as a tool for composite panels, through which design verification is made possible. Depending on the use and loads that will be applied to the element, the software determines the percentage of risk for different verifications according to the relevant annex. Slab design is done using the ComSlab software provided by Ruukki.

# 3 Structural types of a single-family house

A structural drawing is a plan or set of plans and descriptions that aims to present information about how a structure will be constructed. Structural drawings are generally prepared by registered professional engineers, and based on information provided by architectural drawings. Structural drawings are mainly concerned with the load-bearing members of a structure. They outline the sizes and types of materials used as well as general demands for connections. For this project, the structural drawings are made possible according to the client's request. Follow Appendix 11 for drawings of structures and their connections.

## 3.1 Foundation

Foundation is the structural component of a building that makes sure of its connection with the soil. The main purpose of the foundation in the field of engineering is the transfer of loads from the structure to the ground. Through this structure, it is possible to support the main structure which appears above ground level. In this building, two different types of foundations are presented, where the first has the function of supporting the wall structure, while the second aims to support the columns.

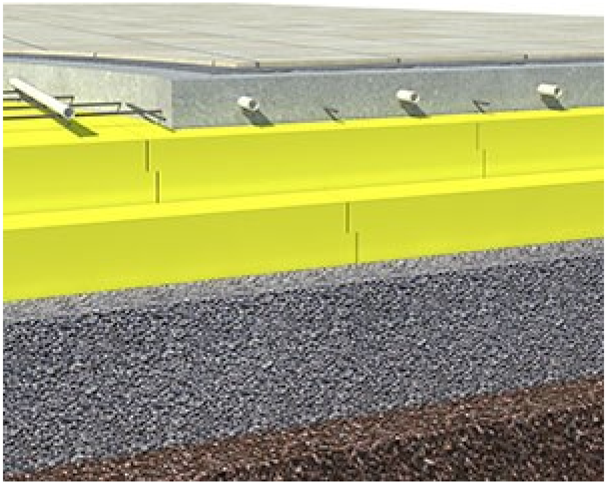
## 3.2 Floor structures

The base floor is a structure, supported by the building foundation directly, that divides space horizontally into stories. In structural terms, for a base floor to be called functional, the assembly must support all loads that are expected to be transferred to this structure, where among them can be mentioned its dead load, plus furnishing and the live load of occupants, which must correspond with values set by the Eurocodes.

### 3.2.1 Floor against the ground

In terms of design, the strength of a structure is one of the most important elements in its functionality and durability. The base floor is a structure supported by the building foundation directly, therefore it must be suitable for the type of soil on which it will stand. To minimize all the risks encountered with a floor structure against the ground, Finnfoam presents a highly efficient design (Figure 4).

Figure 4 Finnfoam floor structure (Finnfoam, n.d.)



The relative humidity is present in very high percentages for structures that have direct contact with the soil, therefore thermal insulation must have enough water vapor resistance, to minimize indoor air issues. The base floor is 100 mm concrete with insulation underneath the slab, which makes the structure more resistant to defects such as water damage. To maintain comfort in terms of heating floor heating pipes will be installed inside the concrete.

### 3.2.2 Composite floor slab

For the slab design that rests on the basement wall, this project presents an innovative design. To build slabs, the traditional method requires the layout of wood elements to create the framework on which the concrete will be poured. Ruukki presents an adaptation of this way using composite load-bearing sheets. These metal sheets are easy to assemble and made of high-quality raw materials. Unlike the traditional slab method with the framework, the installation process is extremely fast making it possible to continue other construction processes. For this slab composite sheet, CS48-36 is used with a length of 3.5 meters (Figure 5).

Figure 5 Composite sheet CS48-36-750 (Ruukki, n.d.)

The concrete strength class is C25/30 and the exposure class belongs to category XC3. Dead load is presented as the weight of the slab itself together with the weight of the chimney as a concentrated load, specifically 12kN. The imposed load is presented by the loads that move through or act on the element in accordance with the National Annex. Once the necessary data is implemented, the software can do all the calculations and detect problems with the design if there are any. The report generated by the software is presented in the Appendix 10.

Advantages of this type of slab include the following:

- Versatility
- Quick installation
- Cost reduction
- Resistance/Weight ratio

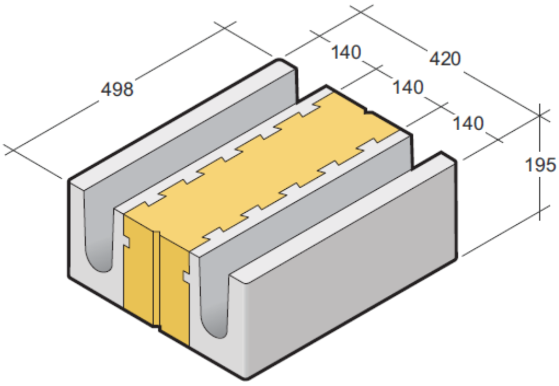
## 3.3 Wall structures

The wall is a structural element in the building that has as its primary function the closure or division of rooms. It defines an area that provides safety and shelter. For this object, the walls are classified into the categories of load-bearing and non-load-bearing.

### 3.3.1 Basement wall

Basement walls and their design differs from the other structural walls. Since these walls are subjected to vertical load from the structure above and the lateral loads from the soil, the design should be carried in a way that they can resist these loads. The construction of these walls is done using Leca blocks (Figure 6). Due to its low U-Value and the ability to handle moisture, the Leca wall structure can be a very sustainable solution to the traditional wooden wall structure.

Figure 6 Leca 420 block (Leca, n.d.)




### 3.3.2 Exterior wooden wall

To achieve energy efficiency goals the wall must be designed in such a way that it is in harmony with all other structural elements. For this object, the primary wall structure will consist of a timber frame otherwise known as ventilated façades. Ventilated façade is a general term for structures that have a uniform ventilation gap between the cladding and the insulating layer which provides improved moisture safety for the wall structure. In ventilated structures, the cladding façade is connected to the load-bearing structure through a layer of insulation. This ventilation space is in contact with the outside air, where through the lower opening of the façade cladding and the outlet openings at the top but as well as additional floor-specific ventilation openings connected to the window and door opening for example. In the ventilation gap, there is gravity ventilation, where the air moving through this ventilation space removes excess moisture accumulated in the ventilation gap (Figure 7).

Figure 7 Wall structure (Paroc, n.d.)


Ventilated facades can be designed for new buildings or for repairing facades in older buildings. It is necessary to check the national building codes to enable the best performance of the wall structure according to the characteristics of load-bearing structures, building envelope, thermal properties, fire protection, and sound insulation.

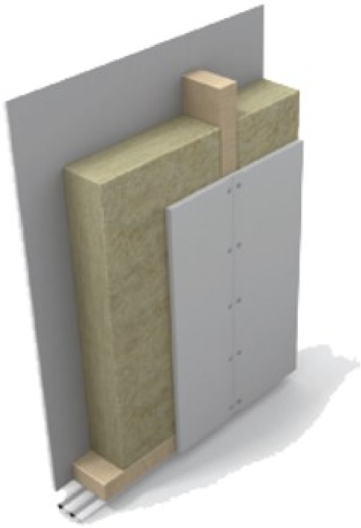
Advantages of this type of wall include the following:

- Long-lasting moisture protection.
- The selection of insulation materials makes it possible to achieve the desired U-value.
- Adjustable façade system through which damage repair is possible.
- Low maintenance costs and recyclability.
- Fire protection is achieved through non-combustible elements.

### 3.3.3 Partition walls

By definition, partition walls are designed as non-load-bearing interior elements, where the only load to be carried is the material of the wall itself. Partition walls in this building are presented as non-load-bearing where the main purpose is to separate the different spaces and uses of the building from each other. The wall structure is made of wooden studs, and two layers of timber panels, together with low-density stone wool to meet the resistance requirements of sound and fire insulation (Figure 8).

Figure 8 Partition wall (Paroc, n.d.)

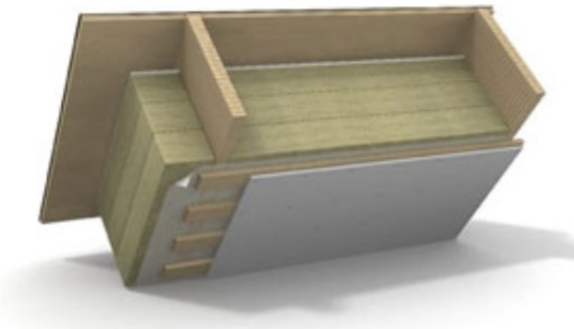


## 3.4 Roof structure

The roof structure is a term that refers to the construction at the top of a building which typically provides protection from the elements. It generally comprises a system of structural members designed to support the roof build-up materials that provide water tightness and thermal and acoustic insulation to the building below. (Designing Building, 2020)

Since the building is analyzed in two different directions, the selection of a suitable roof structure is a key element in this project. The term used for the roof structure in this thesis is joisted roof, a steeply pitched design applied to both residential and commercial buildings (Figure 9).

Figure 9 Roof structure (Paroc, n.d.)



Beyond the fact that sloping roofs have always been a massive choice in terms of their design, including the practicality and elegance they have, it must not be forgotten that this structure is as effective as possible in its role. It is necessary to check the national building codes to enable the best performance of such elements.

Advantages of this roof structure include the following:

- Low maintenance costs due to less frequent repairs.
- Stable and efficient when dealing with rain and snowfall.
- Versatility as sloping roofs can be turned into a living space.
- Sustainability in terms of the installation and maintenance of mechanical systems such as solar and photovoltaic.

# 4 Concepts used for the limit state design of timber

As a building material, timber differs from steel, reinforced concrete, or other composites in several ways. Wood is a biological and natural material with highly variable properties. It is furthermore, hygroscopic, which means that the moisture content in the material constantly changes with the relative humidity of the surrounding environment. When it comes to timber, however, the designer chooses a grade that has been verified by some type of non-destructive strength grading of sawn timber. (Borgström, 2016)

When designing timber structures these characteristic conditions of strength graded wood and the variable traits of timber must always be considered. Some of the characteristics are as follows:

– Duration of load for different types of applied loads.
– Partial factors and modification factors depending on the material properties.
– Different moisture content depending on the service class.

## 4.1 Load duration cases

As a structural element, timber experiences a significant loss of integrity over time. To account for the loss of resistance, load time classes have been established to facilitate the design process. They cover a wide range of possible durations occurring in reality. Actions shall be assigned to one of the load-duration classes given in table 1.

Table 1 Load duration classes (SFS-EN 1995-1-1: 2004)

| **Load-duration class** | **Order of accumulated duration of characteristic load** |
| --- | --- |
| Permanent | more than 10 years |
| Long-term | 6 months – 10 years |
| Medium-term | 1 week – 6 months |
| Short-term | less than one week |
| Instantaneous | |

The impact that the load duration has on the wood capacity is taken into account by assigning a $k_{mod}$ factor, which comes as a function of service class. $K_{mod}$ is presented as a reduction factor for the characteristic strength of times, varying between 0.2 and 1.1. The recommended values for $k_{mod}$ can be selected from the table below.

Table 2 Values of $k_{mod}$ (SFS-EN 1995-1-1: 2004)

| Material | Standard | Service class | Load-duration class | | | | |
|---|---|---|---|---|---|---|---|
| | | | Permanent action | Long term action | Medium term action | Short term action | Instantaneous action |
| Solid timber | EN 14081-1 | 1 | 0,60 | 0,70 | 0,80 | 0,90 | 1,10 |
| | | 2 | 0,60 | 0,70 | 0,80 | 0,90 | 1,10 |
| | | 3 | 0,50 | 0,55 | 0,65 | 0,70 | 0,90 |
| Glued laminated timber | EN 14080 | 1 | 0,60 | 0,70 | 0,80 | 0,90 | 1,10 |
| | | 2 | 0,60 | 0,70 | 0,80 | 0,90 | 1,10 |
| | | 3 | 0,50 | 0,55 | 0,65 | 0,70 | 0,90 |
| LVL | EN 14374, EN 14279 | 1 | 0,60 | 0,70 | 0,80 | 0,90 | 1,10 |
| | | 2 | 0,60 | 0,70 | 0,80 | 0,90 | 1,10 |
| | | 3 | 0,50 | 0,55 | 0,65 | 0,70 | 0,90 |
| Plywood | EN 636 | | | | | | |
| | Part 1, Part 2, Part 3 | 1 | 0,60 | 0,70 | 0,80 | 0,90 | 1,10 |
| | Part 2, Part 3 | 2 | 0,60 | 0,70 | 0,80 | 0,90 | 1,10 |
| | Part 3 | 3 | 0,50 | 0,55 | 0,65 | 0,70 | 0,90 |
| OSB | EN 300 | | | | | | |
| | OSB/2 | 1 | 0,30 | 0,45 | 0,65 | 0,85 | 1,10 |
| | OSB/3, OSB/4 | 1 | 0,40 | 0,50 | 0,70 | 0,90 | 1,10 |
| | OSB/3, OSB/4 | 2 | 0,30 | 0,40 | 0,55 | 0,70 | 0,90 |
| Particle-board | EN 312 | | | | | | |
| | Part 4, Part 5 | 1 | 0,30 | 0,45 | 0,65 | 0,85 | 1,10 |
| | Part 5 | 2 | 0,20 | 0,30 | 0,45 | 0,60 | 0,80 |
| | Part 6, Part 7 | 1 | 0,40 | 0,50 | 0,70 | 0,90 | 1,10 |
| | Part 7 | 2 | 0,30 | 0,40 | 0,55 | 0,70 | 0,90 |
| Fibreboard, hard | EN 622-2 | | | | | | |
| | HB.LA, HB.HLA 1 or 2 | 1 | 0,30 | 0,45 | 0,65 | 0,85 | 1,10 |
| | HB.HLA1 or 2 | 2 | 0,20 | 0,30 | 0,45 | 0,60 | 0,80 |
| Fibreboard, medium | EN 622-3 | | | | | | |
| | MBH.LA1 or 2 | 1 | 0,20 | 0,40 | 0,60 | 0,80 | 1,10 |
| | MBH.HLS1 or 2 | 1 | 0,20 | 0,40 | 0,60 | 0,80 | 1,10 |
| | MBH.HLS1 or 2 | 2 | – | – | – | 0,45 | 0,80 |
| Fibreboard, MDF | EN 622-5 | | | | | | |
| | MDF.LA, MDF.HLS | 1 | 0,20 | 0,40 | 0,60 | 0,80 | 1,10 |
| | MDF.HLS | 2 | – | – | – | 0,45 | 0,80 |

Following the above statement, the values of $k_{mod}$ are dependent on the load duration, and its selection depends on the shortest load duration. For this case study, both snow and live load are presented as a medium-term action, where at the same time they have the shortest presence in the building. Based on this data the value for the modification factor is to be taken as 0.8.

## 4.2 Effect of moisture content and service classes

Moisture content and variations of moisture have a very important role in affecting the properties of woof-based elements. It affects both stiffness and strength of the material, therefore in considering such effects, three service classes have been defined in EN 1995-1-1 page 22, to incorporate during the design phase. Service classes are presented as follows:

- Service class 1, where the average moisture content does not exceed 12 %.
- Service class 2, where the average moisture content does not exceed 20 %.
- Service class 3, where the average moisture content exceeds 20 %.

The material used for the building belongs to service class 1.

## 4.3 Partial and modification factors for material properties

To account for the inverse effect of the material's geometric deviations and the uncertainty of the resistance model used in the design, the design strength value is divided by the partial factor $\gamma_M$ to obtain the design strength of the material. Values for the partial factor $\gamma_M$ varies between 1,2 to 1,3 for most wood-based material when designing in the Ultimate Limit State. Table 3 provides information on value selection for partial factors.

Table 3 Recommended partial factors $\gamma_M$ for material properties (SFS-EN 1995-1-1: 2004)

| | |
|---|---|
| Fundamental combinations: | |
| Solid timber | 1,3 |
| Glued laminated timber | 1,25 |
| LVL, plywood, OSB, | 1,2 |
| Particleboards | 1,3 |
| Fibreboards, hard | 1,3 |
| Fibreboards, medium | 1,3 |
| Fibreboards, MDF | 1,3 |
| Fibreboards, soft | 1,3 |
| Connections | 1,3 |
| Punched metal plate fasteners | 1,25 |
| Accidental combinations | 1,0 |

Given that the type of material belongs to the class of softwood (solid timber), the recommended value for the partial factor is to be taken as 1.3

# 5 Loads

The structure of any building is exposed to various types of loads. Thus, every project should manage the vertical and horizontal loads that are mentioned below:

- Dead load. It includes the self-weight of the whole structure(load-bearing, structural, and non-structural members).
- Live load. Includes the load from occupants and furniture.
- Lateral load. Includes the wind load to this project.
- Snow load.

The load types are described in more detail in the next sections.

## 5.1 Dead Load

Dead load is a structural load of constant magnitude associated with the weight of the structure with respect to time. The weight of any structural elements, including loads of fixtures, attached permanently to the structure is also considered a dead load. Calculation of the dead load with a good degree of accuracy is determined by assessing the weight and the volume of the object. Table 4 presents two of the main dead loads of the structure, in addition to the weight of the specific elements. Both of these loads have the same symbol as they refer to two different structures.

Table 4 Dead load acting on the structure

| Symbol | Description | Value | Unit |
|---|---|---|---|
| $G_{k.1}$ | Dead load on the roof structure | 0.53 | kN/m² |
| $G_{k.1}$ | Dead load on the second floor | 0.45 | kN/m² |

## 5.2 Live load

Live loads refer to the transient forces that move through a building or act on any of its structural elements. They include the possible or expected weight of people, furniture, appliances, cars and other vehicles, and equipment. Because these dynamic loads are variable and often inconsistently applied to a structure, engineers must plan for a maximum imposed load that is likely much more extreme than what a building will experience over the course of its lifetime. (Mtcopeland, 2020)

To determine the imposed loads, the floor and roof areas in buildings should be sub-divided into categories according to their use. Table 5 contains information on imposed loads on the floor.

Table 5 Imposed loads on floors, balconies, and stairs in buildings, uniformly distributed load qk and concentrated load Qk. (Finnish National Annex 2)

| Categories of loaded areas | $q_k$ [kN/m$^2$] | $Q_k$ [kN] |
|---|---|---|
| **Category A** | | |
| – Floors | 2,0 | 2,0 |
| – Stairs | 2,0 | 2,0 |
| – Balconies | 2,5 | 2,0 |
| **Category B** | 2,5 | 2,0 |
| **Category C** | | |
| – C1 | 2,5 | 3,0 |
| – C2 | 3,0 | 3,0 |
| – C3 | 4,0 | 4,0 |
| – C4 | 5,0 | 4,0 |
| – C5 | 6,0 | 4,0 |
| **Category D** | | |
| – D1 | 4,0 | 4,0 |
| – D2 | 5,0 | 7,0 |

According to Finnish National Annex 2, the characteristic imposed load for Category A (Areas for domestic and residential activities) is 2kN/m². It is recommended that the values for imposed loads be taken from the national annex of the respective country where the structure is being studied.

## 5.3 Snow Load

The snow load on the roof depends on the importance of the building, roof slope, exposure to wind, and location of the building. To determine snow load the following expression is used:

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

where:

μi is the snow load shape coefficient.

Ce is the exposure coefficient. The value of Ce is dependen on the topography. (Table 6)

Ct is the thermal coefficient. Recommended value by Eurocode is 1.

Sk is the characteristic value of snow load on tile ground

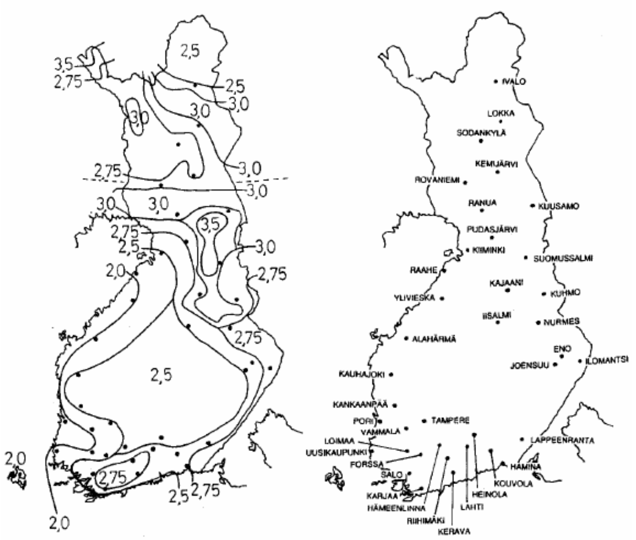
Table 6 - Recommended values of Ce for different topographies (EN 1991-1-3: 2003)

| Topography | $C_e$ |
|---|---|
| Windswept[a] | 0,8 |
| Normal[b] | 1,0 |
| Sheltered[c] | 1,2 |
| [a] *Windswept topography*: flat unobstructed areas exposed on all sides without, or little shelter afforded by terrain, higher construction works or trees. [b] *Normal topography*: areas where there is no significant removal of snow by wind on construction work, because of terrain, other construction works or trees. [c] *Sheltered topography*: areas in which the construction work being considered is considerably lower than the surrounding terrain or surrounded by high trees and/or surrounded by higher construction works. | |

As the location of the building is considered a normal topography, the value of Ce is recommended to be taken as 1.

According to Finnish National Annex 4 and Eurocode 1 the characteristic value of snow load on the ground in the Lapland area, Sk is 3 kN/m2. Figure 10 provides a map of snow loads in Finland.

Figure 10 Snow loads on the ground in Finland (Finnish National Annex 4)



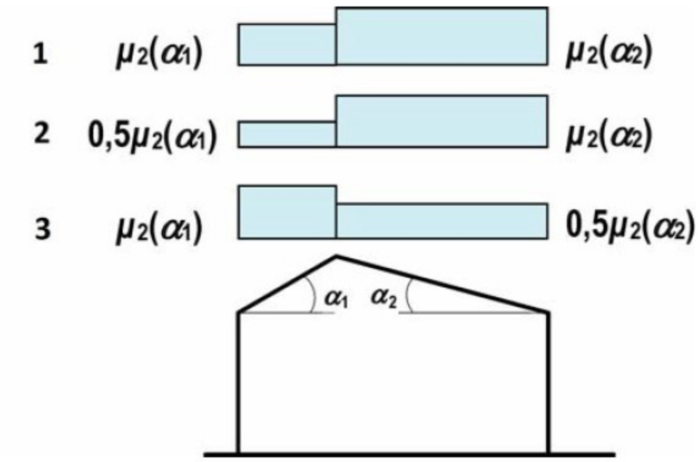
The conditions based on the angle of the roof to determine the snow load shape coefficients are shown in Table 7.

Table 7 - Snow load shape coefficients (SFS-EN 1991-1-3:2003)

| Angle of pitch of roof $\alpha$ | $0° \leq \alpha \leq 30°$ | $30° < \alpha < 60°$ | $\alpha \geq 60°$ |
|---|---|---|---|
| $\mu_1(\alpha)$ | $\mu_1(0°) \geq 0,8$ | $\mu_1(0°)\frac{(60° - \alpha)}{30°}$ | 0,0 |
| $\mu_2(\alpha)$ | 0,8 | $0,8\frac{(60° - \alpha)}{30°}$ | 0,0 |
| $\mu_3(\alpha)$ | $0,8 + 0,8\ \alpha/30°$ | 1,6 | -- |

The roof slope is 1:2 therefore roof angle is smaller than 30°. The value of $\mu_1$ recommended by Eurocode and the Finnish National Annex is 0.8. Since the building has a pitched roof, the load arrangement shown in Figure 11 should be used for the calculation.

Figure 11 Snow load shape coefficients (SFS-EN 1991-1-3:2003)




Considering that in the design phase, the loading scenario is the maximum possible factored loading, which represents the worst case, the same procedure is followed regarding the distribution of snow load on the roof. The results of the snow load on the roof are presented in Appendix 1.

## 5.4 Wind Load

Wind pressure on buildings is based on EN 1991-1-4 along with Finnish National Annex. To calculate the basic wind load and pressure several steps must be followed. This section presents the steps to be followed to determine the wind load pressure, while for the results we refer to the thesis of Ledion Shqefni (Shqefni, 2022).

### 5.4.1 Basic wind velocity

To calculate basic wind velocity the following expression should be used:

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

Where:

$V_b$ is the fundamental value of the basic wind velocity (m/s).
$C_{dir}$ is the direction factor, recommended value is to be taken as 1.0
$C_{season}$ is the season factor, recommended value is to be taken as 1.0

Following the relevant data and location in the building, Lapland, the basic wind velocity obtained $V_b$ is 22 (m/s).

### 5.4.2 Mean wind velocity

The mean wind velocity at height z above the terrain depends on the basic wind velocity and roughness and orography of the terrain. It can be calculated from the following equation:

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

Where:

$C_r$ is the roughness factor of the ground roughness of the terrain upwind of the structure in the wind direction considered.
$C_o$ is terrain orography factor.

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

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

Where:

Z is the height of the structure above ground level (m). Following this case, the height of the building will be considered from the base floor to the highest point of the building.

$Z_0$ is the roughness height (m)

$k_r$ is the terrain factor depending on the roughness length $Z_0$

Terrain category III is considered for the project mentioning that the building is isolated by obstacles and surrounded by a forest (Table 8).

Table 8 Terrain categories and terrain parameters (SFS-EN 1991-1-4)

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

### 5.4.3 Wind turbulence

The turbulence intensity at a reference height is presented as the ratio between the turbulence and mean wind velocity. Wind turbulence can be calculated using the following equation:

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

Where:

$K_I$ is the turbulence factor. The recommended value is to be taken as 1.
$c_0$ is the orography factor.
$z_0$ is the roughness height.

### 5.4.4 Peak velocity pressure

The peak velocity pressure $q_p(z_e)$ at reference height $z_e$ includes mean and short-term velocity fluctuations. (SFS-EN 1991-1-4) This factor is given by the following equation:

$$q_p = \left[1 + 7 \cdot I_v\right] \cdot \frac{1}{2} \cdot \rho \cdot {v_m}^2$$

Where:

$\rho$ is the density of air. The recommended value for air density is 1.25kg/m3.
$v_m$ is the basic wind velocity.
$I_v$ is the wind turbulence.

### 5.4.5 Wind pressure on surfaces

The wind pressure acting on the external surfaces should be obtained from the following equation:

$$W = q_p \left(Ze\right) \cdot C_{pe}$$

Where:

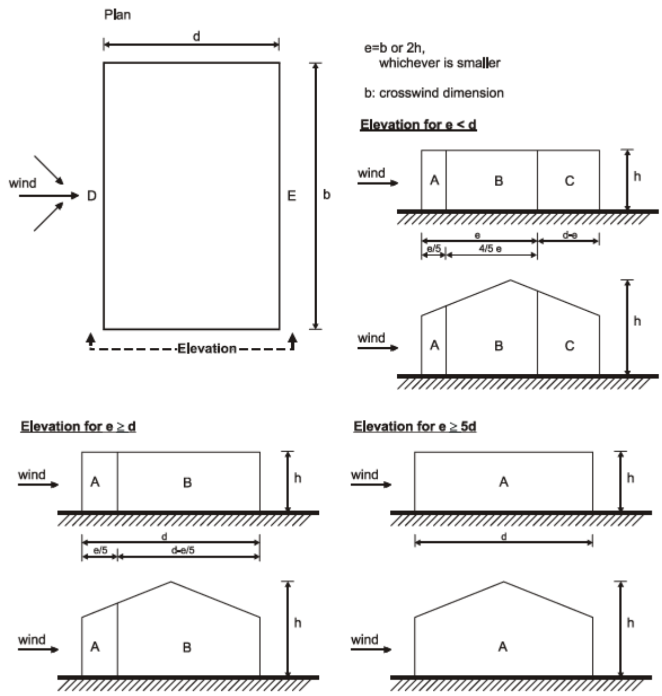
$q_p$ is the peak velocity pressure.

Ze is the reference height for the external pressure.

$C_{pe}$ is the pressure coefficient for the external pressure.

The selection of building zones is one of the main points in determining wind pressure. This is because the external pressure coefficients for certain parts of the building are dependent on these areas. Zones of wind direction and action for the external wind pressure are shown in Figure 12 .

Figure 12 Key for vertical walls (SFS-EN 1991-1-4)



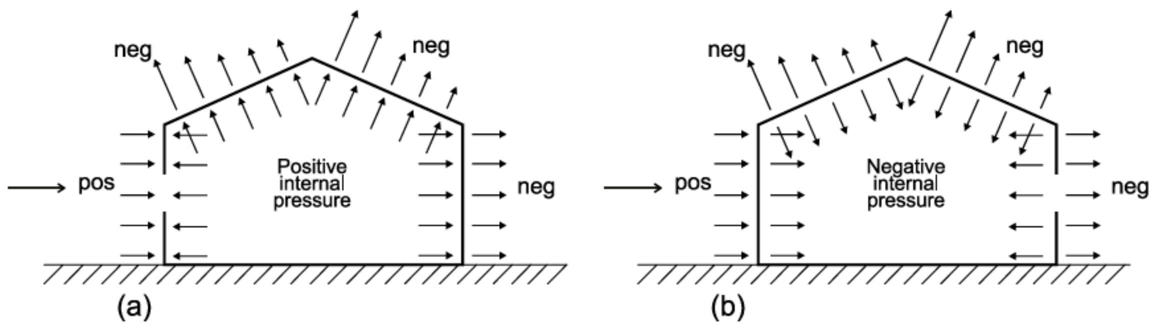

Note that EN 1991-1-4 presents these values for rectangular-plan buildings, while the object under study is presented with a more complicated shape rather than a rectangle. To make an adaptation to the Eurodoce, in both directions the longer spans will represent d and b. B represents the breadth of the building (perpendicular to wind), while D depth of the building (parallel to wind). The external pressure coefficients are given for loaded areas smaller than $1m^2$ and $10m^2$, where Cpe,1 represents areas that are under or equal to $1m^2$, while Cpe,10 represents loaded areas bigger than $10m^2$. The value of Cpe is dependent on the ratio between h/d where h represents the height of the building from the base to the roof, and d represents the depth of the building. Table 9 provides better visualization of the external pressure.

Table 9 - Recommended values of external pressure coefficients for vertical walls of rectangular plan buildings. (SFS EN1991-1-4)

| 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 | |

The wind stress on exclusive surface areas of the building varies to a big quantity relying on numerous factors. Places wherein airflow is disrupted are exposed to better pressure, consisting of the corners and the overhangs of the roof, therefore, when designing for complicated structures, changes in the wind pressure surface must be taken into account (Figure 13). When the value of Cpe is presented as negative, the positive value of internal pressure must be subtracted from the value of Cpe. In case Cpe is presented with a positive value, internal pressure with a negative value shall be deducted from Cpe. The value of internal pressure is considered more onerous at +0.2 and -0.3.

Figure 13 Pressure on surfaces (SFS EN1991-1-4)




By multiplying the peak velocity pressure with the final pressure coefficients, the wind pressure is presented as a force in kN/m$^2$.

## 5.5 Wind pressure simplified method

The application of the above method according to EN1991-1-4 is often presented as complicated as the number of factors to be considered is very high. This subchapter presents another simplified method for determining wind pressure, through adaptations that can be made between EN1991-1-4 and RIL 201-1-2017. In terms of design, this method used is more conservative and the results are to some extent larger than in the above-explained method. The wind force acting on a structure or a structural component may be determined directly by using the following expression:

$$F_w = c_f \cdot c_s \cdot c_d \cdot q_p$$

Where:

$c_s$ is the non-simultaneous occurrence of peak wind pressure on the surface.
$c_d$ is vibration effect to the structure due to turbulence.
$c_f$ is the force coefficient of the structure or structural element.
$q_p$ is the peak velocity pressure.

The recommended value in EN1991-1-4 for buildings with a height lower than 15 meters is to be taken as 1. Considering that this two-apartment building has a height level smaller than 15 meters the structural factor is taken as 1.

The coefficient of force $C_f$ of the structural elements, where the wind blows normally on a side of the building is defined by the following expression:

$$C_f = C_{f.0} \cdot \psi_r \cdot \psi_\lambda$$

Where:

$C_{f0}$ is the force coefficient of rectangular sections with sharp corners.
$\Psi_r$ is the reduction factor for square sections with rounded corners.
$\Psi_\lambda$ is the end-effect factor for elements with free end flow. The recommended value is 1.

Force coefficient $C_{f0}$ can be read from table 10 if the slenderness ratio is smaller than 10. Under the condition that the height of the building is less than 15 meters, the slenderness ratio λ is presented as the ratio between 2 times the height over the width.

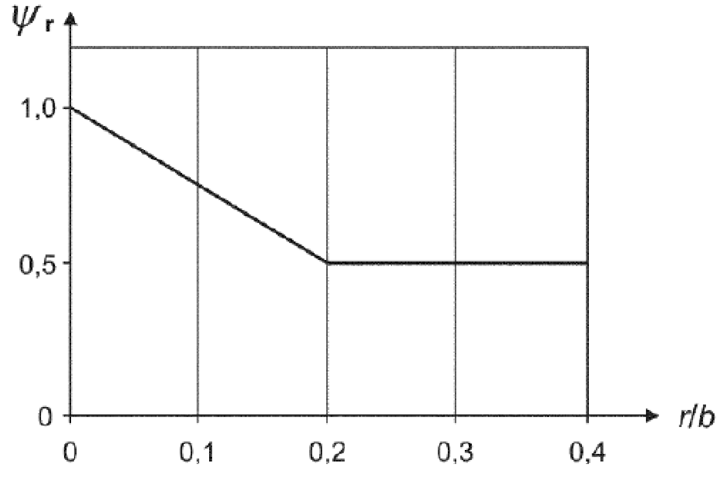
Table 10 - Strength factor $C_{f0}$ taking into account the effect of building dimensions and slenderness. (RIL 201-1-2017)

| | 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 | 0,60 | 0,54 | 0,54 |
| 3 | 1,29 | 1,29 | 1,48 | 1,55 | 1,38 | 1,07 | 0,65 | 0,58 | 0,58 |
| 10 | 1,40 | 1,40 | 1,60 | 1,68 | 1,49 | 1,15 | 0,70 | 0,63 | 0,63 |

The value of the force coefficient shall be determined using the interpolation ratio.

The reduction factor for square sections with rounded corners is dependent on the shape of the object. Figure 14 provides better visualization for the selection of this reduction factor.

Figure 14 Reduction factor Ψr for a square cross-section with rounded corners. (RIL 201-1-2017)



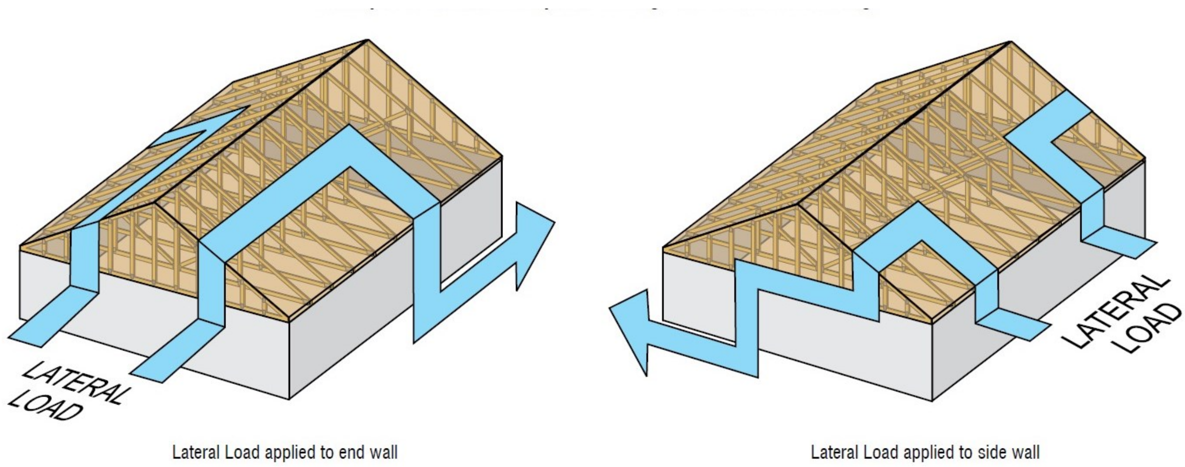

Since the building has no rounded but sharp corners, the value for the reduction factor is 1. The characteristic wind pressure in the x-direction is $0.474 kN/m^2$, while in the y-direction the characteristic wind pressure is $0.646 kN/m^2$. (Shqefni, 2022)

### 5.5.1 Wind load path

Typically in terms of structural behavior loads can be divided into vertical and horizontal loads. If a comparison is made, structural engineers find it more difficult to design the structure for horizontal loads than for vertical support mechanics. Horizontal loads often determine the efficiency of the system used in the building. In general, few mechanisms are commonly used alone or in combination to ensure lateral stability. For wooden structures, the most used system to resist lateral forces is the shear wall mechanism with a horizontal diaphragm, which is also present in this building. Apart from the vertical loads, the bearing walls function as shear walls to resist lateral force.

Load path plays a key role in the function of the lateral system, therefore the roof structure must be designed to be stiff enough to lateral loads and enable their transfer to the stiffening walls. Figure 15 provides better visualization of lateral load transfer through the roof system.

Figure 15 Load path through the roof of a building (McEntee, 2012)




Based on this connection lateral loads are transferred from the roof to the shear walls. It should be noted that based on this information for the calculation of columns, lateral load does not pose a risk, as it is transmitted from the roof and taken by the shear walls.

# 6 Load Combination

In structural design, different load combinations are encountered in structural codes. These combinations vary according to their occupancy, but selecting the right load combinations for the project is the main challenge of the designing phase. Selecting the right combinations without guidance is a tough task, while not using the right one, may result in a design or over design results. Usually, load combinations are categorized into working or service limit state (SLS) or ultimate limit state (ULS). But when to use the SLS or ULS should be clear for us. SLS combination is usually considered for deflection and static checks and ULS is for member design considerations. (The Structural World, 2018)

## 6.1 Ultimate Limit State

The ultimate limit state is defined as the state of design of a structural system, at which the ultimate collapse due to loading occurs. The loading scenario in the ultimate limit state is the maximum possible factored loading, which represents the worst-case scenario. Factored means that safety factors are applied to consider certain overloading of the structure and uncertainties of the material. Load combinations according to the Finnish National Annex, and EN 1990 are formed from the dominant load cases for structural analysis in fracture and service limit states. Loads are classified in the Eurocode as permanent loads G, variable loads Q, and accidental loads.

In the National Finnish Annex, the load combinations relevant to the structural design are defined using the following formulas:

$$1.35 \cdot K_{FI}$$

$$1.15 \cdot K_{FI} + 1.5 \cdot K_{FI} \cdot Q_{k.1} + K_{FI} \cdot \Sigma_I \cdot \Psi_{0.1} \cdot Q_{k.i}$$

Where:

$K_{FI}$ is the load factor for actions

$G_{kj,sup}$ is the permanent load characteristic (adverse effect).

$G_{kj,inf}$ is the permanent load characteristic (beneficial effect)

$Q_{k,1}$ is the first variable load.

$Q_{k,i}$ is the other variable load

$\Psi_{0,1}$ is the load combination factor.

In this project, the permanent load characteristic (adverse effect), represents the dead load. The first variable load represents live loads in the object, while the other variable represents snow. For the cause of reliability differentiation, the consequence class (CC) can be established by considering the results of failure or malfunction of the structure as given in table 11.

Table 11 - Definition of consequence classes (SFS EN1990)

| 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** | Grandstands, public buildings where consequences of failure are high (e.g. a concert hall) |
| CC2 | **Medium** consequence for loss of human life, economic, social or environmental consequences **considerable** | Residential and office buildings, public buildings where consequences of failure are medium (e.g. an office building) |
| CC1 | **Low** consequence for loss of human life, and economic, social or environmental consequences **small or negligible** | Agricultural buildings where people do not normally enter (e.g. storage buildings), greenhouses |

The object in question belongs to consequence class CC2 as fatalities in loss of life, and economic and environmental damage are considered average. The criterion for the classification of consequences is important in terms of consequences of failure. Load factor coefficients are depended on the definition of consequence class during the design phase, as particular elements are designed based on the defined coefficients. Recommended values for load factor selection are given in table 12.

Table 12 - KFI factor for actions (SFS EN1990)

| $K_{FI}$ factor for actions | Reliability class | | |
|---|---|---|---|
| | RC1 | RC2 | RC3 |
| $K_{FI}$ | 0,9 | 1,0 | 1,1 |

Based on consequence class load factor for actions is to be taken as 1.

Load combinations result when more than one load type operates on the structure. To ensure the safety of the structure, the building code generally refers to different load combinations which are associated with specific load factors for each load type. These load factors are combined to determine the required strength of the element being studied. Table 13 provides the recommended values for load combination factors.

Table 13 - Recommended values of Ψ factors for building. (SFS EN1990)

| Action | $\psi_0$ | $\psi_1$ | $\psi_2$ |
|---|---|---|---|
| Imposed loads in buildings, category (see EN 1991-1-1) | | | |
| Category A: domestic, residential areas | 0,7 | 0,5 | 0,3 |
| Category B: office areas | 0,7 | 0,5 | 0,3 |
| Category C: congregation areas | 0,7 | 0,7 | 0,6 |
| Category D: shopping areas | 0,7 | 0,7 | 0,6 |
| Category E: storage areas | 1,0 | 0,9 | 0,8 |
| Category F: traffic area, | | | |
| vehicle weight ≤ 30kN | 0,7 | 0,7 | 0,6 |
| Category G: traffic area, | 0,7 | 0,5 | 0,3 |
| 30 kN < vehicle weight ≤ 160 kN | 0 | 0 | 0 |
| Category H: roofs | | | |
| Snow loads on buildings (see EN 1991-1-3)*) | | | |
| Finland, Iceland, Norway, Sweden | 0,70 | 0,50 | 0,20 |
| Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l. | 0,70 | 0,50 | 0,20 |
| Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l. | 0,50 | 0,20 | 0 |
| Wind loads on buildings (see EN 1991-1-4) | 0,6 | 0,2 | 0 |
| Temperature (non-fire) in buildings (see EN 1991-1-5) | 0,6 | 0,5 | 0 |
| NOTE: The ψ values may be set by the National annex.<br>*) For countries not mentioned below, see relevant local conditions. | | | |

# 7 Stability Calculation

Structural designing is fundamental to the civil engineering discipline as it deals directly with the structural strength and integrity of buildings or structures. It includes planning, designing, and structural analysis of large structures, where the main principle is applying the principles of physical laws and mathematics to structures. With empirical research and analysis, civil engineers have to use structural design to analyze the structural design of the building and the strength of different materials used to determine its safety and economic specifications (Admin, 2019).

As presented in the introduction of the thesis, the second direction of the project has a second floor, that will be used as a living space. With the presence of the second floor, the forces that will be distributed on the retaining walls will be greater than their capacity, therefore the installation of beams and columns is a key element in achieving the structural integrity of the building.

## 7.1 Beam design

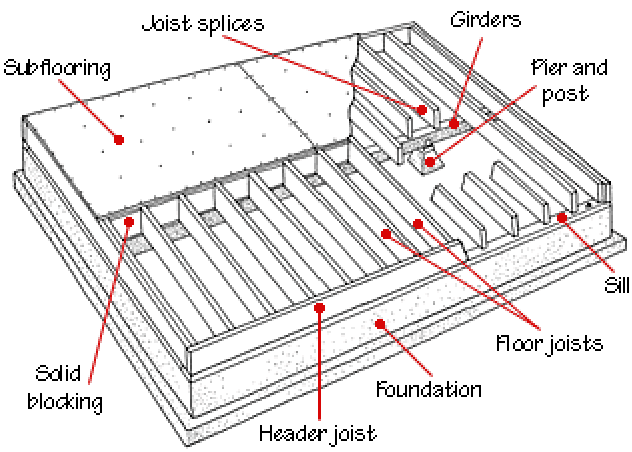
A beam structure, as simply referred with the term beam, is a type of structural elements used in construction and engineering to provide a safe and efficient load path that effectively distributes weight across the entire structure. The beam supports the load by resisting bending under the load pressure. Understanding the structure of a beam is essential in construction and structural design because these beams are the main way to transfer the weight of the building. They provide a stable load path to the building foundation so that the weight of the building's roof, ceiling, and floor can be adequately supported.

### 7.1.1 Bending stress

Elements in a timber structure that are subjected to bending are given the term flexural members. Typical examples of such elements are floor joists, studs, and solid rectangular beams (Figure 16)

This section discusses in detail the general requirements necessary for the design of flexural members with a uniform cross-section, where the grain is essentially parallel to the axis of the element.

Figure 16 Example of a flexural member (Vandervort, 2021)




For beams, design checks have to be done with respect to the shear and bending moment capacity. Following EN 1995, clause 6.1 design of cross-sections subjected to stress in one principal direction applies to members such as solid timber, glued laminated, and wood-based structural products.  The elastic theory Figure17) states that when a solid rectangular member is subjected to a bending moment about the y-y axis (strong axis), the design stress at a distance z should be verified following the following expression:

$$\sigma = \frac{M_y \cdot z}{I_y}$$

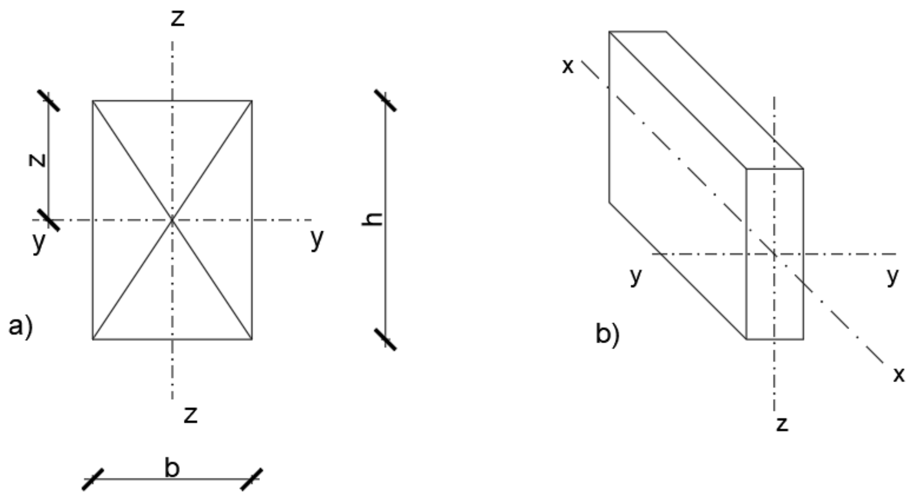
Where:

$I_y$ is the second moment of inertia of the cross-section about the y-axis.

$M_y$ is the bending moment about the y-y axis.

Z is the distance from the neutral axis.

Figure 17. a)Cross-section of a rectangular beam. b) Orientation of coordinate system.




When the compression face of the member is not fully restrained against lateral movement, lateral-torsional instability affects the member bent about the y-y axis. Given that only a moment My exists about the strong axis y, to prove that the maximum bending stress in the section is not exceeding the design bending strength of timber the following expression shall be satisfied:

$$\sigma_{m.d} < k_{crit} \cdot f_{md}$$

Where:

$\sigma_{md}$ is the design bending stress
$f_{md}$ is the design bending strength
$k_{crit}$ is a factor that takes into account the reduced bending strength due to lateral buckling.

The design bending stress is presented as the ratio between the maximum bending moment in the beam to the section modulus. Section modulus W is presented by the expression:

$$W := \frac{b \cdot h^2}{6}$$

Where:

h is the larger cross-sectional dimension

b is the smaller cross-sectional dimension

The value for $k_{crit}$ is determined by following the expression (6.34) in EN1995 1-1. For a value of $\lambda_{rel,m}$ less than 0.75 the element will be stiff enough not to buckle laterally. As this condition is met the value for $k_{crit}$ is recommended to be taken as 1.

The design bending strength shall be determined using the following equation:

$$f_{md} = k_{mod} \cdot \frac{f_{mk}}{\gamma_M} \cdot k_h \cdot k_{sys}$$

Where:

$k_{mod}$ is the modification factor for the duration of load and moisture content

$f_{mk}$ is the characteristic bending strength

$\gamma_M$ is the partial factor for material

$k_h$ is the depth factor

$k_{sys}$ is the system strength factor

For rectangular solid timber the depth factor is given by the following equation:

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

Where:

h is the depth for bending members in mm. The smallest value in parentheses should be used.

Because the factor is dependent on the member size in the direction of bending, the value for the modification factor when bending occurs about the y-y axis can be different from the value when bending is about the z-z axis. The recommended value for $k_{sys}$ is 1 provided that the continuous load distribution system can transmit from one member to the neighboring member.

For a softwood rectangular cross-section, the critical bending stress about its strong axis can be determined by the following expression:

$$\sigma_{mcrit} = \frac{0.78 \cdot b^2}{h \cdot l_{eff}} \cdot E_{0.05}$$

Where:

h is the larger cross-sectional dimension
b is the smaller cross-sectional dimension
$E_{0.05}$ is 5% value of the modulus of elasticity parallel to the grain
$l_{eff}$ is the effective length as a ratio of span. The effective length for the column is increased by 2h, as the axial load is applied at the compression edges.

To link the buckling strength of a beam, $\sigma_{m.crit}$, to its bending strength, $f_{mk}$, the relative slenderness for bending is defined using the following expression:

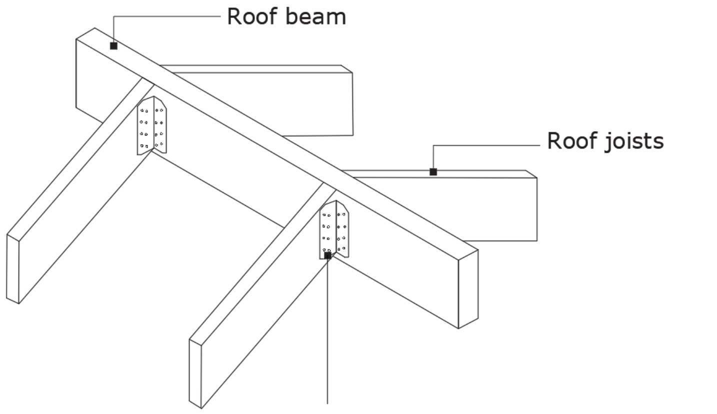
$$\lambda_{rel.m} = \sqrt{\frac{f_{mk}}{\sigma_{mcrit}}}$$

Where:

$f_{mk}$ is the characteristic bending strength
$\sigma_{mcrit}$ is the critical bending stress calculated according to the classical theory of stability, using 5-percentile stiffness values.

Based on the connection shown in figure 18 lateral buckling is not presented as a concern for the beam. The connection between the beam and joists provides enough lateral stiffness.

Figure 18 Roof connection




### 7.1.2 Shear stress

When beams are subjected to bending, shear stresses also appear to be present parallel to the longitudinal axis of the beam (Figure 19 (b). When designing in ULS, the requirement in EN1995-1-1 for shear is represented using the following expression:

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

Where:

$\tau_d$ is the design shear stress at the required level
$f_{v.d}$ is the design shear strength for the actual condition

Regardless of the position studied along the beam, shear stress at the top and the bottom of the cross-section is presented to be zero, and the maximum stress will arise at the neutral axis position. For a rectangular section (Figure 19 (a) the maximum shear stress will is determined following the expression below:

$$\tau_d = \frac{3}{2}\,\frac{V_{ed}}{k_{cr} \cdot b \cdot h}$$

Where:

$V_{ed}$ is the shear force during the design phase
h is the larger cross-sectional dimension
b is the smaller cross-sectional dimension
$k_{cr}$ is the partial safety factor

For solid timbers, the recommended value of the modification factor $k_{cr}$ is 0.67. The introduction of $k_{cr}$ will reduce the shear capacity of solid timber and glulam beams.

The design shear strength for the actual condition should be obtained by the following equation:

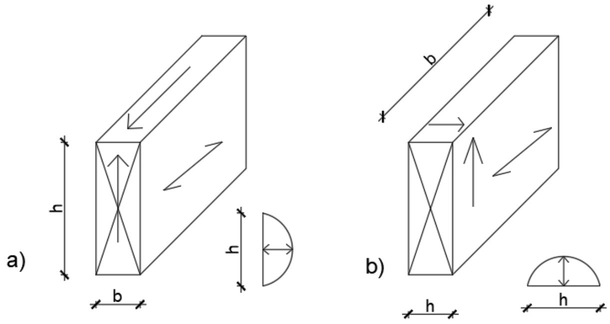
$$f_{v.d} := k_{mod} \cdot \frac{f_{v.k}}{\gamma_M} \cdot k_{sys}$$

Where:

$k_{mod}$ is the modification factor for the duration of load and moisture content
$f_{v.k}$ is the design shear strength factor
$\gamma_M$ is the partial factor for material
$K_{sys}$ is the system strength factor

The value of the modification factor can be selected from table 2. Since snow and live load are presented as both medium-term actions, this modification factor is to be taken as 0.8.

Figure 19 a) Member with a shear stress component parallel to the grain b) Member with both components perpendicular to the grain.




### 7.1.3 Results of bending and shear stresses

This thesis presents the analysis of 2 of the beams of the structure. The first beam has a length of 9.3 meters and is supported in three columns. As the length of the joists connected to this beam varies for different parts of the structure, the analysis is done in different sections, to determine the force that is transferred to it. The second beam has a length of 2.9 meters and is analyzed as a simply supported element, which supports a part of the second floor. Dimensions of each section are identified so that they are not only safe but also cost-effective. Referring to the structural integrity calculations of beams, specifically in appendix 4 and 6, table 14 presents the final results.

Table 14 Results of shear and bending forces acting on beams

| Member | Length | Max. shear force | UR | Max. bending moment | UR |
|---|---|---|---|---|---|
| Beam 1 | 9,3m | 38.93kN | 44,25 % | 34,52kN*m | 55,32 % |
| Beam 2 | 2,9m | 10,48kN | 38.57% | 7,6kN*m | 66,43 % |

## 7.2 Column design

Columns are an important vertical structural part of a building, helping to transmit loads from the building to the foundation. They create a framework that supports the entire load of the structure. Columns are designed to take axial and lateral loads and efficiently transfer them to the base of the structure. Therefore, when all the loads from the beams and floors are transmitted through the column, it is important to design the column to be sturdy.

### 7.2.1 Design of slender members

Timber members are often designed as slender elements, meaning that the member's length is much greater than their cross-section area. Therefore, to avoid problems related to stability, it is crucial to pay attention during the design process.

### 7.2.2 Column buckling

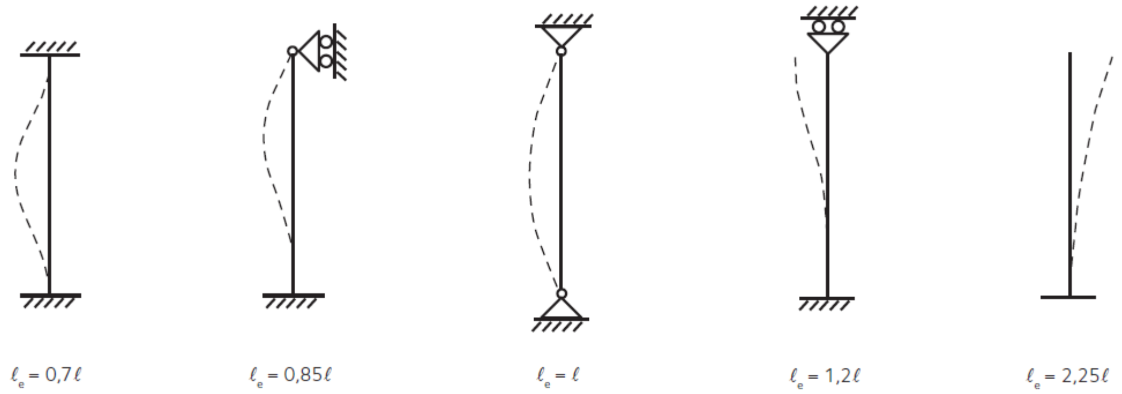
When subjected to axial loads, the slenderness ratio, λ, of the elements increases, due to imperfections in the geometry of the part or variations in its characteristics. Therefore the element will move sideways, and will finally break due to deformation. The more slender the member the larger the possibility of buckling to occur in the element is. Figure 20 provides better visualization of the effective buckling length.

The slenderness ratio is defined as the ratio between the effective length of the column member, $l_{ef}$ with the radius of gyration, i of the given cross-section:

$$\lambda = \frac{l_{ef}}{i}$$

Where the radius of gyration about a specific axis is presented as the ratio of the square root between the second moment of inertia and the surface area of the cross-section.

Figure 20 Effective buckling length $l_{ef}$ for different end conditions. L is the actual column length. (Borgström, 2016)




The relative slenderness ratio $\lambda_{rel}$ for a specific bucking axis is defined:

$$\lambda_{rel} = \frac{\lambda}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$$

Where:

$f_{c,0,k}$ is the characteristic compressive strength of the timber parallel to the grain.
$E_{0.05}$ is 5% value of the modulus of elasticity parallel to the grain.

When taking into account the effects of instability according to EC5, the requirements from columns that are subjected to combined compression and bending forces are different from the strength validation required for beams subjected to combined compression and bending. The equations to be used in design members subjected to combined compression and bending forces are expressed in the following form:

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

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

Where:

$\sigma_{c.0.d}$ is the design compressive stress along the grain. This equation is expressed as the ratio between the maximum axial load applied on the column with the area of the cross-section.

$\sigma_{md.y.d}$ is the design bending stress about the y-axis. This equation is presented as the ratio between the maximum bending moment with the section modulus.

$k_m$ is the reduction factor considering the re-distribution of bending stresses. Recommended value from EN1995 1-1 for rectangular cross-section is to be taken as 0.7. Otherwise select $k_m$ as 1.

$k_{c.y}$ is the reduction factor for buckling depending on the specified axis. Appendix 5 provides guidance on value selection for this coefficient.

$\sigma_{md.z.d}$ is the design bending stress about the z-axis. Bending stress about the z-axis is 0.

In Eurocode 5, $\lambda_{rel,m}$, is defined as the relative slenderness for bending:

$$\lambda_{rel.m} = \sqrt{\frac{f_{mk}}{\sigma_{mcrit}}}$$

For a softwood rectangular cross-section, the critical bending stress about its strong axis can be determined by the following expression:

$$\sigma_{mcrit} = \frac{0.78 \cdot b^2}{h \cdot l_{eff}} \cdot E_{0.05}$$

Where:

h is the larger cross-sectional dimension

b is the smaller cross-sectional dimension

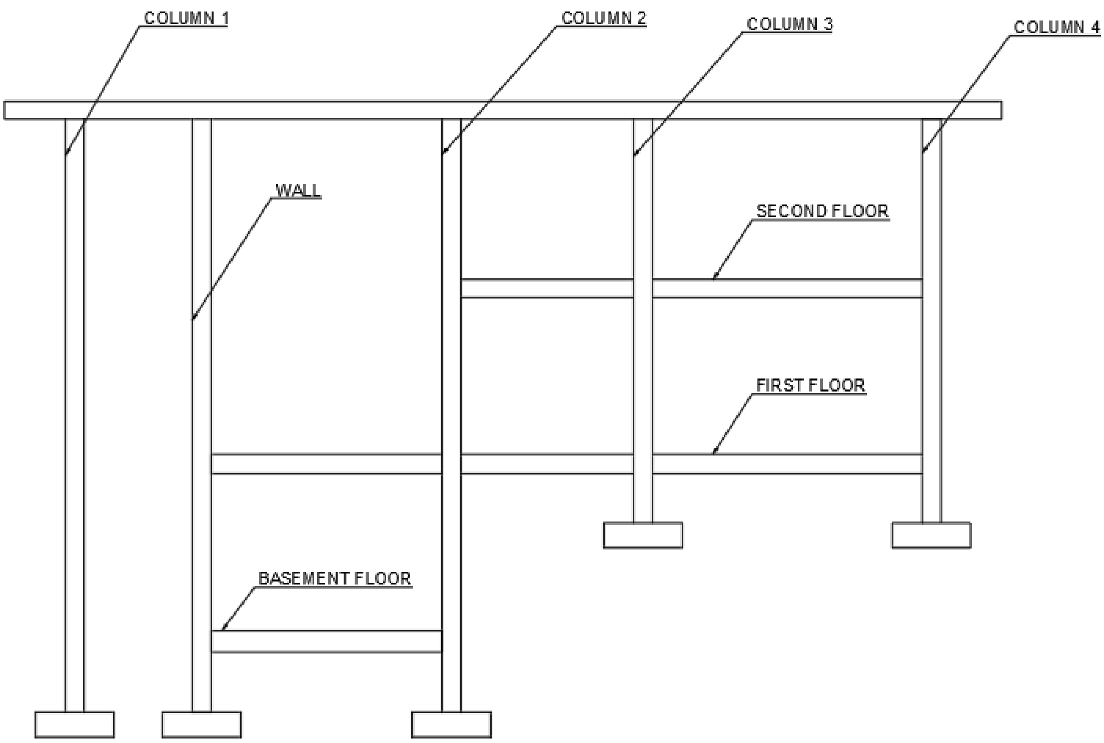
$E_{0.05}$ is 5% value of the modulus of elasticity parallel to the grain

$l_{eff}$ is the effective length as a ratio of span. The effective length for the column is increased by 2h, as the axial load is applied at the compression edges.

Once the slenderness ratio for bending $\lambda$ rel,m is determined, lateral bucking can be neglected, provided that this value is less than 0.75. In reference to the given statement, lateral bucking can be neglected for the column.

This report presents the analysis of the two most critical columns in the building. The first column, which is located outside the building, is subjected to axial load only. This column is presented as the most endangered to buckling as its length is 9 meters. The second column analyzed is located inside the building with a length of 5.4 meters. It is subjected to axial load and bending moment transferred from the beam supporting the second floor (Figure 21).

Figure 21 Location of columns



The second column analyzed is 5.4 meters long and does not extend to the end of the foundation pad, due to the fact that timber elements should not have direct contact with the soil. Referring to the structural integrity calculations of columns, specifically in appendix 5 and 7, table 15 presents the final results for beams.

Table 15 Results of actions applied to columns

| Member | Length | Axial compression | Combined axial and bending |
|---|---|---|---|
| | | UR | UR |
| Column 1 | 9m | 0.547% | - |
| Column 2 | 5.4m | - | 0.622% |

## 7.3 Pad foundation design

The implementation of columns and beams to support the second floor is presented as a necessary option for achieving the structural stability of the building. Yet also the columns must transfer the load they receive from the structure itself. As a result, the design of a second foundation model is necessary to enable the transfer of loads from the columns to the ground as the floor capacity is not enough. This section will cover the steps to follow when designing a pad foundation.

The geotechnical design of a pad foundation is carried out according to the requirements of EN 1997-1: 2005. Eurocode 7 gives three design approaches for the design of the foundation (page 32), however, it permits only design approach 1. In this design approach, partial factors are applied to the action and resistance parameters of the soil.

### 7.3.1 Load bearing capacity

Using the direct method, all limit states should be verified. The final load-bearing capacity of the pad foundation shall be determined using the expression below:

$$q_{ult} = A_{ef} \cdot \frac{\left(c_d \cdot N_c \cdot b_c \cdot s_c \cdot i_c + q' \cdot N_q \cdot b_q \cdot s_q \cdot i_q + 0.5 \cdot \gamma_2 \cdot B_{ef} \cdot N_\gamma \cdot b_\gamma \cdot s_\gamma \cdot i_\gamma\right)}{\gamma_R}$$

Where:

$A_{ef}$ is the effective area of the foundation

$C_d$ is the cohesion

$N_q$ is the overburden factor

$N_\gamma$ is the bodyweight factor

q' is the overburden pressure at the foundation level

$b_c, b_q, b_\gamma$ are coefficients that depend on the slope of the foundation base

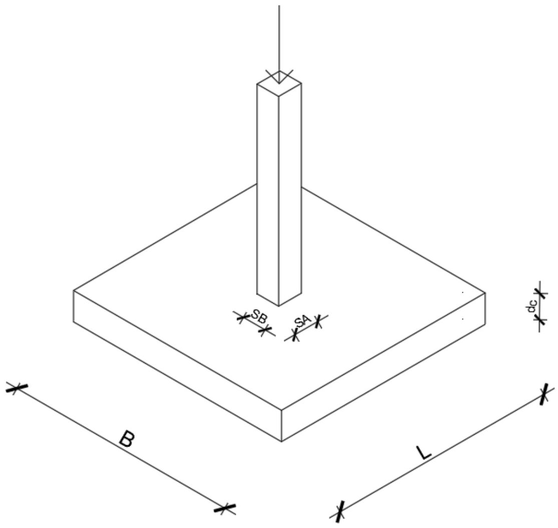
$i_c, i_q, i_\gamma$ are coefficients that depend on the inclination of the load

$s_c, s_q, s_\gamma$ are coefficients that depend on the shape of the foundation

In the static design of an isolated foundation, the reaction force under an axial load can be considered to be evenly distributed when the load is applied to the center without a bending moment. Otherwise, it can be assumed that the pressure distribution changes linearly across the base. Three forces are applied to the column that transfers the largest load to the foundation, where two of them create eccentricity, however, the magnitude of these forces and the moment they create is not significant. On this fact, the total force that is transferred to the foundation is presented as the sum of these forces. The effective area is expressed as the area of the mass concrete pad, considering that the eccentricity does not play an important role in determining the loaded area.

For this design given the fact that the loads transferred to the columns are not very large, the dimensions of the pad are taken as assumptions, where it is then verified whether these preliminary dimensions are suitable in terms of the capacity they should carry.

Figure 22 - Pad foundation




For the design of this foundation these steps have been followed:

- Design ultimate bearing pressure
- Design of reinforcement
- Beam and punching shear

### 7.3.2 Bearing pressure

When designing a foundation, the safe bearing capacity of the soil must withstand the maximum pressure without any damage. If the soil is unable to bear the load it will result in the settlement of the structure, where later the damages will develop into cracks. Therefore, the controls of soil pressure and settlement of the foundation under the factored gravitational loads must be done.

The design soil pressure is given by the expression below:

$$P_{Ed} = \frac{V_{ED}}{L_{ef} \cdot B_{ef}}$$

Where:

$V_{ED}$ is the force applied to the foundation.
$L_{ef}$ is the effective length of the pad
$B_{ef}$ is the effective width of the pad

Following the results presented in appendix 8, it is seen that the soil capacity of 250kN/m$^2$, is sufficient for the force to be applied to it.

### 7.3.3 Design of reinforcement

The use of reinforcements makes it possible to provide additional strength, in areas where concrete does not have the desired capacity. Beyond the main purpose, the presence of reinforcement in the pad foundations makes it possible to reduce the required thickness of the pad. For ease of construction, the pads are usually designed to be square plan areas, especially where eccentric and inclined loadings are present.

The required number of reinforcement is determined using the expression below:

$$A_{s.req} = \beta \cdot L_{ef} \cdot d \cdot \frac{f_{cd}}{f_{yd}}$$

Where:

d is the effective depth of the pad
$f_{cd}$ is the design compressive strength of the concrete class.
$f_{yd}$ is the design yield strength of reinforcement

Effective depth is the functional area of the foundation itself which is expressed as the distance from the edge of the concrete in compression to the center of the longitudinal tension reinforcement.

The design compressive strength is defined using the following expression:

$$f_{cd} := \alpha_{cc} \cdot \frac{f_{ck}}{\gamma_c}$$

Where:

$f_{ck}$ is the characteristic compressive strength. The value can be selected from table 16.
$\alpha_{cc}$ is a coefficient that takes into account the long-term effects on the compressive strength. The recommended value from Finnish National Annex for this factor is 0.85.
$\gamma_C$ is the partial safety factor for concrete. The recommended value is to be taken as 1.5.

The yield strength of reinforcement is defined using the following expression:

$$f_{yd} := \frac{f_{yk}}{\gamma_s}$$

Where:

$f_{yk}$ is the characteristic yield strength of reinforcement. For steel grade AH500 the characteristic yield strength is 500MPa.
$\gamma_s$ is the partial safety factor for steel. The recommended value is to be taken as 1.15.

Following the results in appendix 8, it is seen that the required area of reinforcement is 56.44 $mm^2$, however, EN 1992-1-1 represents a minimum value for the area of reinforcement, regardless of the element that is being designed. The minimum value of the reinforcement is determined using the following expression:

$$A_{s.min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot L_{ef} \cdot d$$

Where:

$f_{ctm}$ is the mean tensile strength of concrete. The value for this factor can be selected from table 16.

Since the minimum required area of reinforcement is greater than the required area, the minimum area is used to determine the number of rebars. The number of rebars is 4 on each side.

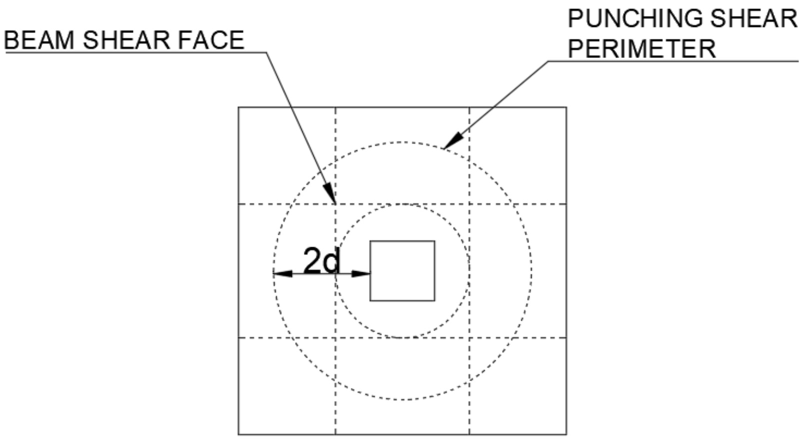
Table 15 Strength and deformation characteristics for concrete (SFS EN 1992-1-1:2004)

| Strength classes for concrete | | | | | | | | | | | | | | | Analytical relation / Explanation |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $f_{ck}$ (MPa) | 12 | 16 | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 | 60 | 70 | 80 | 90 | |
| $f_{ck,cube}$ (MPa) | 15 | 20 | 25 | 30 | 37 | 45 | 50 | 55 | 60 | 67 | 75 | 85 | 95 | 105 | |
| $f_{cm}$ (MPa) | 20 | 24 | 28 | 33 | 38 | 43 | 48 | 53 | 58 | 63 | 68 | 78 | 88 | 98 | $f_{cm} = f_{ck}+8$(MPa) |
| $f_{ctm}$ (MPa) | 1,6 | 1,9 | 2,2 | 2,6 | 2,9 | 3,2 | 3,5 | 3,8 | 4,1 | 4,2 | 4,4 | 4,6 | 4,8 | 5,0 | $f_{ctm}=0,30\times f_{ck}^{(2/3)} \leq$C50/60<br>$f_{ctm}=2,12\cdot\ln(1+(f_{cm}/10))$ > C50/60 |
| $f_{ctk,0,05}$ (MPa) | 1,1 | 1,3 | 1,5 | 1,8 | 2,0 | 2,2 | 2,5 | 2,7 | 2,9 | 3,0 | 3,1 | 3,2 | 3,4 | 3,5 | $f_{ctk;0,05} = 0,7\times f_{ctm}$<br>5% fractile |
| $f_{ctk,0,95}$ (MPa) | 2,0 | 2,5 | 2,9 | 3,3 | 3,8 | 4,2 | 4,6 | 4,9 | 5,3 | 5,5 | 5,7 | 6,0 | 6,3 | 6,6 | $f_{ctk;0,95} = 1,3\times f_{ctm}$<br>95% fractile |
| $E_{cm}$ (GPa) | 27 | 29 | 30 | 31 | 33 | 34 | 35 | 36 | 37 | 38 | 39 | 41 | 42 | 44 | $E_{cm} = 22[(f_{cm})/10]^{0,3}$<br>($f_{cm}$ in MPa) |
| $\varepsilon_{c1}$ (‰) | 1,8 | 1,9 | 2,0 | 2,1 | 2,2 | 2,25 | 2,3 | 2,4 | 2,45 | 2,5 | 2,6 | 2,7 | 2,8 | 2,8 | see Figure 3.2<br>$\varepsilon_{c1}$ (‰) $= 0,7\, f_{cm}^{0,31} < 2.8$ |
| $\varepsilon_{cu1}$ (‰) | 3,5 | | | | | | | | | 3,2 | 3,0 | 2,8 | 2,8 | 2,8 | see Figure 3.2<br>for $f_{ck} \geq 50$ Mpa<br>$\varepsilon_{cu1}$(‰)$=2,8+27[(98-f_{cm})/100]^4$ |
| $\varepsilon_{c2}$ (‰) | 2,0 | | | | | | | | | 2,2 | 2,3 | 2,4 | 2, 5 | 2,6 | see Figure 3.3<br>for $f_{ck} \geq 50$ Mpa<br>$\varepsilon_{c2}$(‰)$=2,0+0,085(f_{ck}-50)^{0,53}$ |
| $\varepsilon_{cu2}$ (‰) | 3,5 | | | | | | | | | 3,1 | 2,9 | 2,7 | 2,6 | 2,6 | see Figure 3.3<br>for $f_{ck} \geq 50$ Mpa<br>$\varepsilon_{cu2}$(‰)$=2,6+35[(90-f_{ck})/100]^4$ |
| $n$ | 2,0 | | | | | | | | | 1,75 | 1,6 | 1,45 | 1,4 | 1,4 | for $f_{ck} \geq 50$ Mpa<br>$n=1,4+23,4[(90-f_{ck})/100]^4$ |
| $\varepsilon_{c3}$ (‰) | 1,75 | | | | | | | | | 1,8 | 1,9 | 2,0 | 2,2 | 2,3 | see Figure 3.4<br>for $f_{ck} \geq 50$ Mpa<br>$\varepsilon_{c3}$(‰)$=1,75+0,55[(f_{ck}-50)/40]$ |
| $\varepsilon_{cu3}$ (‰) | 3,5 | | | | | | | | | 3,1 | 2,9 | 2,7 | 2,6 | 2,6 | see Figure 3.4<br>for $f_{ck} \geq 50$ Mpa<br>$\varepsilon_{cu3}$(‰)$=2,6+35[(90-f_{ck})/100]^4$ |

### 7.3.4 Beam and punching shear

The sum of the loads that act outside the section considered is given the term the vertical shear force. Shear stress is checked at a distance d from the face of the column (Figure 23 ). It is recommended to make the base of a structure deep enough so that the shear reinforcement is not required. The effective depth is presented when calculating bending reinforcements.

Figure 23 Beam and punching shear perimeters



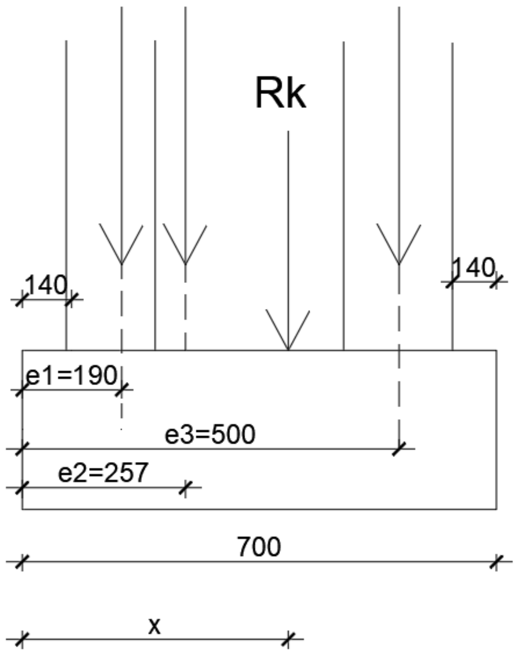

The summation of loads applied outside the periphery of the critical section is given the term punching shear. Shear stress is checked at a distance of 2d from the face of the column. Conservatively, when designing this phenomenon, the assumption that the pressure affects the whole area outside the circuit is followed. Following the results presented in appendix 5 the pad with dimensions 800 by 800 passes the necessary checks.

## 7.4 Strip foundation design

The strip foundation is widely used in the construction industry. They are designed for structures where the load is relatively small, from low to medium-rise domestic buildings. For this project, the continuous strip acts as a support for the load-bearing walls, which transfer force in a distributed manner.

To design this foundation, all the forces that have an effect on its capacity must be analyzed, together with the moment they create, as shown in figure 24. The design process presents various tests performed on the selection of dimensions, to make it possible that the ratio between the pressure exerted on the allowable bearing capacity of the soil is less than 100%.

Figure 24 Preliminary size of the foundation

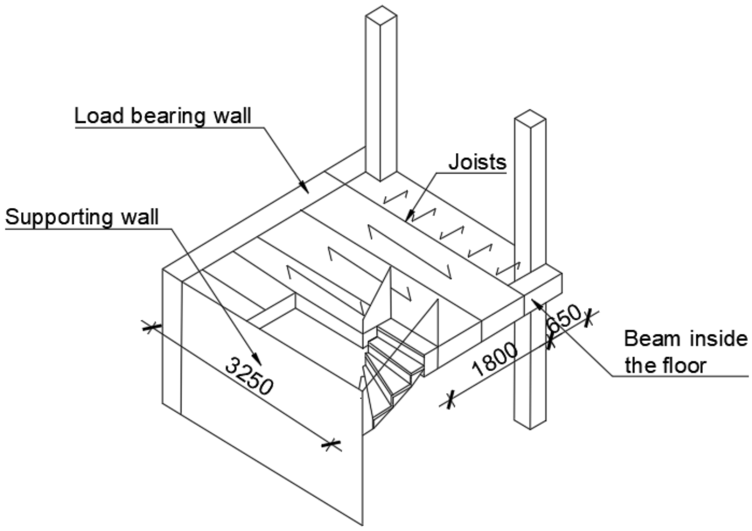



To achieve this result the foundation is moved to the left until the distance x is 390mm. Following the results in the appendix 9 the dimensions of the foundation are 700x250 mm. The presented results belong to the first design of the building, where the forces transferred on it come only from the walls of the first floor. The forces transferred to this foundation in the first design are greater than in the design addressed by this thesis, therefore the same dimensions are used. As in the case of the design for pad foundation, the required number of reinforcement is too small and unreasonable. It is important to understand that beyond the results presented during the calculations, the numbers used must be acceptable, therefore a reinforced concrete mesh 5/150 is used for the design.

## 7.5 Staircase landing support design

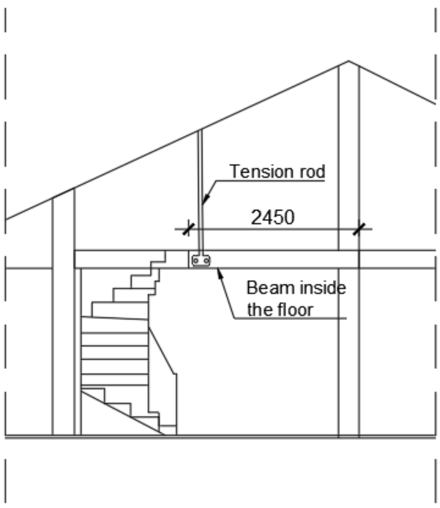
The design of the stairs is yet another task to consider with the presence of the second floor. In the analytical model, this part of the second floor is presented as a cantilever structure, however, the bending moment in the joists would be very large, and eventually would lead to the collapse of the structure. The stairs have two points of support on the wall shown in figure 25 whereas the landing is supported on the second floor.

Figure 25 Staircase landing




To support this structure, a 2.45-meter-long beam has been installed inside the floor structure. This beam has as support a steel rod that will be installed on the left side of it, and the column on the right side (Figure 26). The placement of the joists distributes the weight in two support, respectively the load-bearing wall and the newly installed beam. The load coming from the stairs can be interpreted as a two-point load, however, being more conservative during the design, it is continued with the assumption that the whole load is transferred as a point load to the support (steel rod).

Figure 26 Tension rod



Following the results in appendix 3, tension inside the rod is 16.47kN. To carry the load a circular hollow section (CHS 48.3/3.2) is selected. As a vertical support 2 joists are joined together, on which this steel rod will be connected.

# 8 Analysis and discussion

This thesis seeks to present a workflow for the design of timber structures in the concept design phase, where the main focus is on structural optimization. The research questions were formulated rather widely, however, the intention for them to serve as an efficient searchlight and approach to the work was achieved. The findings are analyzed in a way that can be used as a basis for further discussions on the main topic.

Referring to the first research question that arises, for a design to be considered successful it must be presented as a function of respecting realistic criteria. The need to have a clear understanding of the current properties of the wood used in real designs, as well as to move in a direction that aims at the reliability-based design, made the need for factors in the limit state. This thesis deals with the characteristic conditions of the following:

- Duration of applied loads
- Partial factors and modification factors.
- Moisture content depending on the service class.

As a structural element, timber experiences a significant loss of integrity over time. To account for the loss of resistance, load time classes have been established to facilitate the design process. Making a correlation between the impact of the service class and the duration of the loads applied, the element is assigned a reduction factor named $k_{mod}$. By increasing the duration of the load and service class at the same time the capacity of the element decreases significantly as the value of $k_{mod}$ decreases too.

The second question of the project arises from the hypothesis that the presented option for the implementation of columns and beams is appropriate to achieve structural integrity. Two different case studies provided a welcome overview of the possibilities, potentials, and challenges of structural applications. Since the building has a limit on living space, each part of the building must be used more efficiently. From this fact, the roof structure differs from that with trusses to that with joists, changing the way loads are transferred through the structure. It is here the first problem encountered for roof ridge support. In the first model, the truss elements served as such, while for this design the support of the roof ridge is necessary, otherwise, the structure will collapse. This fact, but also the other that the forces to be distributed in the retaining walls will be greater than their capacity, make the installation of beams and columns a key element in achieving the structural integrity of the building. Normally the design offers different possibilities on the location of columns and beams, however, based on the layout of the building, well-thought-out ideas should be put into practice. The same can be said about stairway support design. Installing a beam and a column below the landing would make it very simple from a structural point of view. But it should not be forgotten that such an option blocks part of the entrance to the stair case and at the same time affects the aesthetics of the building.

Furthermore, the methodology provides a form for gathering and analyzing information through Eurocodes. Eurocodes represent the latest standards used in the construction industry as guidelines for engineers and other construction professionals. There are opportunities to be had, and if the right guidance is used, they could be a real benefit. On this fact, this thesis consists of a theoretical and an experimental part. The theory provides the framework and context for the proposed workflow, whereas the experimental part uses that knowledge for recreating different parts of a design process. It is worth mentioning that Eurocodes present information to verify the capacity of an element, however, they can not be used as manuals nor offer any solution in case a structure fails. It is very important to master the basic concepts of the design stages.

Referring to the above statement during the project, problems are encountered, on which the design of the foundation can be mentioned. Information and concepts for this element have not been addressed during the study years, therefore this design process is validated, using Eurocode and online materials as references.

# 9 Conclusions

The result of the thesis project is a comprehensive understanding of the step-by-step calculation for the main structural elements in a timber building. Based on this work, the client can order items with the respective sizes specified through calculations.

As a result of the design work, the project is presented in 4 main phases. The first phase provides information on structural elements and their connections, that will later be used on the site. The second phase presents general information on the knowledge and concepts for the limit state design of timber. Factors that affect the integrity of the elements being studied are numerous so maximum care should be taken during their selection. In the third and fourth phases are presented all the forces and factors that must be considered during the design of the elements. Solutions are also addressed where the design of a steel rod can be mentioned to enable the support of stairs, as well as the implementation of new techniques such as the methodology used with composite slabs with load-bearing metal sheets.

Nowadays, modern software allows designers and engineers to boost the working flow and manage all the aspects of the project. It has evolved into advancements that bring each step of the design and calculation together seamlessly, replacing unnecessary steps and parts of traditional methods. The use of RFEM software plays a very important role in this project too, however, it must be noted that the software is capable of solving problems based on the commands applied to it. Despite its advantages and cutting edge technology of programming, every designer must master the basic concepts of the design stages.

This document aims to work as a reference and give the reader a clear view of the practice calculation routines quickly while the theory is going through and make it easier to find and obtain additional information based on the source references.

# 10 References

SFS - EN1990 Eurocode (2002, 2005). Basis of structural design. SFS Online. Retrieved 10 April 2022 from https://online.sfs.fi

SFS - EN1991-1-1 Eurocode (2002). Actions on structures. Part 1-1: General actions. Densities, self-weight, imposed loads for buildings. SFS Online. Retrieved 10 April 2022 from https://online.sfs.fi

SFS - EN1991-1-3 Eurocode (2003). Actions on structures. Part 1-3: General actions. Snow loads. SFS Online. Retrieved 10 April 2022 from https://online.sfs.fi

SFS - EN1991-1-4 Eurocode (2005). Actions on structures. Part 1-4: General actions. Wind actions. SFS Online. Retrieved 10 April 2022 from https://online.sfs.fi

SFS - EN1995-1-1 Eurocode (2004). Design of timber structures. Part 1-1: General. Common rules and rules for buildings. SFS Online. Retrieved 10 April 2022 from https://online.sfs.fi

RIL 201-1-2017 Design criteria and structural loads. Eurocodes EN 1990, EN 1991-1-1, EN 1991-1-3, EN 1991-1-4. Retrieved 10 April 2022 from https://www.ellibslibrary.com/

National Annex 2 to standard SFS-EN1991-1-1. Retrieved 10 April 2022 from https://www.eurocodes.fi

National Annex 4 to standard SFS-EN1991-1-3. Retrieved 10 April 2022 from https://www.eurocodes.fi

Shqefni, L. (2022).Design of a two apartment house [Bachelor's thesis, HAMK Häme University of Applied Sciences].

Admin, S. (2019, November 2019). Indore institute. From https://indoreinstitute.com/the-significance-of-structural-design-in-civil-engineering//

Borgström, E. (2016). Design of timber structures Volume 1. Stockholm: Swedish Wood.

Designing Building. (2020, December 02). From https://www.designingbuildings.co.uk/wiki/Roof_structure

Finnfoam. (n.d.). From https://www.finnfoam.fi/kayttokohteet/alapohja/maanvarainen-alapohja

Lang, A. (2016). docplayer. From https://docplayer.net/18443906-Chapter-7-structural-design.html

McEntee, P. (2012, July 18). Seblog Strongtie. From https://seblog.strongtie.com/tag/lateral-loads/

Mtcopeland. (2020, October 13). From https://mtcopeland.com/blog/live-load-vs-dead-load-vs-environmental-load-differences/

Paroc. (n.d.). From https://www.paroc.com/applications/building-insulation/partitions

Paroc. (n.d.). From https://www.paroc.fi/kayttokohteet/rakennusten-eristaminen/harjakatot-ja-ullakot

The Structural World. (2018, November 3). From https://www.thestructuralworld.com/2018/11/03/how-important-is-the-load-combinations-in-structural-design/

Vandervort, D. (2021, March 23). Hometips. From https://www.hometips.com/how-it-works/floor-subflooring.html

## Appendix 1: Snow load

**Snow load**

$\alpha := \operatorname{atan}\left(\frac{1}{2}\right) \cdot \frac{180}{\pi} = 26.565$

Roof angle in degress.

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

From EN1991-1-3, (5.1).

$s_k := 3\ \frac{kN}{m^2}$

Characteristic value of snow load on tile ground

$\mu_i := 0.8$

Snow load shape coefficient (see Section 5.3 and Annex B)

$C_e := 1$

Exposure coefficient

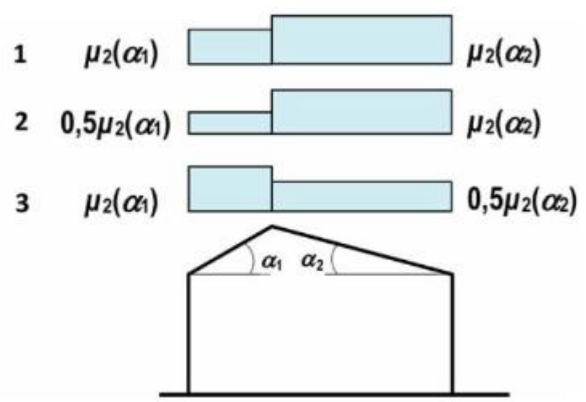
$C_t := 1$

Thermal coefficient

$S := \mu_i \cdot C_e \cdot C_t \cdot s_k = 2.4\ \frac{kN}{m^2}$

Snow load on the roof

Load arrangement




**Case (i)**

$Load := S = 2.4\ \frac{kN}{m^2}$

**Case (ii)**

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

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

**Case (iii)**

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

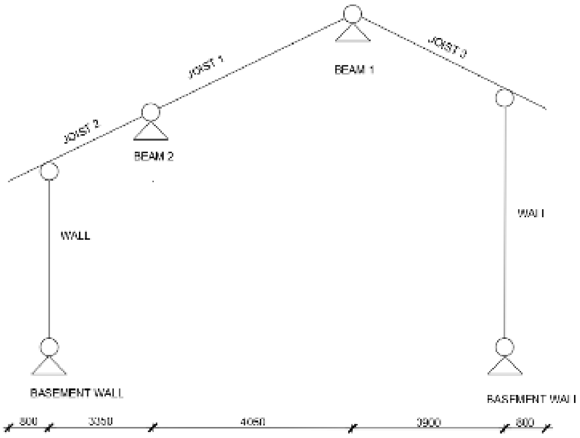
$$Load_{left.side} := S = 2.4\ \frac{kN}{m^2}$$

Considering that in the design phase, the loading scenario is the maximum possible factored loading, which represents the worst case, the same procedure is followed regarding the distribution of snow load on the roof is 2.4kN/m2 on each side.

**Appendix 2: Joist design**

**JOIST CAPACITY**

From this section cut the longest joist can be determined however depending on the connection of the joist with beams the most critical would be between beam 1 and 2. Analyse it as a simply supported element.




**Geometric properties of the joist**

| | |
|---|---|
| $b := 45\ \text{mm}$ | Width of joist |
| $h := 360\ \text{mm}$ | Depth of joist |
| $L_{joist} := 4.05\ \text{m}$ | Length of joist studied |
| $W_y := \dfrac{b \cdot h^2}{6}$ | Section modulus |
| $l_1 := 0.6\ \text{m}$ | Tributary length of joist |

**Laminated veneer lumber (LVL) properties**

| | |
|---|---|
| $f_{mk} := 36\ \text{MPa}$ | Characteristic bending strength |
| $f_{v.k} := 4.1\ \dfrac{\text{N}}{\text{mm}^2}$ | Design shear strength factor |
| $E_{0.05} := 9.8\ \dfrac{\text{kN}}{\text{mm}^2}$ | 5% modulus elasticity parallel to the grain |

$G_{mean} := 0.6\ \frac{kN}{mm^2}$ Mean shear modulus

**Partial safety factors**

$\gamma_G := 1.15$ Permanent action

$\gamma_Q := 1.5$ Variable action

$\gamma_M := 1.25$ Partial safety factor, EN1995-1-1 Table 2.3

$K_{FI} := 1$ Consequent class 2, EN1990

**Actions**

$G_{k.1} := 0.553\ \frac{kN}{m^2}$ Characteristic dead load of the roof

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

$q_{Ed} := \gamma_G \cdot K_{FI} \cdot G_{k.1} + \gamma_Q \cdot K_{FI} \cdot Q_{k.1} = 4.236\ \frac{kN}{m^2}$ EN1990, equation 6.10a

$q_{dist} := q_{Ed} \cdot l_1 = 2.542\ \frac{kN}{m}$ Distributed load along the beam in design phase

**Modification factors**

$k_{mod} := 0.8$ Modification factor, EN1995-1-1, 3.2, table 3.1. Snow is medium-term load.

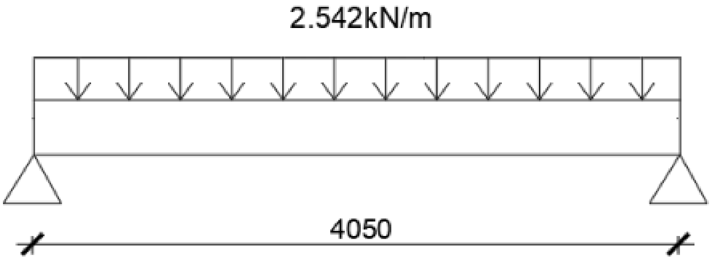
$s := 0.12$ Bending strenght size effect parameter. (LVL properties)

$k_h := min\left(\frac{300\ mm}{h}\right)^s = 0.978$ Reference depth in bending, EN1995-1-1, (3.3).

$l_{eff} := L_{joist} + 2 \cdot h = 4.77\ m$ Effective length of joist

$k_{sys} := 1$ System strength factor EN1995-1-1, 6.6, Figure 6.12

**CHECKS FOR BENDING STRESS**




$V_{Ed} := \frac{q_{dist} \cdot L_{joist}}{2} = 5.147\ kN$ Shear force in design phase

$M_{Ed} := \frac{q_{dist} \cdot L_{joist}^{2}}{8} = 5.211\ kN \cdot m$ Bending moment in design phase

$\sigma_{m.y.d} := \frac{M_{Ed}}{W_y} = 5.361\ MPa$ Design bending stress about the principal y-axis

$f_{myd} := k_{mod} \cdot \frac{f_{mk}}{\gamma_M} \cdot k_h \cdot k_{sys} = 22.541\ MPa$ Design bending strength about the principal y-axis

$k_{crit} := 1$

if $\sigma_{m.y.d} < k_{crit} \cdot f_{myd}$
  "OK"
else
  "CHECK BENDING STRESS "
= "OK"

$UR := \frac{\sigma_{m.y.d}}{k_{crit} \cdot f_{myd}} = 23.783\%$ Utilization ratio

**SHEAR CONTROL**

$\tau_d \le f_{v.d}$ Condition to be fulfilled

$V_{Ed} = 5.147\ \text{kN}$ Shear force in design phase

$k_{cr} := 1$ Modification factor for shear. Recommended value for LVL

$$\tau_d := \frac{3}{2} \frac{V_{Ed}}{k_{cr} \cdot b \cdot h} = 0.477\ \text{MPa}$$

Design shear stress

$$f_{v.d} := k_{mod} \cdot \frac{f_{v.k}}{\gamma_M} \cdot k_{sys} = 2.624\ \text{MPa}$$

Shear design strength for the actual condition

```
if τd < fv.d              | = "OK"
  || "OK"                 |
else                      |
  || "CHECK SHEAR CONTROL"|
```

$$UR := \frac{\tau_d}{f_{v.d}} = 18.161\%$$

## Appendix 3: Staircase landing support design

**STAIRCASE SUPORT ROD DESIGN**

$A_1 := 1.3\ m^2$ — Area of the stairs

$\rho_{C24} := 4.2\ \frac{kN}{m^2}$ — Mean density of C24 wood

$Q_{k.1} := 2\ \frac{kN}{m^2}$ — Characteristic live load in stairs(NATIONAL ANNEX TO STANDARD SFS-EN 1990)

$N_d := A_1 \cdot \rho_{C24} = 5.46\ kN$ — Normal force coming from the self weigh of the stairs

$N_l := Q_{k.1} \cdot A_1 = 2.6\ kN$ — Normal force coming from the live load of the stairs

When designing the tension rod, load transferred from the stairs can be applied as 2 point loads. To be more conservative in design, it is assumed that the entire load of the stairs comes as a point load at the support (steel rod).

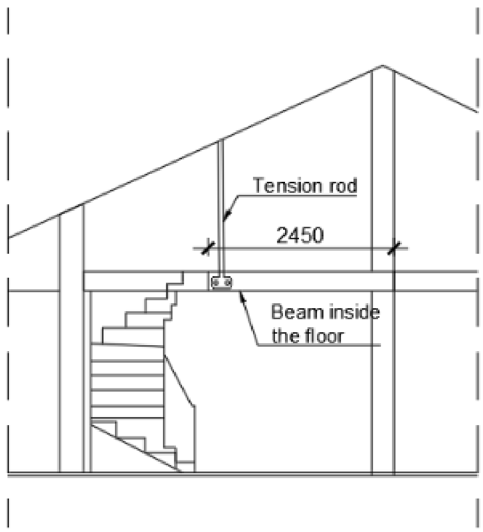
**Partial safety factors**

$\gamma_G := 1.15$ — Permanent action

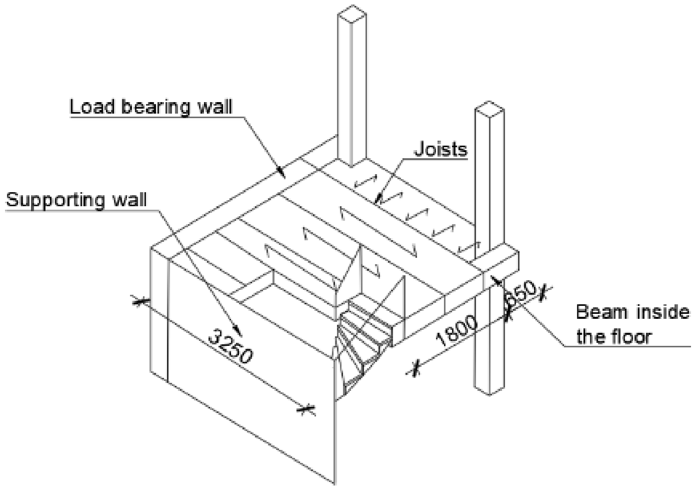
$\gamma_Q := 1.5$ — Variable action

$K_{FI} := 1$ — Consequent class 2, EN1990

$N_{Ed} := \gamma_G \cdot K_{FI} \cdot N_d + \gamma_Q \cdot K_{FI} \cdot N_l = 10.179\ kN$ — Normal force applied on the left support.



In addition to the point load of the beam to which the steel rod is connected, the weight of the second floor is transferred, which is supported on one side by this beam and on the other side by the load bearing wall.




$L_{beam} := 2.45\ m$ Length of beam

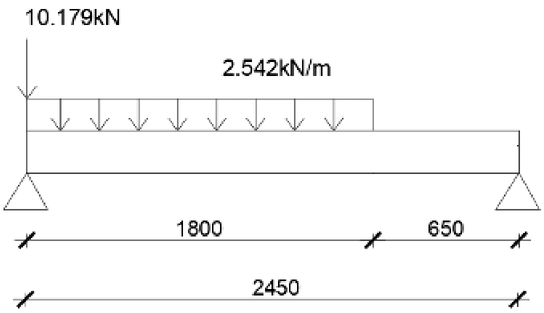
$l_1 := \dfrac{3250\ mm}{2} = 1.625\ m$ Tributary length of the beam

$G_{k.1} := 0.45\ \dfrac{kN}{m^2}$ Characteristic dead load of the second floor

$Q_{k.1} := 2\ \dfrac{kN}{m^2}$ Characteristic live load on the second floor

$q_{Ed} := \gamma_G \cdot K_{FI} \cdot G_{k.1} + \gamma_Q \cdot K_{FI} \cdot Q_{k.1} = 3.518\ \dfrac{kN}{m^2}$ EN1990, equation 6.10a

$q_{dist} := q_{Ed} \cdot l_1 = 5.716\ \dfrac{kN}{m}$ Distributed load along the beam in design phase

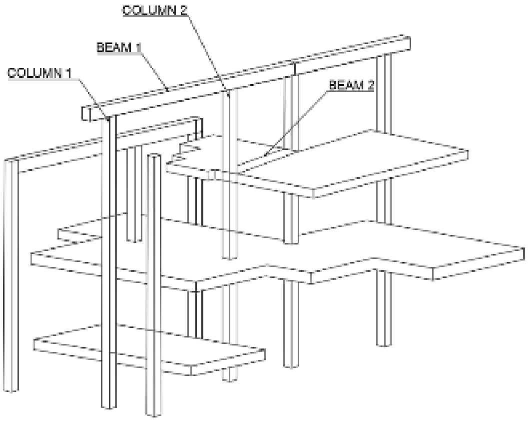


Results generated from RFEM


| | |
|---|---|
| $R_a := 16.69\ \text{kN}$ | Reaction on the left support(rod) |
| $R_b := 3.378\ \text{kN}$ | Reaction on the right support(column) |
| CHS 48.3 / 3.2 | Circular Hollow Section S355 |
| $A := 453\ \text{mm}^2$ | Area of the section |
| $f_y := 355\ \text{MPa}$ | Yield strength |
| $\gamma_{M0} := 1$ | Recommended value for cross section class 1 |
| $N_{Ed} := R_a = 16.69\ \text{kN}$ | |
| $N_{t.Rd} := \dfrac{A \cdot f_y}{\gamma_{M0}} = 160.815\ \text{kN}$ | Design plastic resistance |
| $UR := \dfrac{N_{Ed}}{N_{t.Rd}} = 10.378\%$ | Utilization ratio |

## Appendix 4: Design of most critical beam

**DESIGN OF A BEAM 1 (CONTINUOUS)**




**Geometric properties of the beam 1**

$b := 200\ mm$ — Width of beam

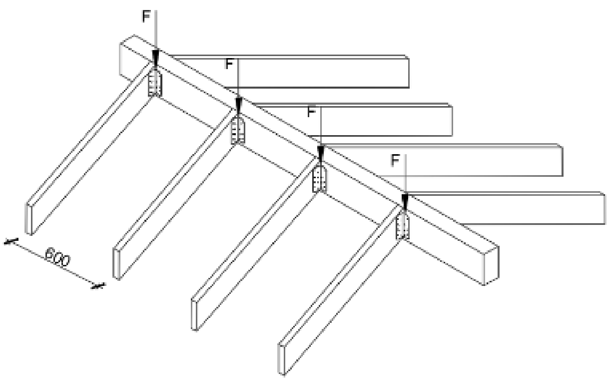
$h := 400\ mm$ — Depth of beam

$L_{beam} := 9.3\ m$ — Length of beam

$W_y := \frac{b \cdot h^2}{6}$ — Section modulus

$l_1 := 0.6\ m$ — Spacing between joists

The weight of the roof is transferred to the beam through studs as point load. As a start, the studs and the forces they tranfer to the beam from both sides is analyzed. The resultant of the forces on both sides is divided by 0.6m which is the space of the studs, to display this force as distributed load.



**Softwood C24 properties**

| | |
|---|---|
| $f_{mk} := 24\ \text{MPa}$ | Characteristic bending strength |
| $f_{v.k} := 4\ \dfrac{N}{mm^2}$ | Design shear strength factor |
| $E_{0.05} := 7.4\ \dfrac{kN}{mm^2}$ | 5% modulus elasticity paralel to the grain |

**Partial safety factos**

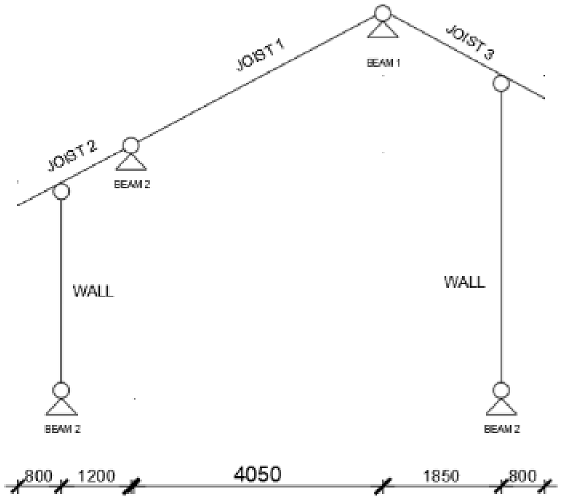
| | |
|---|---|
| $\gamma_G := 1.15$ | Permanent action |
| $\gamma_Q := 1.5$ | Variable action |
| $\gamma_M := 1.3$ | Partial facety factor, EN1995-1-1 Table 2.3 |
| $K_{FI} := 1$ | Consequent class 2, EN1990 |

**Actions**

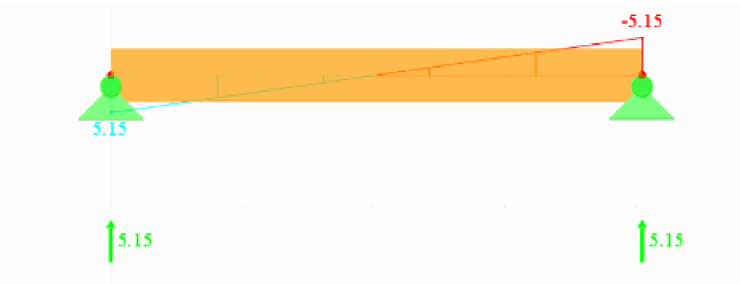
| | |
|---|---|
| $G_{k.1} := 0.553\ \dfrac{kN}{m^2}$ | Characteristic dead load of the roof. |
| $Q_{k.1} := 2.4\ \dfrac{kN}{m^2}$ | Snow load on the roof |
| $q_{Ed} := \gamma_G \cdot K_{FI} \cdot G_{k.1} + \gamma_Q \cdot K_{FI} \cdot Q_{k.1} = 4.236\ \dfrac{kN}{m^2}$ | EN1990, equation 6.10a |
| $q_{joist} := q_{Ed} \cdot l_1 = 2.542\ \dfrac{kN}{m}$ | Distributed load on the joist |

Since the length of the joist varies for different parts of the building, the analysis is done in 2 sections.

SECTION 1

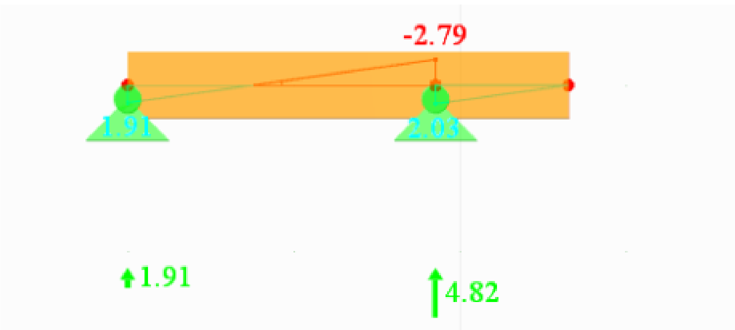



Analysing joist 1 to determine force transferred to beam 1 from the left side.




$R_l := 5.15\ kN$ Reaction force from the left side

Analysing joist 3 to determine force transferred to beam 1 from the right side.(continuos)



$R_r := 1.91\ \text{kN}$

Reaction force from the right side

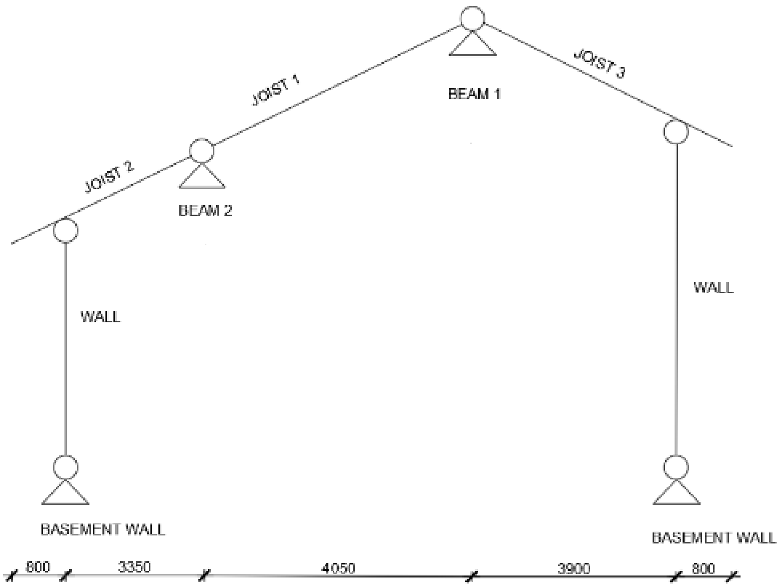
$R_{tot1} := R_l + R_r = 7.06\ \text{kN}$

Total force coming as a point load on the beam (section 1)

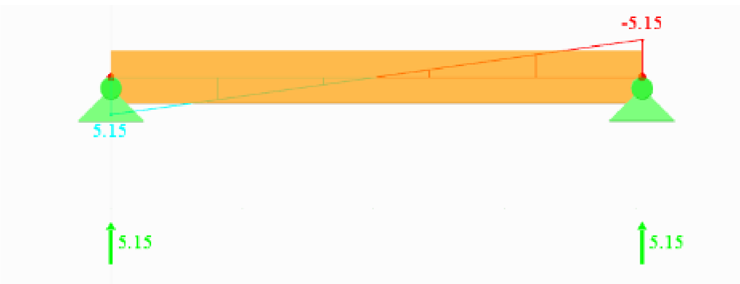
$q_{Ed1} := \dfrac{R_{tot1}}{0.6\ \text{m}} = 11.767\ \dfrac{\text{kN}}{\text{m}}$

Distributed load along the first section of the beam.

SECTION 2

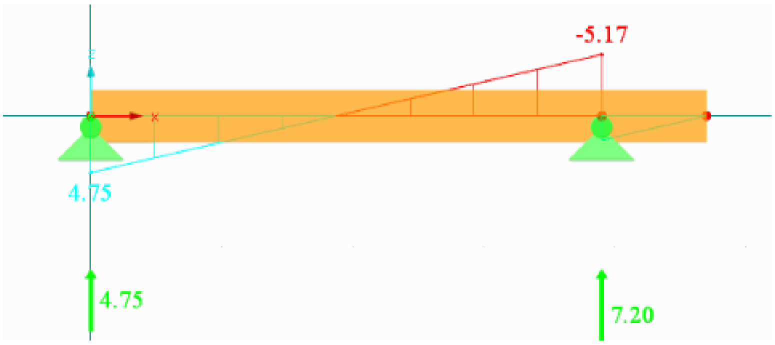



Analysing joist 1 to determine force transferred to beam 1 from the left side.




$R_{l2} := 5.15\ \text{kN}$

Reaction force from the left side




$R_{r2} := 4.75\ kN$ Reaction force from the righ side (section 2)

$R_{tot2} := R_{l2} + R_{r2} = 9.9\ kN$ Total force coming as a point load on the beam (section 1)

$$q_{Ed2} := \frac{R_{tot2}}{0.6\ m} = 16.5\ \frac{kN}{m}$$

Distributed load along the first section of the beam.

**Modification factors**

$k_{mod} := 0.8$ Modification factor, EN1995-1-1, 3.2, table 3.1. Snow is medium-term load.

$$k_h := min\left(1.3, \left(\frac{150\ mm}{h}\right)^{0.2}\right) = 0.822$$

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

$l_{eff} := L_{beam} = 9.3\ m$ Effective length of beam

$k_{sys} := 1$ System strength factor EN1995-1-1, 6.6, Figure 6.12

**LATERAL TORSIONAL BUCKLING CHECK**

To check stability of the beams it must be ensured that lateral buckling is not an issue, however the connection between the joists and beams provides enough lateral support. Nevertheless when designing the bending stress about y-axis , kcrit is required. Following EN1995-1-1, 6.3.3(6.30), the value of kcrit is depended on the relative slenderness ratio.

$\lambda_{rel.m} < 0.75$

Condition to be fulfilled for lateral buckling to be neglected

$$\sigma_{mcrit} := \frac{0.78 \cdot b^2}{h \cdot l_{eff}} \cdot E_{0.05} = 62.065 \text{ MPa}$$

Critical bending stress 6.3.3 (6.32)

$$\lambda_{rel.m} := \sqrt{\frac{f_{mk}}{\sigma_{mcrit}}} = 0.622$$

Relative slenderness, EN1995-1-1, 6.3.3(6.30).

$$\begin{array}{l} \text{if } \lambda_{rel.m} \le 0.75 \\ \quad \| \text{“NEGLECT LATERAL BUCKLING”} \\ \text{else} \\ \quad \| \text{“CHECK LATERAL BUCKLING”} \end{array} \Bigg| = \text{“NEGLECT LATERAL BUCKLING”}$$

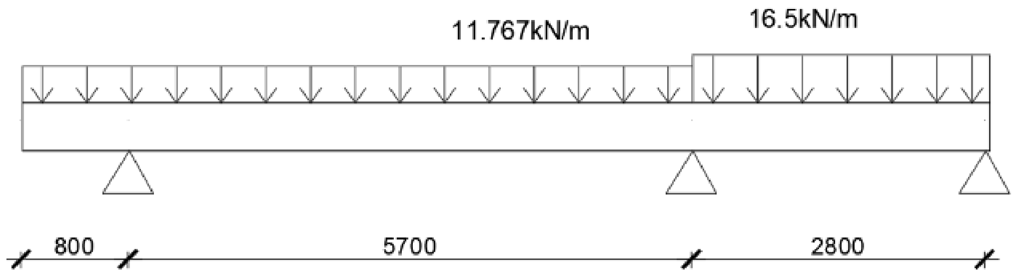
$$k_{crit} := \begin{array}{l} \text{if } \lambda_{rel.m} \le 0.75 \\ \quad \| 1 \\ \text{else if } 0.75 < \lambda_{rel.m} \le 1.4 \\ \quad \| 1.56 - 0.75 \cdot \lambda_{rel.m} \\ \text{else if } 1.4 < \lambda_{rel.m} \\ \quad \| \dfrac{1}{\lambda_{rel.m}^{2}} \end{array} \Bigg| = 1$$

Factor taking into account the reduced bending strength due to lateral buckling. EN 1995-1-1, 6.3.3, clause 6.34.

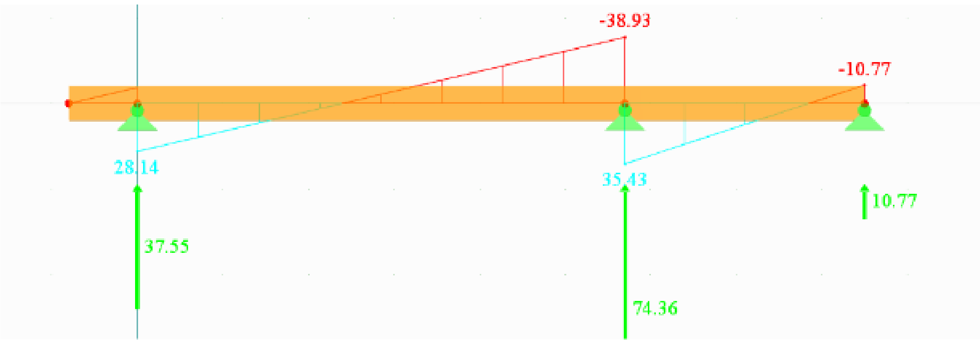
**BENDING STRESS**

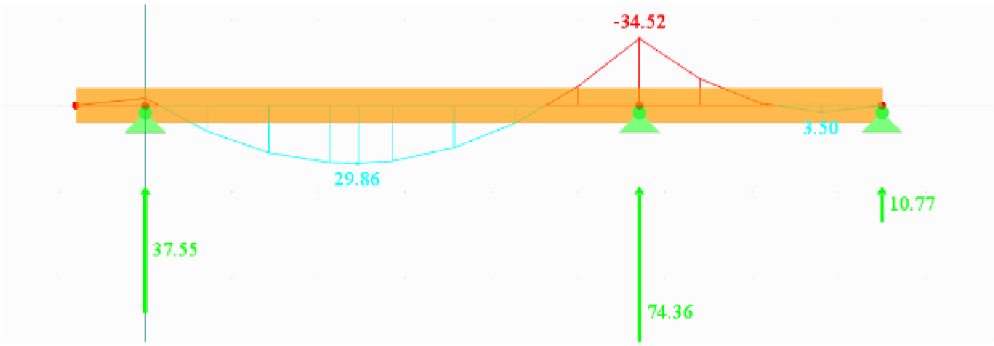
$\sigma_{m.y.d} < k_{crit} \cdot f_{md}$

Condition to be fullfilled



Results generated from RFEM




$V_{Ed} := 38.93\ \text{kN}$ — Shear force in design phase

$M_{Ed} := 34.52\ \text{kN} \cdot \text{m}$ — Bending moment in design phase

$\sigma_{m.y.d} := \dfrac{M_{Ed}}{W_y} = 6.473\ \text{MPa}$ — Design bending stress about the principal y-axis

$f_{myd} := k_{mod} \cdot \dfrac{f_{mk}}{\gamma_M} \cdot k_h \cdot k_{sys} = 12.138\ \text{MPa}$ — Design bending strength about the principal y-axis

$k_{crit} = 1$ — Factor used for lateral buckling

$$\left.\begin{array}{l} \text{if } \sigma_{m.y.d} < k_{crit} \cdot f_{myd} \\ \quad \| \text{“OK”} \\ \text{else} \\ \quad \| \text{“CHECK BENDING STRESS ”} \end{array}\right| = \text{“OK”}$$

$UR := \frac{\sigma_{m.y.d}}{k_{crit} \cdot f_{myd}} = 53.322\%$ Utilization ratio

**SHEAR CONTROL**

$\tau_d \leq f_{v.d}$ Condition to be fullfilled

$V_{Ed} = 38.93\ kN$ Shear force in design phase

$k_{cr} := 0.67$ Modification factor for shear. Recommended value for softwood rectangluar cross section.

$\tau_d := \frac{3}{2} \frac{V_{Ed}}{k_{cr} \cdot b \cdot h} = 1.089\ MPa$ Design shear stress

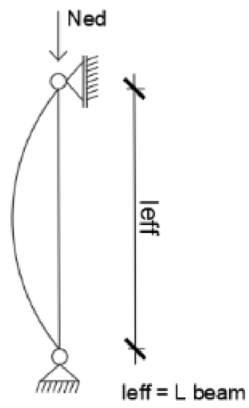
$f_{v.d} := k_{mod} \cdot \frac{f_{v.k}}{\gamma_M} \cdot k_{sys} = 2.462\ MPa$ Shear design strength for the actual condition

```
if τd < fv.d
  ‖ "OK"
else
  ‖ "CHECK SHEAR CONTROL"
```
= "OK"

$UR := \frac{\tau_d}{f_{v.d}} = 44.259\%$ Utilization ratio

## Appendix 5: Design of column 1 most critical to buckling

**COLUMN VERIFICATION 1 MOST CRITICAL TO BUCKLING**




Connection of the column is assumed to be pinned on the bot an bottom of the columns therefore the buckling coefficents about each axis is to be taken as 1. This column is subjected to axial load only.It must be noted that columns do not take any wind load, as lateral forces are carried by the shear walls.

**Geometric propreties of the colum 1**

| | |
|---|---|
| $b := 200\ mm$ | Width of column |
| $h := 200\ mm$ | Depth of column |
| $L_c := 9\ m$ | Length of column |
| $W_y := \frac{b \cdot h^2}{6}$ | Section modulus |
| $l_{eff} := L_c = 9\ m$ | Effective length. |
| $A_c := b \cdot h$ | Area of column |

**Softwood C24 properties**

| | |
|---|---|
| $f_{mk} := 24\ MPa$ | Characteristic bending strength |
| $f_{v.k} := 4\ \frac{N}{mm^2}$ | Design shear strength factor |
| $E_{0.05} := 7.4\ \frac{kN}{mm^2}$ | 5% modulus elasticity paralel to the grain |
| $f_{c.0.k} := 21\ MPa$ | Compression parallel to the grain. |

**ACTIONS**

$N_{Ed} := 37.55\ \text{kN}$ — Normal force in design phase

**LATERAL TORSIONAL BUCKLING CHECK**

$\lambda_{rel.m} < 0.75$ — Condition to be fulfilled for lateral buckling to be neglected

$$\sigma_{mcrit} := \frac{0.78 \cdot b^2}{h \cdot l_{eff}} \cdot E_{0.05} = 128.267\ \text{MPa}$$

Critical bending stress 6.3.3 (6.32)

$$\lambda_{rel.m} := \sqrt{\frac{f_{mk}}{\sigma_{mcrit}}} = 0.433$$

Relative slenderness, EN1995-1-1, 6.3.3(6.30).

$$\begin{array}{l|l} \text{if } \lambda_{rel.m} \le 0.75 & = \text{"NEGLECT LATERAL BUCKLING"} \\ \quad \text{"NEGLECT LATERAL BUCKLING"} & \\ \text{else} & \\ \quad \text{"CHECK LATERAL BUCKLING"} & \end{array}$$

$$k_{crit} := \begin{array}{l|l} \text{if } \lambda_{rel.m} \le 0.75 & = 1 \\ \quad 1 & \\ \text{else if } 0.75 < \lambda_{rel.m} \le 1.4 & \\ \quad 1.56 - 0.75 \cdot \lambda_{rel.m} & \\ \text{else if } 1.4 < \lambda_{rel.m} & \\ \quad \dfrac{1}{\lambda_{rel.m}^{2}} & \end{array}$$

**AXIAL LOAD FAILURE CONTROL**

**Buckling around Y-axis check**

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

$L_{ey} := k_{ey} \cdot L_c = 9\ \text{m}$

Buckling length about Y-axis.

$I_y := \dfrac{b \cdot h^3}{12} = \left(1.33 \cdot 10^8\right)\ \text{mm}^4$

Moment of intertia about Y-direction.

$i_y := \sqrt{\dfrac{I_y}{A_c}} = 57.74\ \text{mm}$

Radius of gyration about Y-direction.

$\lambda_y := \dfrac{L_{ey}}{i_y} = 155.885$

Slenderness ratio.

$\lambda_{rel.y} := \dfrac{\lambda_y}{\pi} \cdot \sqrt{\dfrac{f_{c.0.k}}{E_{0.05}}} = 2.643$

Slenderness ratio corresponding to bending about y-axis(deflection in the z-direction), EN1995-1-1, 6.3.2(6.21)

$\beta_c := 0.2$

Factor for members withing the straightness limits, EN1995-1-1, 6.3.2(6.29).

$k_y := 0.5\ \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + {\lambda_{rel.y}}^2\right) = 4.228$

$k_{c.y} := \dfrac{1}{k_y + \sqrt{{k_y}^2 - {\lambda_{rel.y}}^2}} = 0.133$

Buckling deduction coefficients to consider the buckling effect.

**Buckling around Z-axis check**

$k_{ez} := 1$

Buckling coefficient about Z-axis.

$L_{ez} := k_{ez} \cdot L_c = 9\ \text{m}$

Buckling length about Z-axis.

$I_z := \dfrac{h \cdot b^3}{12} = \left(1.33 \cdot 10^8\right)\ \text{mm}^4$

Moment of intertia about Z-direction.

$$i_z := \sqrt{\frac{I_z}{A_c}} = 57.74\ \text{mm}$$

Radius of gyration about Z-direction.

$$\lambda_z := \frac{L_{ez}}{i_z} = 155.885$$

Slenderness ratio.

$$\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} = 2.643$$

Slenderness ratio corresponding to bending about z-axis(deflection in the y-direction), EN1995-1-1, 6.3.2(6.21)

$$\beta_c := 0.2$$

Factor for members withing the straightness limits, EN1995-1-1, 6.3.2(6.29).

$$k_z := 0.5\ \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + {\lambda_{rel.z}}^2\right) = 4.228$$

$$k_{c.z} := \frac{1}{k_z + \sqrt{{k_z}^2 - {\lambda_{rel.z}}^2}} = 0.133$$

Buckling deduction coefficients to consider the buckling effect.

$$f_{c.0.d} := k_{mod} \cdot k_{sys} \cdot \frac{f_{c.0.k}}{\gamma_M} = 12.923\ \text{MPa}$$

Design compressive strength along the grain.

$$\sigma_{c.0.d} := \frac{N_{Ed}}{A_c} = 0.939\ \text{MPa}$$

Design compressive stress along the grain

When $\lambda_{rel.y}$ and $\lambda_{rel.z}$ are both bigger than 0.3 the following equation must be satisfied.

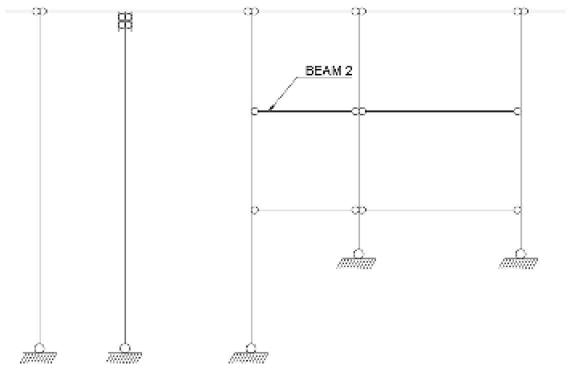
$$\text{if } \frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} \le 1 \wedge \frac{\sigma_{c.0.d}}{k_{c.z} \cdot f_{c.0.d}} \le 1 \;\|\; \text{“OK”} \;\; \text{else} \;\|\; \text{“CHECK”} \quad = \text{“OK”}$$

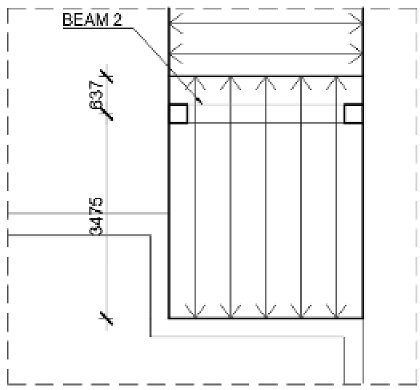
$$UR := \frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} = 0.547$$

Utilization ratio

## Appendix 6: Design of second floor beam

**SIMPLY SUPPORTED BEAM**




**Geometric properties of the beam 2**

$b := 115\ mm$ — Width of beam

$h := 215\ mm$ — Depth of beam

$L_{beam} := 2.9\ m$ — Length of beam

$$W_y := \frac{b \cdot h^2}{6}$$ Section modulus

$$l_1 := \frac{3475\ mm + 637\ mm}{2} = 2.056\ m$$ Tributary length of beam 1

**Softwood C24 properties**

$f_{mk} := 24\ MPa$ — Characteristic bending strength

$$f_{v.k} := 4\ \frac{N}{mm^2}$$ Design shear strength factor

$$E_{0.05} := 7.4\ \frac{kN}{mm^2}$$ 5% modulus elasticity paralel to the grain

**Partial safety factors**

$\gamma_G := 1.15$ — Permanent action

$\gamma_Q := 1.5$ — Variable action

$\gamma_M := 1.3$ — Partial safety factor, EN1995-1-1 Table 2.3

$K_{FI} := 1$ — Consequent class 2, EN1990

**Actions**

$G_{k.1} := 0.45 \ \frac{kN}{m^2}$ — Characteristic dead load of the second floor

$Q_{k.1} := 2 \ \frac{kN}{m^2}$ — Characteristic live load on the second floor

$q_{Ed} := \gamma_G \cdot K_{FI} \cdot G_{k.1} + \gamma_Q \cdot K_{FI} \cdot Q_{k.1} = 3.518 \ \frac{kN}{m^2}$ — EN1990, equation 6.10a

$q_{dist} := q_{Ed} \cdot l_1 = 7.232 \ \frac{kN}{m}$ — Distributed load along the beam in design phase

**Modification factors**

$k_{mod} := 0.8$ — Modification factor, EN1995-1-1, 3.2, table 3.1. Snow is medium-term load.

$k_h := min\left(1.3, \left(\frac{150 \ mm}{h}\right)^{0.2}\right) = 0.931$ — Reference depth in bending, EN1995-1-1, (3.1).

$l_{eff} := L_{beam} + 2 \cdot h = 3.33 \ m$ — Effective length of beam

$k_{sys} := 1$ — System strength factor EN1995-1-1, 6.6, Figure 6.12

**LATERAL TORSIONAL BUCKLING CHECK**

To check stability of the beams it must be ensured that lateral buckling is not an issue, however the connection between the joists and beams provides enough lateral support. Nevertheless when designing the bending stress about y-axis , kcrit is required. Following EN1995-1-1, 6.3.3(6.30), the value of kcrit is depended on the relative slenderness ratio.

$\lambda_{rel.m} < 0.75$

Condition to be fulfilled for lateral buckling to be neglected

$$\sigma_{mcrit} := \frac{0.78 \cdot b^2}{h \cdot l_{eff}} \cdot E_{0.05} = 106.62\ \text{MPa}$$

Critical bending stress 6.3.3 (6.32)

$$\lambda_{rel.m} := \sqrt{\frac{f_{mk}}{\sigma_{mcrit}}} = 0.474$$

Relative slenderness, EN1995-1-1, 6.3.3(6.30).

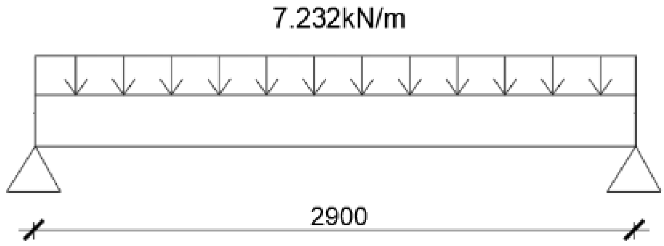
$$\begin{array}{l|l} \text{if } \lambda_{rel.m} \le 0.75 & = \text{“NEGLECT LATERAL BUCKLING”} \\ \quad \| \text{“NEGLECT LATERAL BUCKLING”} & \\ \text{else} & \\ \quad \| \text{“CHECK LATERAL BUCKLING”} & \end{array}$$

$$\begin{array}{l|l} k_{crit} := \text{if } \lambda_{rel.m} \le 0.75 & = 1 \\ \qquad \| 1 & \\ \qquad \text{else if } 0.75 < \lambda_{rel.m} \le 1.4 & \\ \qquad \| 1.56 - 0.75 \cdot \lambda_{rel.m} & \\ \qquad \text{else if } 1.4 < \lambda_{rel.m} & \\ \qquad \| \dfrac{1}{\lambda_{rel.m}^{\ 2}} & \end{array}$$

Factor taking into account the reduced bending strength due to lateral buckling. EN 1995-1-1, 6.3.3, clause 6.34.

**BENDING STRESS**

$\sigma_{m.y.d} < k_{crit} \cdot f_{md}$ Condition to be fulfilled




$V_{Ed} := \frac{q_{dist} \cdot L_{beam}}{2} = 10.486 \text{ kN}$ Shear force in design phase

$M_{Ed} := \frac{q_{dist} \cdot L_{beam}^{2}}{8} = 7.603 \text{ kN} \cdot \text{m}$ Bending moment in design phase

$\sigma_{m.y.d} := \frac{M_{Ed}}{W_y} = 8.581 \text{ MPa}$ Design bending stress about the principal y-axis

$f_{myd} := k_{mod} \cdot \frac{f_{mk}}{\gamma_M} \cdot k_h \cdot k_{sys} = 13.743 \text{ MPa}$ Design bending strength about the principal y-axis

$k_{crit} = 1$

if $\sigma_{m.y.d} < k_{crit} \cdot f_{myd}$
    "OK"
else
    "CHECK BENDING STRESS "
= "OK"

$UR := \frac{\sigma_{m.y.d}}{k_{crit} \cdot f_{myd}} = 62.438\%$ Utilization ratio

**SHEAR CONTROL**

$\tau_d \le f_{v.d}$ — Condition to be fulfilled

$V_{Ed} = 10.486\ \text{kN}$ — Shear force in design phase

$k_{cr} := 0.67$ — Modification factor for shear. Recommended value for softwood rectangluar cross section.

$$\tau_d := \frac{3}{2} \frac{V_{Ed}}{k_{cr} \cdot b \cdot h} = 0.95\ \text{MPa}$$

Design shear stress

$$f_{v.d} := k_{mod} \cdot \frac{f_{v.k}}{\gamma_M} \cdot k_{sys} = 2.462\ \text{MPa}$$

Shear design strength for the actual condition

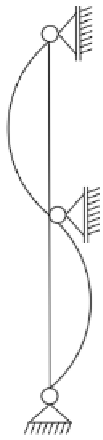
```
if τd < fv.d          = "OK"
  ‖ "OK"
else
  ‖ "CHECK SHEAR CONTROL"
```

$$UR := \frac{\tau_d}{f_{v.d}} = 38.574\%$$

Utilization ratio

**Appendix 7: Design of column 2 combined axial and bending**

**COLUMN VERIFICATION (COLUMN 2)**



Connection of the column is assumed to be pinned on the bot an bottom of the columns, but the second floor strcture also provides stiffnes to the column, therefore the buckling coefficients about each axis is to be taken as 0.5. This column is subjected to axial load and also bending moment in both axis .It must be noted that columns do not take any wind load, as lateral forces are carried by the shear walls.

**Geometric properties of the column 2**

| | |
|---|---|
| $b := 200\ mm$ | Width of column |
| $h := 200\ mm$ | Depth of column |
| $L_c := 5.4\ m$ | Length of column |
| $W_y := \frac{b \cdot h^2}{6}$ | Section modulus |
| $l_{eff} := \frac{L_c}{2} = 2.7\ m$ | Effective length. |
| $A_c := b \cdot h$ | Area of column |

**Softwood C24 properties**

| | |
|---|---|
| $f_{mk} := 24\ MPa$ | Characteristic bending strength |
| $f_{v.k} := 4\ \frac{N}{mm^2}$ | Design shear strength factor |
| $E_{0.05} := 7.4\ \frac{kN}{mm^2}$ | 5% modulus elasticity parallel to the grain |
| $f_{c.0.k} := 21\ MPa$ | Compression parallel to the grain. |

**ACTIONS**

| | |
|---|---|
| $N_{Ed1} := 74.36\ kN$ | Maximum normal force in design phase coming from the roof loads |
| $N_{Ed2} := V_{Ed} = 10.486\ kN$ | Maximum normal force in design phase coming from the second floor throught beam 1 |
| $N_{Ed3} := 3.78\ kN$ | Maximum normal force in design phase coming beam (connected to the vertical rod) |
| $N_{Ed} := N_{Ed1} + N_{Ed2} + N_{Ed3} = 88.626\ kN$ | Maximum normal force transferred to the column |
| $e_{y1} := 100\ mm$ | Eccentricity y axis (beam 1 second floor |
| $e_{y2} := 100\ mm$ | Eccentricity y axis(beam 2 second floor) |
| $M_{Ed.y} := N_{Ed2} \cdot e_{y1} - N_{Ed3} \cdot e_{y2} = 0.671\ m \cdot kN$ | Moment in design phase y axis |

**LATERAL TORSIONAL BUCKLING CHECK**

| | |
|---|---|
| $\lambda_{rel.m} < 0.75$ | Condition to be fulfilled for lateral buckling to be neglected |
| $\sigma_{mcrit} := \dfrac{0.78 \cdot b^2}{h \cdot l_{eff}} \cdot E_{0.05} = 427.556\ MPa$ | Critical bending stress 6.3.3 (6.32) |
| $\lambda_{rel.m} := \sqrt{\dfrac{f_{mk}}{\sigma_{mcrit}}} = 0.237$ | Relative slenderness, EN1995-1-1, 6.3.3(6.30). |

$$\begin{array}{l} \text{if } \lambda_{rel.m} \le 0.75 \\ \quad \| \text{“NEGLECT LATERAL BUCKLING”} \\ \text{else} \\ \quad \| \text{“CHECK LATERAL BUCKLING”} \end{array} \Bigg| = \text{“NEGLECT LATERAL BUCKLING”}$$

$$k_{crit} := \begin{array}{l} \text{if } \lambda_{rel.m} \le 0.75 \\ \quad \| 1 \\ \text{else if } 0.75 < \lambda_{rel.m} \le 1.4 \\ \quad \| 1.56 - 0.75 \cdot \lambda_{rel.m} \\ \text{else if } 1.4 < \lambda_{rel.m} \\ \quad \| \dfrac{1}{\lambda_{rel.m}^{2}} \end{array} \Bigg| = 1$$

**AXIAL LOAD FAILURE CONTROL**

**Buckling around Y-axis check**

$k_{ey} := 0.5$

Buckling coefficient about Y-axis.

$L_{ey} := k_{ey} \cdot L_c = 2.7\ \text{m}$

Buckling length about Y-axis.

$I_y := \dfrac{b \cdot h^3}{12} = \left(1.33 \cdot 10^8\right)\ \text{mm}^4$

Moment of intertia about Y-direction.

$i_y := \sqrt{\dfrac{I_y}{A_c}} = 57.74\ \text{mm}$

Radius of gyration about Y-direction.

$\lambda_y := \dfrac{L_{ey}}{i_y} = 46.765$

Slenderness ratio.

$\lambda_{rel.y} := \dfrac{\lambda_y}{\pi} \cdot \sqrt{\dfrac{f_{c.0.k}}{E_{0.05}}} = 0.793$

Slenderness ratio corresponding to bending about y-axis(deflection in the z-direction), EN1995-1-1, 6.3.2(6.21)

$\beta_c := 0.2$

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

$k_y := 0.5\ \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + \lambda_{rel.y}^{\ 2}\right) = 0.864$

$$k_{c.y} := \frac{1}{k_y + \sqrt{k_y^{\ 2} - \lambda_{rel.y}^{\ 2}}} = 0.829$$

Buckling deduction coefficients to consider the buckling effect.

**Buckling around Z-axis check**

$k_{ez} := 0.5$

Buckling coefficient about Z-axis.

$L_{ez} := k_{ez} \cdot L_c = 2.7\ m$

Buckling length about Z-axis.

$$I_z := \frac{h \cdot b^3}{12} = \left(1.33 \cdot 10^8\right)\ mm^4$$

Moment of intertia about Z-direction.

$$i_z := \sqrt{\frac{I_z}{A_c}} = 57.74\ mm$$

Radius of gyration about Z-direction.

$$\lambda_z := \frac{L_{ez}}{i_z} = 46.765$$

Slenderness ratio.

$$\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} = 0.793$$

Slenderness ratio corresponding to bending about z-axis(deflection in the y-direction), EN1995-1-1, 6.3.2(6.21)

$\beta_c := 0.2$

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

$k_z := 0.5\ \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + \lambda_{rel.z}^{\ 2}\right) = 0.864$

$$k_{c.z} := \frac{1}{k_z + \sqrt{{k_z}^2 - {\lambda_{rel.z}}^2}} = 0.829$$

Buckling deduction coefficients to consider the buckling effect.

$$f_{c.0.d} := k_{mod} \cdot k_{sys} \cdot \frac{f_{c.0.k}}{\gamma_M} = 12.923 \text{ MPa}$$

Design compressive strength along the grain.

$$\sigma_{c.0.d} := \frac{N_{Ed}}{A_c} = 2.216 \text{ MPa}$$

Design compressive stress along the grain

Following the given statement from EC5 with the condition that $\lambda_{rel.y}$ and $\lambda_{rel.z}$ are both bigger than 0.3, and $\lambda_{rel.m}$ is smaller than 0.75 the following equation must be satisfied. $\sigma_{m.z.d}$ is 0.

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

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

$$\sigma_{c.0.d} := \frac{N_{Ed}}{A_c} = 2.216 \text{ MPa}$$

Design compressive stress along the grain

$$\sigma_{m.y.d} := \frac{M_{Ed}}{W_y} = 5.702 \text{ MPa}$$

Design bending stress about the principal y-axis

$$f_{m.y.d} := k_{mod} \cdot k_{sys} \cdot k_h \cdot \frac{f_{mk}}{\gamma_M} = 13.743 \text{ MPa}$$

Design bending strength about the principal y-axis

$$k_m := 0.7$$

$$\left\| \begin{array}{l} \text{if } \dfrac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + \dfrac{\sigma_{m.y.d}}{f_{m.y.d}} < 1 \\ \quad \| \text{“OK”} \\ \text{else} \\ \quad \| \text{“CHECK”} \end{array} \right\| = \text{“OK”}$$

$$\left\| \begin{array}{l} \textbf{if } \dfrac{\sigma_{c.0.d}}{k_{c.z} \cdot f_{c.0.d}} + k_m \dfrac{\sigma_{m.y.d}}{f_{m.y.d}} < 1 \\ \quad \| \text{“OK”} \\ \textbf{else} \\ \quad \| \text{“CHECK”} \end{array} \right\| = \text{“OK”}$$

$$UR := \frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + \frac{\sigma_{m.y.d}}{f_{m.y.d}} = 0.622$$ Utilization ratio

$$UR := \frac{\sigma_{c.0.d}}{k_{c.z} \cdot f_{c.0.d}} + k_m \frac{\sigma_{m.y.d}}{f_{m.y.d}} = 0.497$$ Utilization ratio

## Appendix 8: Design of pad foundation

**PAD FOUDATION**

When determining the effective foot area of the foundation eccentricity of loads plays in important role. When making checks for column number 2, 2 forces are applied with 100mm eccentricity, however moment created is considered as a local moment. Their magnitute could be considered negligible and the maximum normal force applied on the column is presented as the summation on there two forces together with the main one coming form the roof.

| | |
|---|---|
| $V_{ED} := 88.626 \text{ kN}$ | Load magnitute transfered from the structure to the foundation |
| $G_k := V_{ED} = 88.626 \text{ kN}$ | |
| $P_g := 250 \dfrac{\text{kN}}{\text{m}^2}$ | Soil capacity Sand moraine |
| $A := 1.1 \cdot \dfrac{G_k}{P_g} = 0.39 \text{ m}^2$ | Base area of the footing |

Minimum dimension of footing

$\sqrt{A} = 624.463 \text{ mm}$

| | |
|---|---|
| $B := 0.8 \text{ m}$ | Width of the base |
| $L := B$ | Length of the base |
| $d_c := 0.25 \cdot m$ | Height of the base |
| $h := d_c = 0.25 \text{ m}$ | |
| $A_{prov} := B \cdot L = 0.64 \text{ m}^2$ | Provided area |
| $D := 0.5 \cdot m$ | |
| $e_x := 9 \cdot m + D = 9.5 \text{ m}$ | Depth of establishment |
| $SA := 0.2 \cdot m$ | Side dimension of the concrete part of the column |
| $SB := SA$ | Side dimension of the concrete part of the column |

$\phi := 32 \cdot deg$

Powerful friction angle,
Estimated ground friction angle

$\gamma := 18 \cdot \frac{kN}{m^3}$

Volume weight of land below establishment level

$\gamma_1 := 20 \cdot \frac{kN}{m^3}$

Volumetric weight of the land above the establishment level

$Z_w := 2.5 \cdot m$

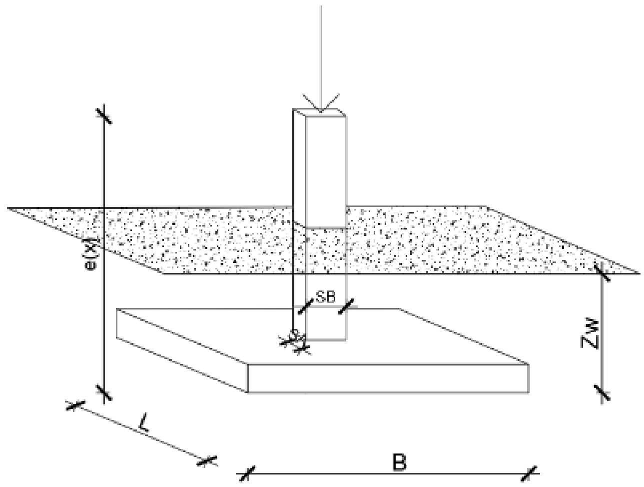
Groundwater level

$\gamma_2 := 11 \cdot \frac{kN}{m^3}$

Effective bulk density below groundwater level

$\gamma_C := 25 \cdot \frac{kN}{m^3}$

Concrete density




**Ground bearing capacity inspection**

Assume that the soil type is friction.

$q_a := 1.15 \cdot L \cdot B \cdot d_c \cdot \gamma_C = 4.6\ kN$

Foot plate tare weight

$q_p := 1.15 \cdot \left(SA \cdot SB \cdot 2.5\ m \cdot \gamma_C\right) = 2.875\ kN$

Own weight of the pillar

$q_m := 1.15 \cdot L \cdot B \cdot \left(D - d_c\right) \cdot \gamma_1 = 3.68\ kN$

The weight of the land on the foundations

$V_{ED} := V_{ED} + q_a + q_p + q_m = 99.781\ kN$ Force applied to the foundation

$e_B := 0\ mm$ Eccentricity at width of the pad

$e_L := 0\ m$ Eccentricity at length of the pad

Effective foot area

$L_{ef} := L - 2 \cdot e_L = 800\ mm$ Effective length

$B_{ef} := B - 2 \cdot e_B = 800\ mm$ Effective width

$A_{ef} := L_{ef} \cdot B_{ef} = 0.64\ m^2$ Effective Area

```
if D < 2.5 • Bef    = "Ok"
  ‖ "Ok"
else
  ‖ "NOT OK"
```

OK

```
if L < 2.5 • Lef    = "Ok"
  ‖ "Ok"
else
  ‖ "NOT OK"
```

OK

$N_q := e^{\pi \cdot \tan(\phi)} \cdot \left( \tan\left(45 \cdot deg + \frac{\phi}{2}\right)^2 \right) = 23.177$ Overburden factor

$N_c := \left(N_q - 1\right) \cdot \cot(\phi) = 35.49$ Cohesion factor

$N_\gamma := 2 \cdot \left(N_q - 1\right) \cdot \tan(\phi) = 27.715$ Body weight factor

The level of the base of the foundations is horizontal

$\alpha := 0 \cdot deg$

$b_q := (1 - \alpha \cdot \tan(\phi))^2 = 1$

Coefficient depended on the slope of the foudation base

$b_\gamma := b_q = 1$

Coefficient depended on the slope of the foudation base

$$b_c := b_q - \frac{(1 - b_q)}{(N_c \cdot \tan(\phi))} = 1$$

Coefficient depended on the slope of the foudation base

$$s_q := 1 + \left(\frac{B_{ef}}{L_{ef}}\right) \cdot \sin(\phi) = 1.53$$

Coefficient depended on the shape of the foundation

$$s_\gamma := 1 - 0.3 \cdot \left(\frac{B_{ef}}{L_{ef}}\right) = 0.7$$

Coefficient depended on the shape of the foundation

$$s_c := \frac{(s_q \cdot N_q - 1)}{(N_q - 1)} = 1.554$$

Coefficient depended on the shape of the foundation

$$m_B := \frac{\left(2 + \left(\frac{B_{ef}}{L_{ef}}\right)\right)}{\left(1 + \left(\frac{B_{ef}}{L_{ef}}\right)\right)} = 1.5$$

$$m_L := \frac{\left(2 + \left(\frac{L_{ef}}{B_{ef}}\right)\right)}{\left(1 + \left(\frac{L_{ef}}{B_{ef}}\right)\right)} = 1.5$$

$\theta := 90 \cdot deg$ — Angle of the horizontal forces applied to the foundation.

$H_{Ed} := 0\ kN$ — Lateral load maginute (not present)

$$m_\theta := m_L \cdot (\cos(\theta))^2 + m_B \cdot (\sin(\theta))^2 = 1.5$$

$c_d := 0$ — Cohesion factor.

$$i_q := \left(1 - \frac{H_{Ed}}{(V_{ED} + 0)}\right)^{m_\theta}$$

$$i_c := 0.8 - \frac{(1 - i_q)}{N_c \cdot \tan(\phi)} = 0.8$$

$$i_\gamma := \left(1 - \frac{H_{Ed}}{(V_{ED} + 0)}\right)^{m_\theta + 1} = 1$$

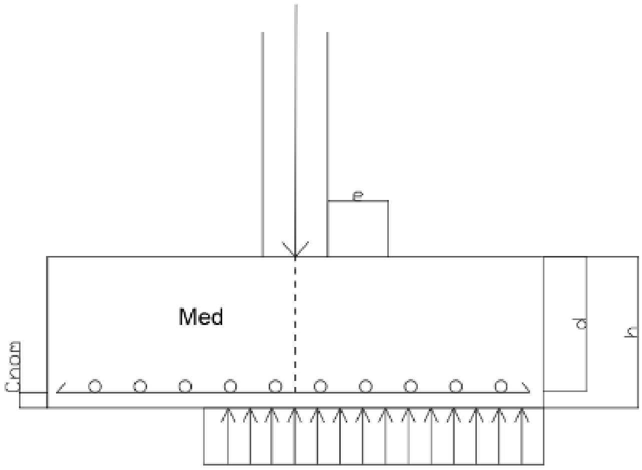
$$q' := \gamma_1 \cdot D = 10\ \frac{kN}{m^2}$$

$\gamma_R := 1.55$

$$q_{ult} := A_{ef} \cdot \frac{(0 + q' \cdot N_q \cdot b_q \cdot s_q \cdot i_q + 0.5 \cdot \gamma_2 \cdot B_{ef} \cdot N_\gamma \cdot b_\gamma \cdot s_\gamma \cdot i_\gamma)}{\gamma_R} = 181.656\ kN$$

$$\frac{V_{ED}}{q_{ult}} = 54.928\%$$

**Structural engineering dimensioning (bending reinforcement and puncture resistance)**




$h = 0.25\ m$ Pillar foot thickness

$f_{ck} := 25\ MPa$ Strength of concrete C25/30

$f_{yk} := 500\ MPa$ Reinforcement B500B

$\gamma_s := 1.15$ Partial safety factor for steel

$E_s := 200\ GPa$ Coefficient of elasticity of steel

$\alpha_{cc} := 0.85$

$\gamma_c := 1.5$

$f_{cd} := \alpha_{cc} \cdot \frac{f_{ck}}{\gamma_c} = 14.167\ MPa$ The design compressive strength of concrete

$f_{ctm} := 2.6\ MPa$ Mean tensile strength EN 1992-1-1:2004, Table 3.2

$f_{ctk} := 1.8\ MPa$ Characteristic tensile strength of concrete at 28 days EN 1992-1-1:2004, Table 3.2

$f_{ctd} := \frac{f_{ctk}}{\gamma_c} = 1.2\ MPa$ The design tensile strength of concrete

$f_{yd} := \frac{f_{yk}}{\gamma_s} = 434.783\ \text{MPa}$

Design value for the yield strength of rebar

**Concrete cover against the ground**

$\varphi_s := 10\ \text{mm}$

Diameter of rebar

$c_{min.b} := \varphi_s$

Minimum cover to meet the bond requirement.

$c_{min.dur} := 25\ \text{mm}$

Minimum cover stipulated for the environmental conditions. EN 1992-1-1:2004 (4.4.1.2)

$c_{min} := \max\left(c_{min.b}, c_{min.dur}, 10\ \text{mm}\right) = 25\ mm$

Exposure class XC2 EN 1992-1-1:2004 Table 4.1

$\Delta c_{dev} := 40 \cdot \text{mm}$

$c_{nom} := c_{min} + \Delta c_{dev} = 65\ mm$

Nominal cover EN 1992-1-1:2004 (4.4.1.1)

$h = 0.25\ \text{m}$

Cross section height

$d := h - \frac{\varphi_s}{2} - c_{nom} = 180\ mm$

Effective depth

**Bottom Pressure**

$P_{Ed} := \frac{V_{ED}}{L_{ef} \cdot B_{ef}} = 155.908\ \frac{kN}{m^2}$

Bearing pressure in design phase

$L = 0.8\ \text{m}$

Length of the footing

$B = 0.8\ \text{m}$

Width of the footing

$L_{ef} = 0.8\ \text{m}$

Effective length of the footing

$B_{ef} = 0.8\ \text{m}$

Effective width of the footing

**Bending moments**

$$L_x := min\left(\frac{B - SB}{2}, B_{ef}\right) = 0.3\ \text{m}$$

Length of compressed area

$$L_y := min\left(\frac{L - SA}{2}, L_{ef}\right) = 0.3\ \text{m}$$

Length of compressed area

Moment created from the second floor

$$M_{Ed.y} := P_{Ed} \cdot L_{ef} \cdot \frac{L_x^{\ 2}}{2} = 5.613\ kN \cdot m$$

The fracture limit moment about the y-axis

$$M_{Ed.x} := P_{Ed} \cdot B_{ef} \cdot \frac{L_y^{\ 2}}{2} = 5.613\ kN \cdot m$$

The fracture limit moment about the x-axis

Bending reinforcement

$$\mu := \frac{M_{Ed.y}}{L_{ef} \cdot d^2 \cdot f_{cd}} = 0.015$$

$$\beta := 1 - \sqrt{1 - 2 \cdot \mu} = 0.015$$

Limit value for relative compression height

$$\beta_{bd} := 0.493$$

Equilibrium reinforced cross section βbd

.

Condition of the height of the compression zone

$$\left.\begin{array}{l} \text{if } \beta < \beta_{bd} \\ \quad \Vert \text{“OK”} \\ \text{else} \\ \quad \Vert \text{“NOT OK”} \end{array}\right| = \text{“OK”}$$

$A_{s.req} := \beta \cdot L_{ef} \cdot d \cdot \frac{f_{cd}}{f_{yd}} = 72.274\ mm^2$ Required area of reinforcement

Maximum and minimum reinforcement

$A_{s.min} := 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot L_{ef} \cdot d = 194.688\ mm^2$ Minumun required area of reinforcement

$A_c := d_c \cdot L_{ef} = 0.2\ m^2$ Area of concrete

$A_{s.max} := 0.06 \cdot A_c = \left(1.2 \cdot 10^4\right)\ mm^2$ Maximum allowable area of reinforcement

$A_{s.tB} := \frac{\pi \cdot \varphi_s^2}{4} = 78.54\ mm^2$ Area of one rebar

$n := \frac{A_{s.min}}{A_{s.tB}} = 2.479$ Number of the required rebars

**However, select 4 10 mm rods in both directions.**

Anchoring length (preliminary calculation)

$\eta_1 := 0.7$ Coefficient depending on the infection conditions

$\eta_2 := 1$ Coefficient depending on the infection conditions

$f_{bd} := 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} = 1.89\ \frac{N}{mm^2}$ Nominal adhesion strength

$\varphi_s := 10\ mm$ Diamater of rebar

$L_{b.rqd} := \frac{\varphi_s}{4} \cdot \frac{f_{yd}}{f_{bd}} = 575.109\ mm$ Required anchorage length

$\alpha_1 := 0.7$

$L_{b.d} := \alpha_1 \cdot L_{b.rqd} = 402.576\ mm$

Section to be bent upwards

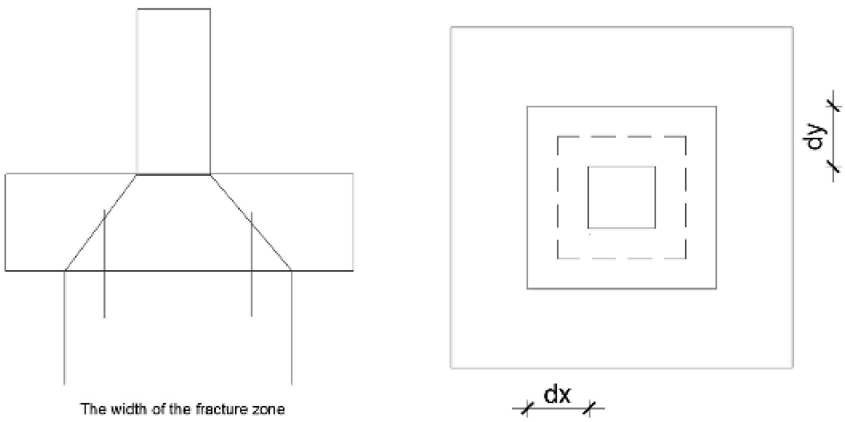
$L_x = 300\ mm$ Odd anchoring length in direction x

$c_{nom} = 65\ mm$ Nominal cover

$L_{b.d} = 402.576\ \text{mm}$ Anchorge length

$L_{bd.req} := \max\left(3 \cdot \varphi_s, L_{b.d} - \left(L_x - c_{nom} - \frac{\varphi_s}{2}\right)\right) = 172.576\ mm$ Required anchorage length

**Punching shear resistance.**




$u := 2 \cdot (d + SB) + 2 \cdot (d + SA) = 1.52\ \text{m}$ Perimeter control

$$\frac{V_{ED}}{A_{prov}} = 0.156\ \text{MPa}$$

Suppose the pressure affects the entire area outside the circuit (assumption & simplification on the safe side)

$$P_{Ed} = 155.908\ \frac{kN}{m^2}$$

$$A := B \cdot L - (d + SB) \cdot (d + SA) = \left(4.956 \cdot 10^5\right)\ mm^2$$ Area within perimeter

$$V_{Ed} := A \cdot P_{Ed} = 77.268\ kN$$

Puncture resistance when not taking into account the effect of reinforcing shear strength

$$C_{Rd.c} := \frac{0.18}{\gamma_c} \cdot MPa = 0.12\ MPa$$

$$k := min\left(2, 1 + \sqrt{\frac{200 \cdot mm}{\varphi_s}}\right) = 2$$

$$A_{sl} := 8 \cdot \frac{\pi \cdot {\varphi_s}^2}{4} = 628.319\ mm^2$$ Four rebars pass through a cutting circuit (in one direction only)

$$\rho_1 := \frac{A_{sl}}{u \cdot d} = 0.002$$

$$V_{Rd.c} := C_{Rd.c} \cdot k \cdot \left(100 \cdot \rho_1 \cdot \frac{f_{ck}}{MPa}\right)^{\frac{1}{3}} \cdot u \cdot d = 117.579\ kN$$

$$UR := \frac{V_{Ed}}{V_{Rd.c}} = 65.716\%$$ Utilization ratio

## Appendix 9: Design of strip foundation

**STRIP FOUNDATION DESIGN**

C25/30 — Concrete class.

**Soil capacity**

Sand moraine — Soil capacity.

$P_g := 250\ \frac{kN}{m^2}$ — Ground pressure.

Soil capacity was assumed to be around 250 kN/m2 conservatively as yet there is not information regarding the soil capacity.

**Characteristic loads**

**Dead load**

$b_{span} := 12.05\ m$ — Longest span of the building.

$T_{length} := \frac{b_{span}}{2} = 6.03\ m$ — Tributary length.

$g_{D1.k} := 0.683\ \frac{kN}{m^2}$ — Characteristic roof.

$g_{D2.k} := 0.2\ \frac{kN}{m^2}$ — Characteristic dead load at the upper eaves.

$g_{D.k} := g_{D1.k} + g_{D2.k} = 0.88\ \frac{kN}{m^2}$ — Characteristic dead load in $\frac{kN}{m^2}$.

$g_{D.k} := \left(g_{D1.k} + g_{D2.k}\right) \cdot T_{length} = 5.32\ \frac{kN}{m}$ — Characteristic dead load in $\frac{kN}{m}$.

**Snow load**

$e_{length} := 800\ mm$ — Eaves length.

$q_{S.k} := 2.4\ \frac{kN}{m^2}$ — Characteristic snow load in $\frac{kN}{m^2}$.

$q_{S.k} := q_{S.k} \cdot (T_{length} + e_{length}) = 16.38\ \frac{kN}{m}$ Characteristic dead load in $\frac{kN}{m}$.

**Self weight of the floor**

$t_{wall} := 420\ mm$ Thickness of wall.

$t_{floor} := 200\ mm$ Thickens of concrete floor

$\gamma_c := 25\ \frac{kN}{m^3}$

$g_{f.k} := t_{floor} \cdot \gamma_c = 5\ \frac{kN}{m^2}$ Characteristic self weight of floor in $\frac{kN}{m^2}$.

$g_{f.k} := g_{f.k} \cdot \left(T_{length} - \frac{2 \cdot t_{wall}}{2}\right) = 28.03\ \frac{kN}{m}$ Characteristic self weight of floor $\frac{kN}{m}$.

**Live load**

$q_{L.k} := 2\ \frac{kN}{m^2}$ Characteristic live load $\frac{kN}{m^2}$.

$q_{L.k} := q_{L.k} \cdot T_{length} = 12.05\ \frac{kN}{m}$ Characteristic live load $\frac{kN}{m}$.

**Inner wall weight**

$t_{pwall} := 130\ mm$ Thickness of partition wall.

$h_{pwall} := 2.5\ m$ Height of partition wall.

$q_{iw.k} := \gamma_c \cdot t_{pwall} \cdot h_{pwall} = 8.13\ \frac{kN}{m}$ Characteristic load of partition walls $\frac{kN}{m^2}$.

**Inner wall weight**

C24 Grade of timber material.

$\gamma_{wood} := 4.2\ \frac{kN}{m^3}$ Density of wood.

$t_{upperwall} := 173\ mm$ Thickness of partition wall.

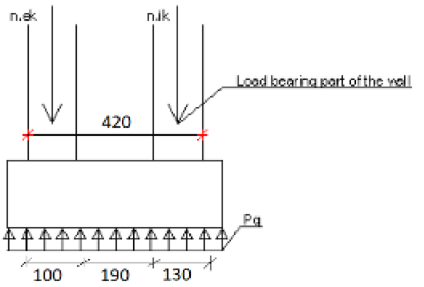
$h_{upperwall} := 2.5 \ m$ Height of partition wall.

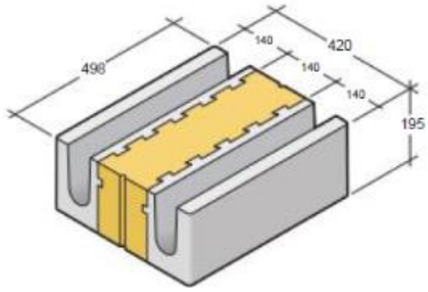
$n_{upperwall} := \gamma_{wood} \cdot t_{upperwall} \cdot h_{upperwall} = 1.82 \ \frac{kN}{m}$ Characteristic load of partition walls $\frac{kN}{m^2}$.

**Total load in 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**




$t_{ek} := 100 \ mm$ Thickness of external wall.

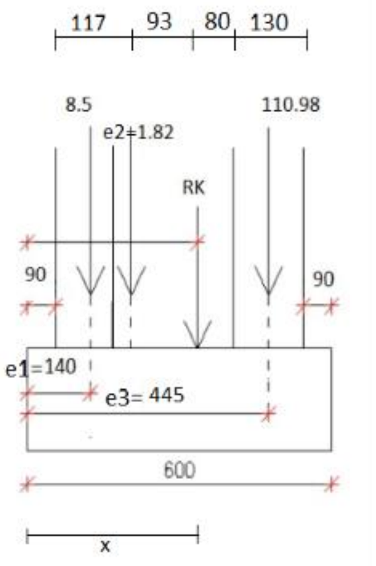
$h_{ek} := 3.4 \ m$ Height of the wall.

$$n_{ek} := \gamma_c \cdot t_{ek} \cdot h_{ek} = 8.5 \ \frac{kN}{m}$$

**Preliminary selection of footing width**

$$R_k := n_{ik} + n_{ek} + n_{upperwall} = 120.29 \ \frac{kN}{m}$$

Assume a width of 600mm




$e_1 := 140\ mm$ Eccentricity of the force.

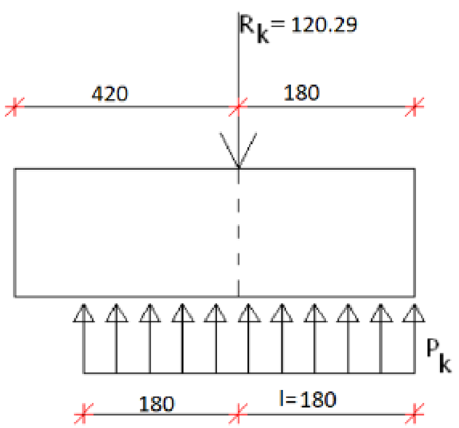
$e_2 := 117\ mm + 90\ mm = 207\ mm$ Eccentricity of the force.

$e_3 := 445\ mm$ Eccentricity of the force.

$$x := \frac{n_{ek} \cdot e_1 + n_{upperwall} \cdot e_2 + n_{ik} \cdot e_3}{R_k} = 419.85\ mm$$

$x := 420\ mm$

**Ground pressure**



$l := 180\ mm$

$p_k := 2 \cdot l = 360\ mm$ Ground pressure.

$$P_k := \frac{R_k}{p_k} = 334.14\ \frac{kN}{m^2}$$

$$\begin{array}{l} \text{if } P_k > P_g \\ \quad \| \text{“move footing adequately to the left”} \\ \text{else} \\ \quad \| \text{“okay!”} \end{array} = \text{“move footing adequately to the left”}$$

**Defining new x distance**

$e := 30\ mm$ Distance to be moved.

$e_{1n} := e_1 - e = 110\ mm$ New eccentricity of the force.

$e_{2n} := e_2 - e = 177\ mm$ New eccentricity of the force.

$e_{3n} := e_3 - e = 415\ mm$ New eccentricity of the force.

$$x := \frac{n_{ek} \cdot e_{1n} + n_{upperwall} \cdot e_{2n} + n_{ik} \cdot e_{3n}}{R_k} = 389.85\ mm$$

$x := 390\ mm$

**Ground pressure**

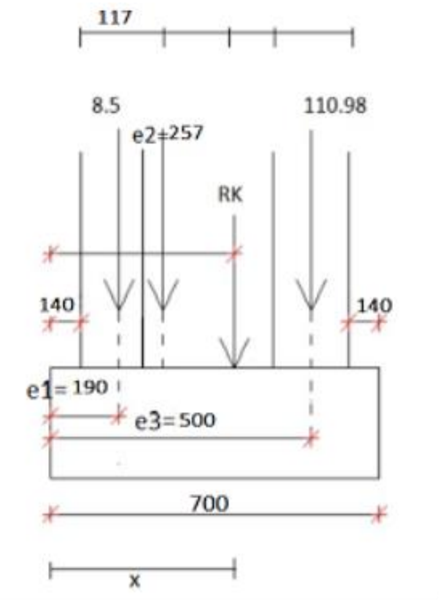
$l := 180\ mm + e = 210\ mm$

$p_k := 2 \cdot l = 420\ mm$ Ground pressure.

$$P_k := \frac{R_k}{p_k} = 286.41\ \frac{kN}{m^2}$$

if $P_k > P_g$   $=$ "move footing to the right"
  "move footing to the right"
else
  $\frac{P_k}{P_g}$

**It can be seen that footing with width of 600mm would not work, therefore checking with 700mm width is done as follows:**



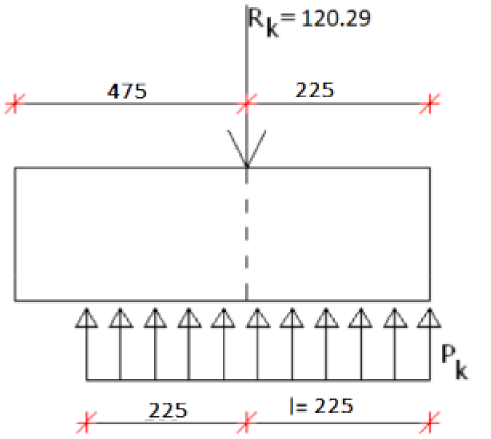

$e_1 := 140\ mm + 50\ mm = 190\ mm$   Eccentricity of the force.

$e_2 := 117\ mm + 140\ mm = 257\ mm$   Eccentricity of the force.

$e_3 := 700\ mm - 140\ mm - 120\ \frac{mm}{2} = 500\ mm$   Eccentricity of the force.

$$x := \frac{n_{ek} \cdot e_1 + n_{upperwall} \cdot e_2 + n_{ik} \cdot e_3}{R_k} = 474.43\ mm$$

**Ground pressure**




$l := 225\ mm$

$p_k := 2 \cdot l = 450\ mm$ Ground pressure.

$$P_k := \frac{R_k}{p_k} = 267.31\ \frac{kN}{m^2}$$

if $P_k > P_g$
‖ "move footing adequately to the left"
else
‖ "okay!"

= "move footing adequately to the left"

**Defining new x distance**

$e := 60\ mm$ Distance to be moved.

$e_{1n} := e_1 - e = 130\ mm$ New eccentricity of the force.

$e_{2n} := e_2 - e = 197\ mm$ New eccentricity of the force.

$e_{3n} := e_3 - e = 440\ mm$ New eccentricity of the force.

$$x := \frac{n_{ek} \cdot e_{1n} + n_{upperwall} \cdot e_{2n} + n_{ik} \cdot e_{3n}}{R_k} = 414.43\ mm$$

$x := 390\ mm$

**Ground pressure**

$l := 225\ mm + e = 285\ mm$

$p_k := 2 \cdot l = 570\ mm$ Ground pressure.

$P_k := \frac{R_k}{p_k} = 211.04\ \frac{kN}{m^2}$

$$\left.\begin{array}{l} \textbf{if } P_k > P_g \\ \quad \| \text{"move footing to the right"} \\ \textbf{else} \\ \quad \| \frac{P_k}{P_g} \end{array}\right| = 84.42\%$$

Okay!

$N_{Ed} := 1.15 \cdot (n_{ek} + n_{upperwall}) = 11.86\ \frac{kN}{m}$

$N_{id} := 1.15 \cdot (g_{D.k} + 2 \cdot g_{f.k} + q_{iw.k}) + 1.5 \cdot (q_{L.k} + 0.7 \cdot q_{L.k} + 0.7 \cdot q_{S.k}) = 127.85\ \frac{kN}{m}$

$R_d := N_{id} + N_{Ed} = 139.71\ \frac{kN}{m}$

**Ground pressure**

$P_k := 2 \cdot l = 570\ mm$

$\delta_{gd} := \frac{R_d}{P_k} = 245.1\ \frac{kN}{m^2}$

$\alpha_{ct} := 0.85$ National Annex of Finland.

$f_{c.t.k} := 1.8\ MPa$

$\psi_c := 1.5$ Partial safety factor of concrete.

$f_{ctd} := \alpha_{ct} \cdot \frac{f_{c.t.k}}{\psi_c} = 1.02\ MPa$

$a := 140\ mm + e = 200\ mm$

Guess Values

$h_f := 200\ m$ Guess value.

Constraints

$$\frac{0.85 \cdot h_f}{a} - \sqrt{\frac{3 \cdot \delta_{gd}}{f_{ctd}}} = 0$$

Solver

$find\left(h_f\right) = 199.78\ mm$

$h_f := 250\ mm$

Footing dimensions

**Reinforcement**

C30/37 Concrete class.

$f_{ctm} := 2.9\ MPa$ Mean tensile strength.

$b := 1000\ mm$ Designing for 1 meter strip footing.

$d := 250\ mm$ Depth of footing.

$t := 420\ mm$

$f_{yk} := 500\ MPa$ Steel characteristic yield strength.

$$A_{s.min} := min\left(\frac{0.26 \cdot f_{ctm} \cdot b \cdot t \cdot d}{f_{yk} \cdot m}, \frac{0.0013 \cdot d \cdot t \cdot d}{m}\right) = 34.13\ mm^2$$

Reinforced concrete mesh 5/150 is used for the design.

**Appendix 10: ComSlab Report**

# **Project:** Project

**Updated:** 2022-05-19 10:58 (GMT)

**Created:** 2022-04-20 13:26 (GMT)

**Customer:**

**National annex:** Finnish NA

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**Contact person:** Igert Loka

**Engineer's contact info:** Visamaentie 23

**Email:** igert1800@student.hamk.fi

**Telephone number:** 0414987705

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| ID | Structural part | Updated | Created |
|---|---|---|---|
| 1 | Part | 2022-05-12 19:46 (GMT) | 2022-04-20 13:26 (GMT) |

COMSLAB

Project

2022-05-19 10:58 (GMT)

RESULTS ARE VALID ONLY FOR RUUKKI COMPOSITE SHEETS

# Structural part: Part

**Updated:** 2022-05-12 19:46 (GMT) **Version:** 1.0.11 (2022-05-04)

**Created:** 2022-04-20 13:26 (GMT)

**Reliability class:** RC2

**Deflection limit:** L/100 (according to Eurocode)

**Exposure class:** XC3, **Intended service life:** L100, **Allowed crack width:** 0.30 mm

**Concrete strength class:** C25/30

**Reinforcement grade:** A500HW / B500B

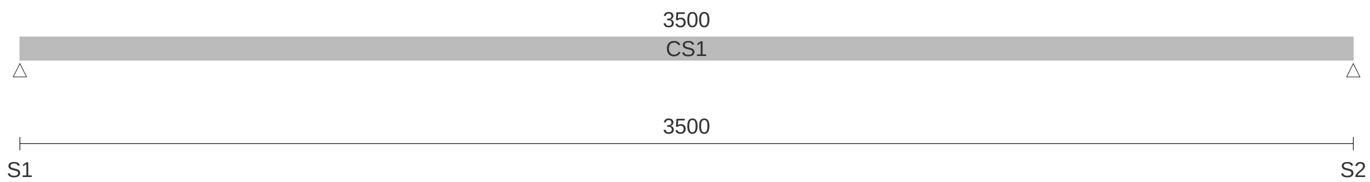
**Concrete density:** 25.0 kN/$m^3$,

**Fire resistance class:** R0

**Concrete top cover:** 25 mm, **Concrete bottom cover:** 47 mm

Insufficient amount of transverse reinforcement. According to Section 9.2.1(4) of EN 1994-1-1, the amount of reinforcement should be not less than 80 $mm^2$/m.

## Structural model



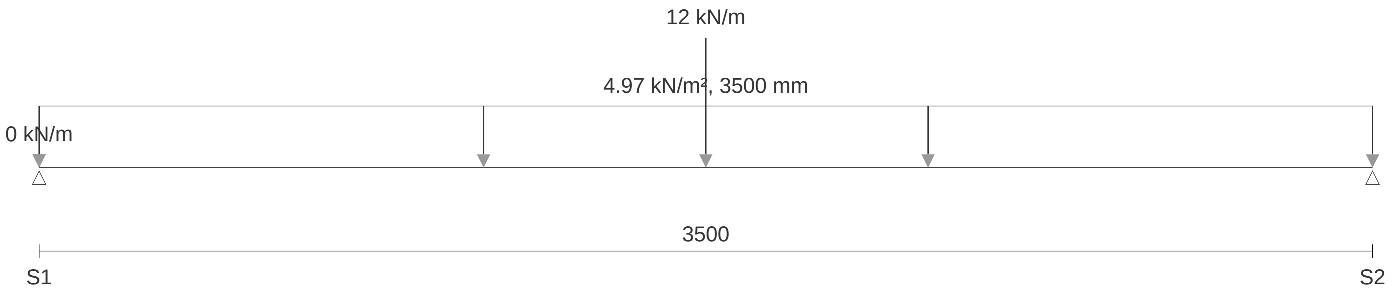

## Spans

| Span | Length | Slab thickness | Sheet | Sheet thickness |
|---|---|---|---|---|
| | [mm] | [mm] | | [mm] |
| CS1 | 3500 | 200 | CS48-36-750 Zn | 0.7 |

## Supports

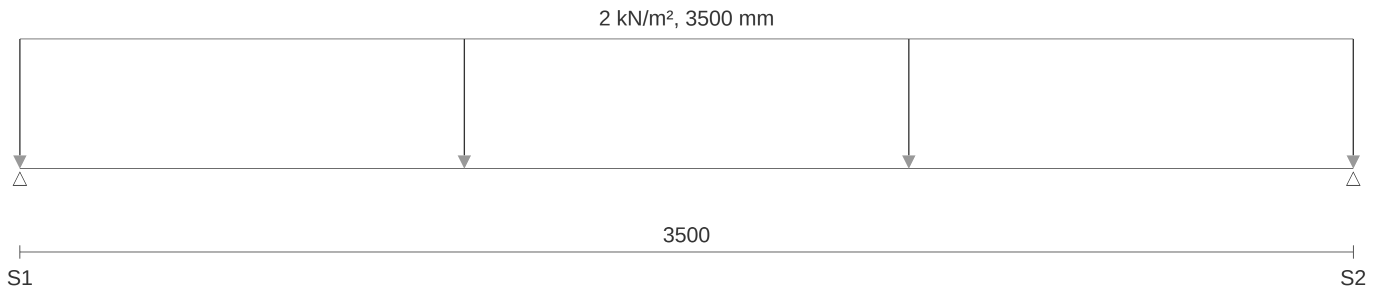
| Support | Width |
|---|---|
| | [mm] |
| S1 | 110 |
| S2 | 110 |

## Dead load




## Live load

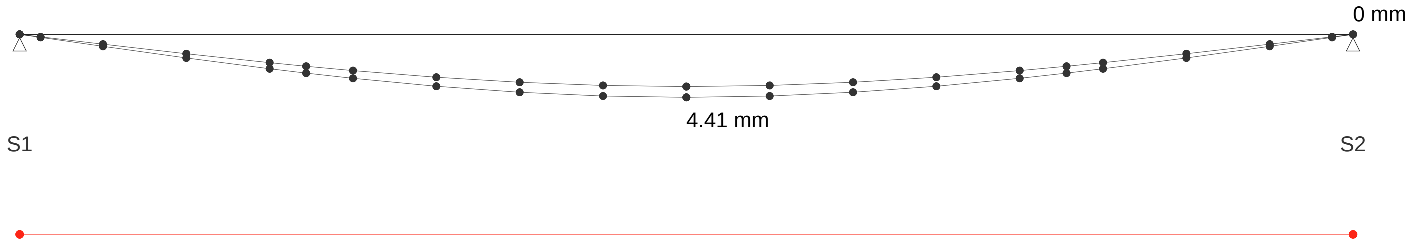
**Load category:** A: areas in residental buildings



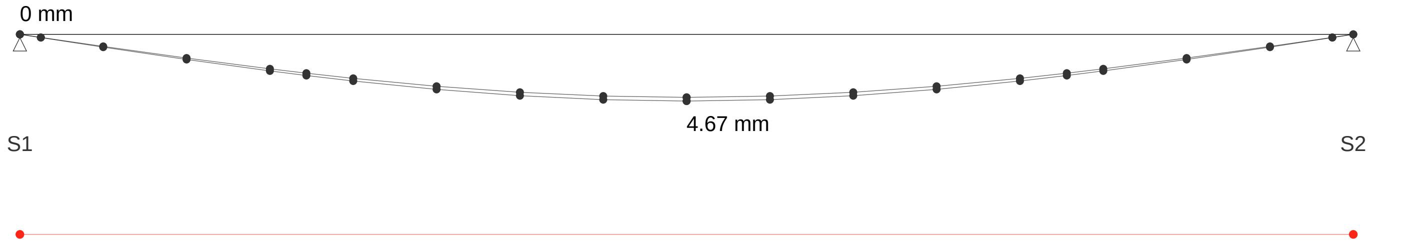

## Span utilization ratios

| | $M_{pos}$ [kNm/m] | $M_{neg}$ [kNm/m] | $V_V$ [kN/m] | $D_{ST}$ [mm] | $D_{LT}$ [mm] | $W_{ST}$ [mm] | $W_{LT}$ [mm] |
|---|---|---|---|---|---|---|---|
| CS1 | 25.4 / 62.6<br>41.0 % | 0.0 / 0.0<br>0.0 % | 21.6 / 79.3<br>27.0 % | 4.4 / 14.0<br>31.0 % | 4.7 / 14.0<br>33.0 % | 0.0 / 0.0<br>0.0 % | 0.0 / 0.0<br>0.0 % |

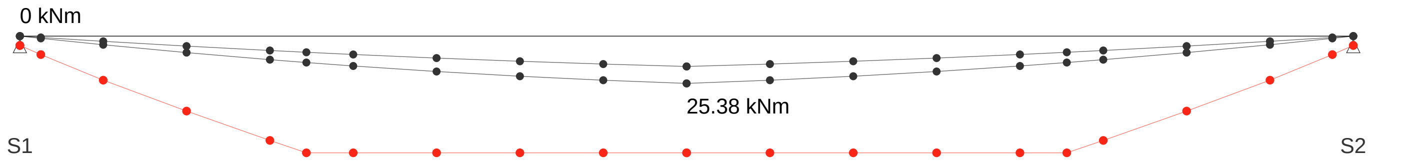
## Deflection, short term



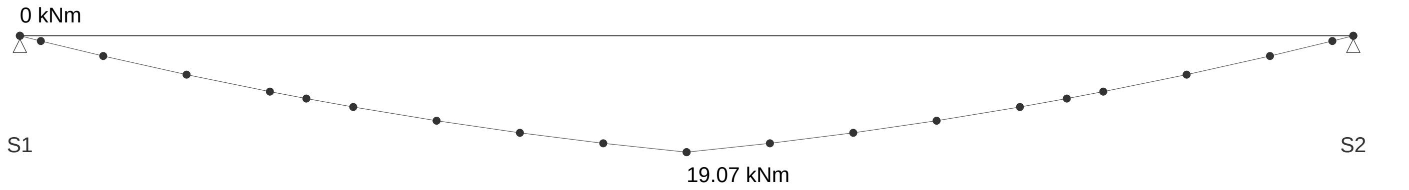

## Deflection, long term



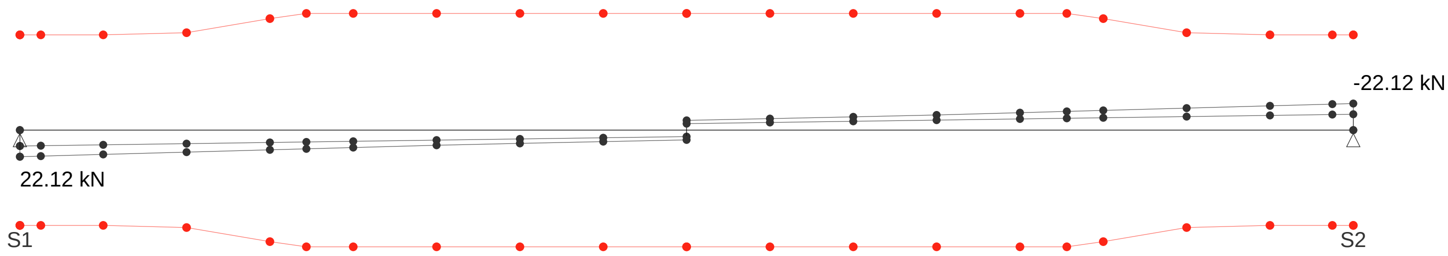

## Bending moment




## Fire situation



## Shear force




## Support reactions

| Support | Min | Max |
|---|---|---|
| | [kN] | [kN] |
| S1 | 16.61 | 27.8 |
| S2 | 13.22 | 22.12 |

## Reinforcements


### Span reinforcements

-

### Support reinforcements

-

### Transverse reinforcement

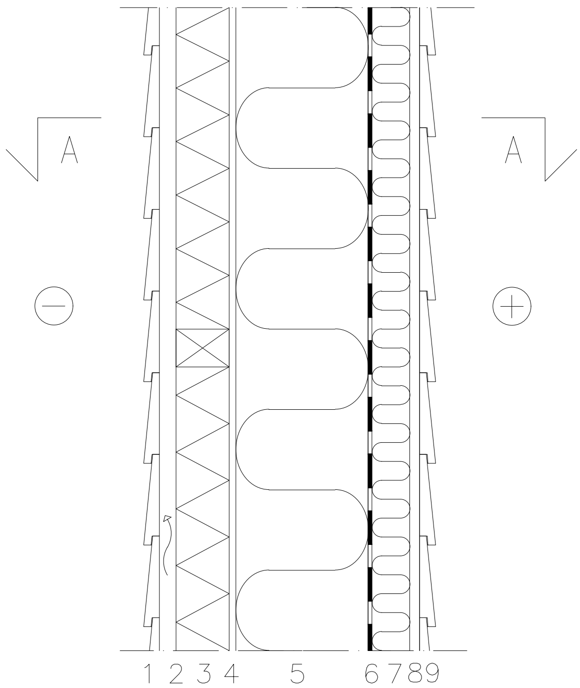
| Type | Required area | Selected area |
|---|---|---|
| | [$mm^2$/m] | [$mm^2$/m] |
| Ø4-250 | 80 | 50 |

## Propping

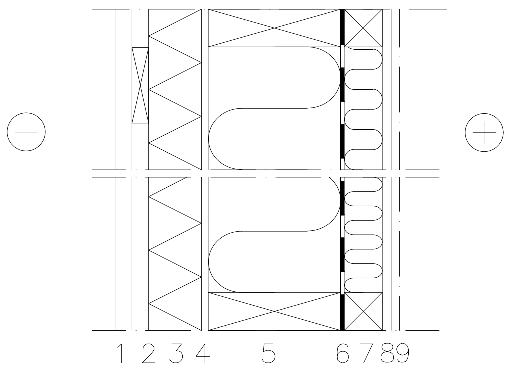

| | Distance from structure end [mm] | Distance from support [mm] |
|---|---|---|
| **CS1** | | |
| 1 | 1185 | S1 + 1185 |
| 2 | 2315 | S1 + 2315 |

**Appendix 11: Structural types**

| | | |
|---|---|---|
| Two apartment house | Exterior wall of the warm room, wooden frame, | |
| Igert Loka | 1<br>13.05.2022 | US 1 |




Section A–A




| | |
|---|---|
| | 1 Timber cladding 28x220 |
| 22 mm | 2 VENTILATION SPACE + board 22x100mm, k600 |
| 70 mm | 3 WINDSCREEN INSULATION, stone wool PAROC Cortex pro (1200x1800) and collar 68mm, collar spacing 1200m |
| 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 173x48mm, k600 |
| | 6 STEAM BARRIER between body and collar |
| 50 mm | 7 THERMAL INSULATION, stone wool PAROC eXtra and collar 50x50mm, k600 |
| 13 mm | 8 BUILDING BOARD, gypsum board |
| | 9 SURFACE MATERIAL OR TREATMENT |

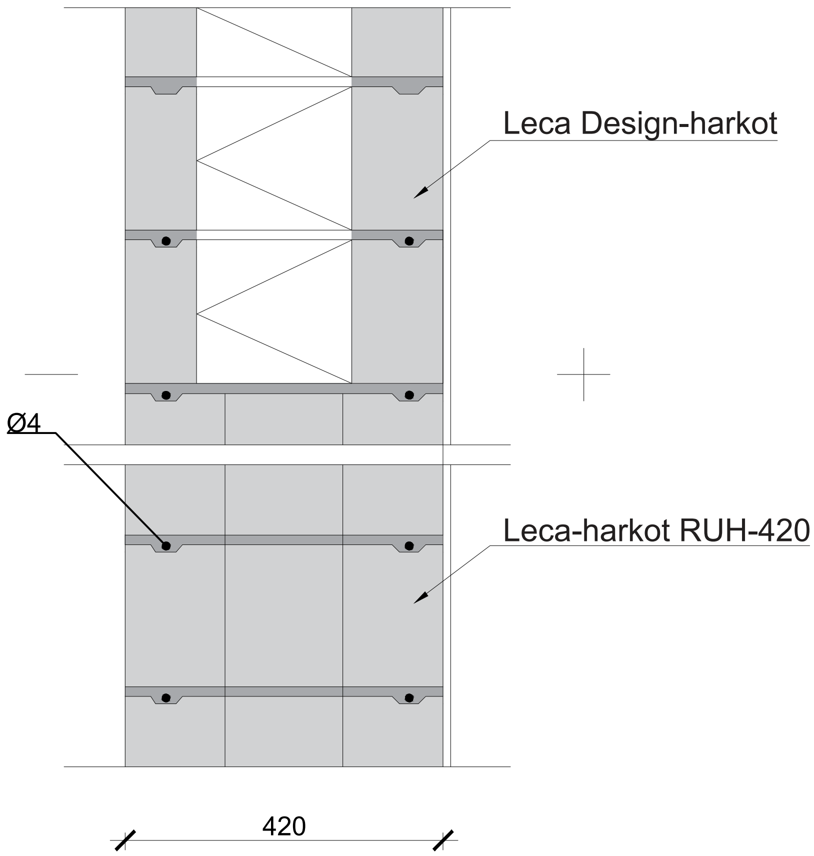
| | |
|---|---|
| U–value | 0.13 W / $m^2K$ |
| Fire class | REI 60 |
| Airborne sound insulation | Rw 57 dB, Rw + C 50 dB, Rw + Ctr 42 dB |

HUMIDITY ANALYSIS OF THE STRUCTURE

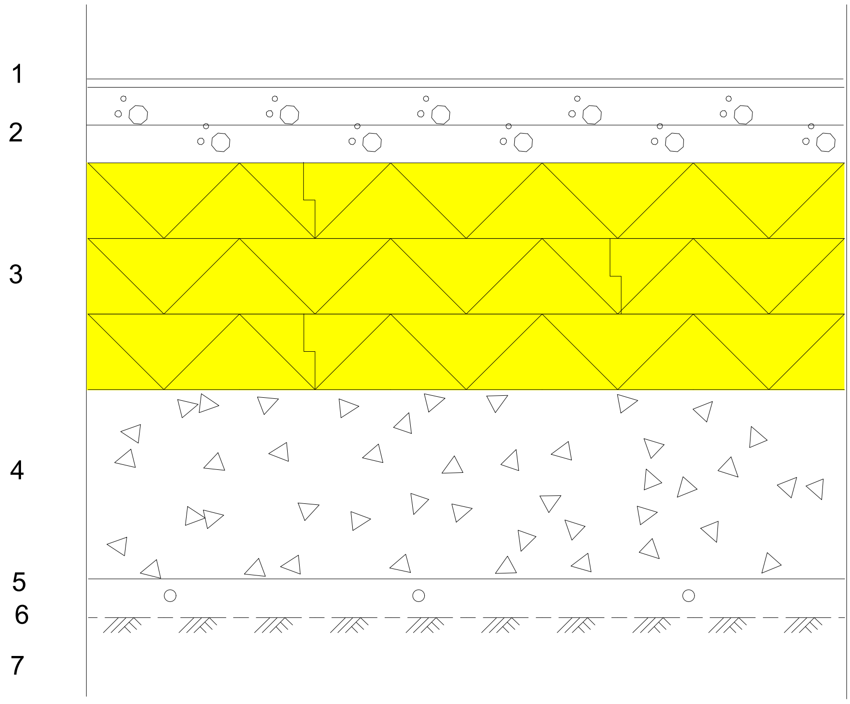
VF–10.11 works well in building physics. A more detailed examination of the humidity can be found in connection with the external wall structure VF–10.9.

FIRE LOAD <20 MJ/$m^2$ INSULATED INSULATION IN THE STRUCTURE

| | | |
|---|---|---|
| Two apartment house | Basement wall | |
| Igert Loka | 2<br>13.05.2022 | US 2 |



| | |
|---|---|
| Two apartment house | Concrete base floor |
| Igert Loka | 3 / 13.05.2022 / VP 1 |




1 Floor covering according to room description

2 Reinforced concrete slab 80 ... 100 mm, =1,7 W/mK, calk $\lambda_U$
according to the structural plan

3 Finnfoam FL-300 insulation 300 mm

- Thermal conductivitys = 0,037 W/mK $\lambda_U$ = 0,038 W/mK $\lambda_D$
- Short-term compressive strength CS (10) 250 kPa
- Water absorption by immersion WL (T) 0.7
- Water vapor permeability = 150 $\mu$
- Dimensional stability DS (70,90)
- Load flow CC (2 / 1.5 / 50) 130 kPa (used as design basis)

4 Crushed stone 8 ... 16 mm ≥200 mm

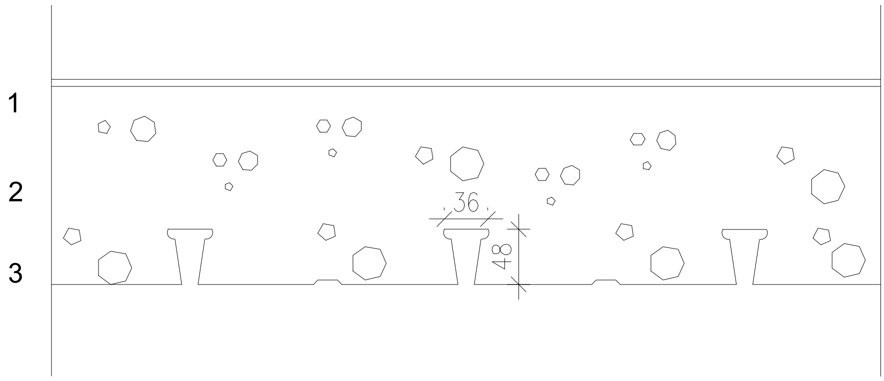
5 Sand layer with passive heat / cold pipes

6 Filter cloth

7 Base, tilt to the secret 1:50

| | |
|---|---|
| U-VALUE | 0,10 W/m K |

| | | |
|---|---|---|
| Two apartment house | Composite slab | |
| Igert Loka | 4<br>13.05.2022 | VP 2 |

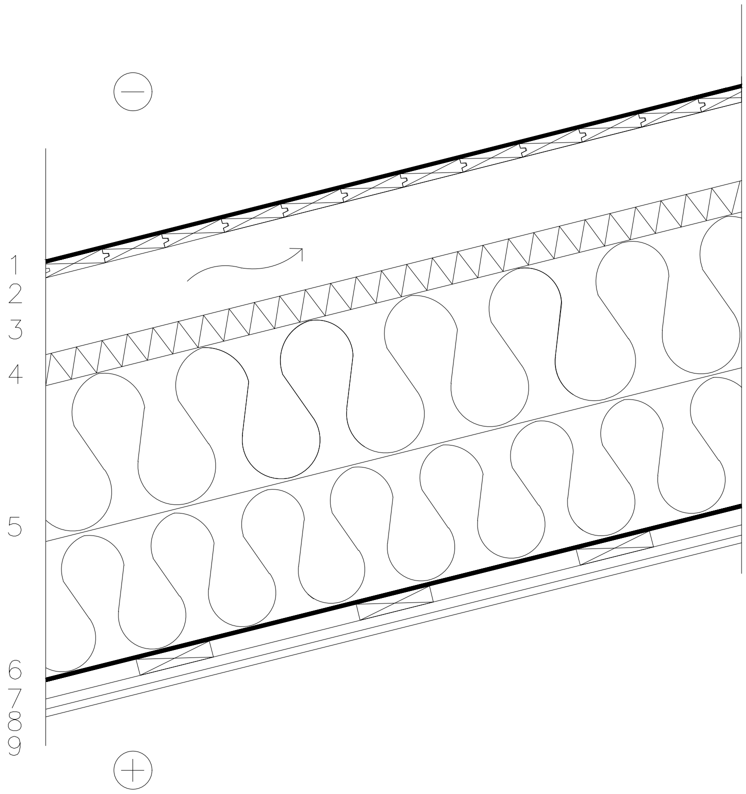



1 SURFACE MATERIAL ACCORDING TO DESIGNER SPECIFICATION

2 REINFORCED CONCRETE SLAB ACCORDING TO CONSTRUCTION DRAWING

3 LOAD-BEARING CS48-36 PROFILED SHEET MOULD

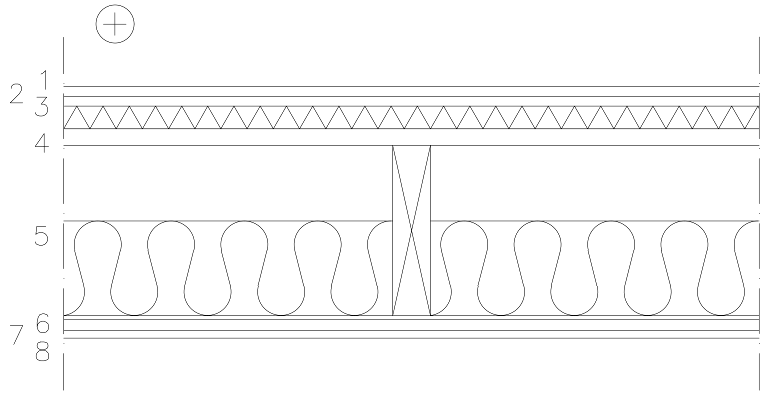
| | | |
|---|---|---|
| Two apartment house | Wooden roof structure | |
| Igert Loka | 5<br>13.05.2022 | YP 1 |




1 BITUMEN SHEET 3mm
2 TIMBER 23X95
3 VENTILATED AIR GAP 100 mm
4 WINDSCREEN INSULATION, PAROC Cortex pro 40 mm
5 THERMAL INSULATION, stone wool PAROC eXtra (175 + 200mm)
6 AIR OR STEAM SEAL 22 mm
7 RARE BOARDING 22x100mm, k300
8 BUILDING BOARD, gypsum board 13 mm
9 SURFACE MATERIAL OR TREATMENT, depending on room description

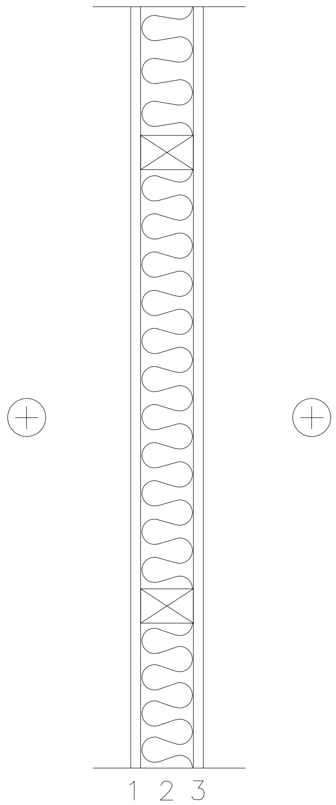
U–value 0.09 W / $m^2K$
Research technology Rw 56 dB, Rw + C 54 dB, Rw + Ctr 52 dB (incl.)
Rw 55 dB, Rw + C 54 dB, Rw + Ctr 49 dB (sheet metal coating)

| | | |
|---|---|---|
| Two apartment house | Intermediate floor | |
| Igert Loka | 6<br>13.05.2022 | VP3 |




1 FL□□R C□VERING (parquet, etc.)
2 BUILDING PANEL LAYERS (floor plasterboard)
30 mm 3 STEP INSULATI□N, stone wool PAR□C SSB 2t
22 mm 4 FL□□R CHIPB□ARD or similar building board
5 SUPP□RTING STRUCTURE + fire / sound insulation according to a separate construction plan
PAR□C eXtra / PAR□C eXtra F / PAR□C Natura Lana
6 C□NNECTI□N / SPRING FRAME as required
7 BUILDING PLATE
8 SURFACE MATERIAL □R TREATMENT

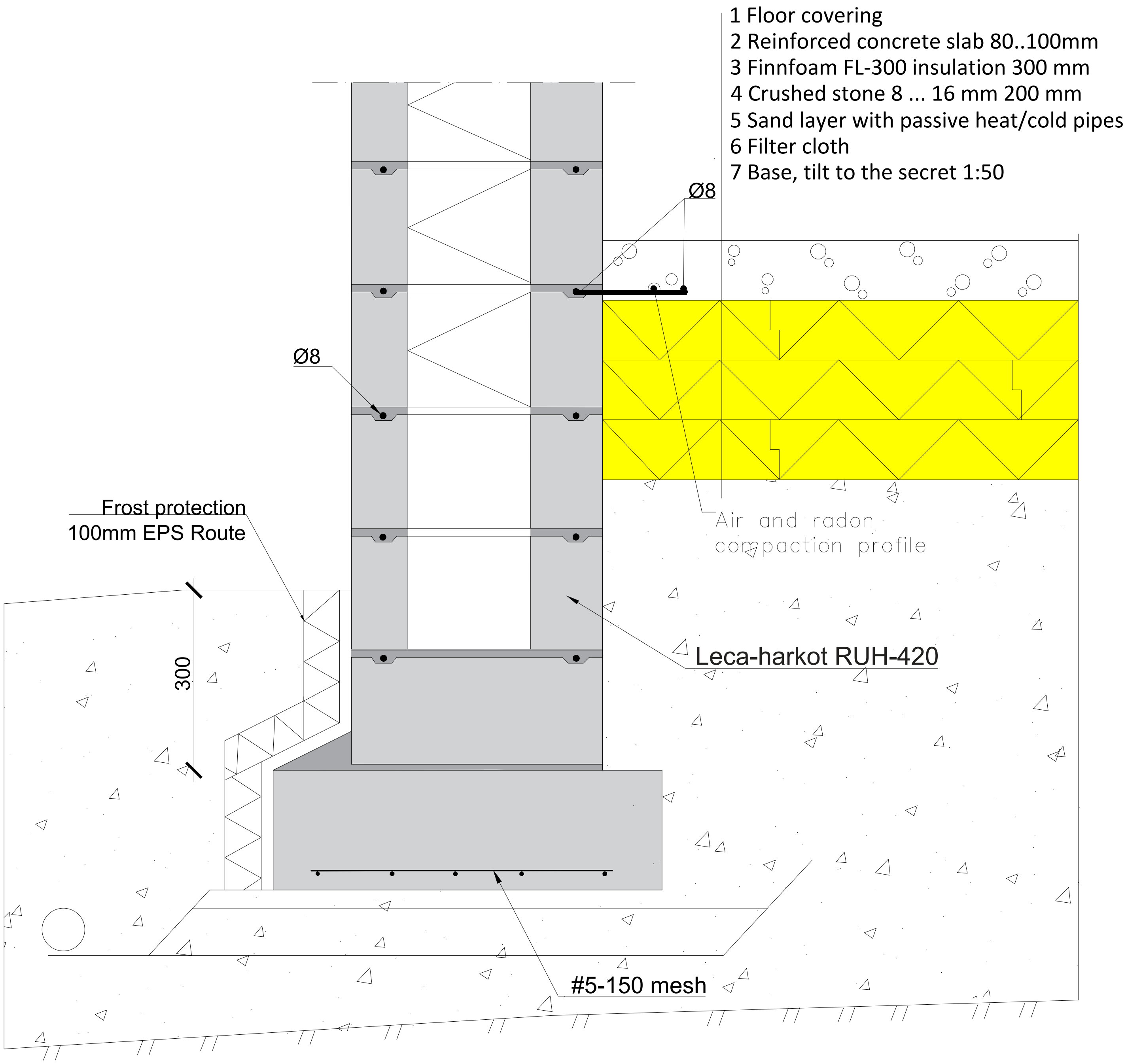
| | | |
|---|---|---|
| Two apartment house | Partition wall | |
| Igert Loka | 7<br>13.05.2022 | VS 1 |



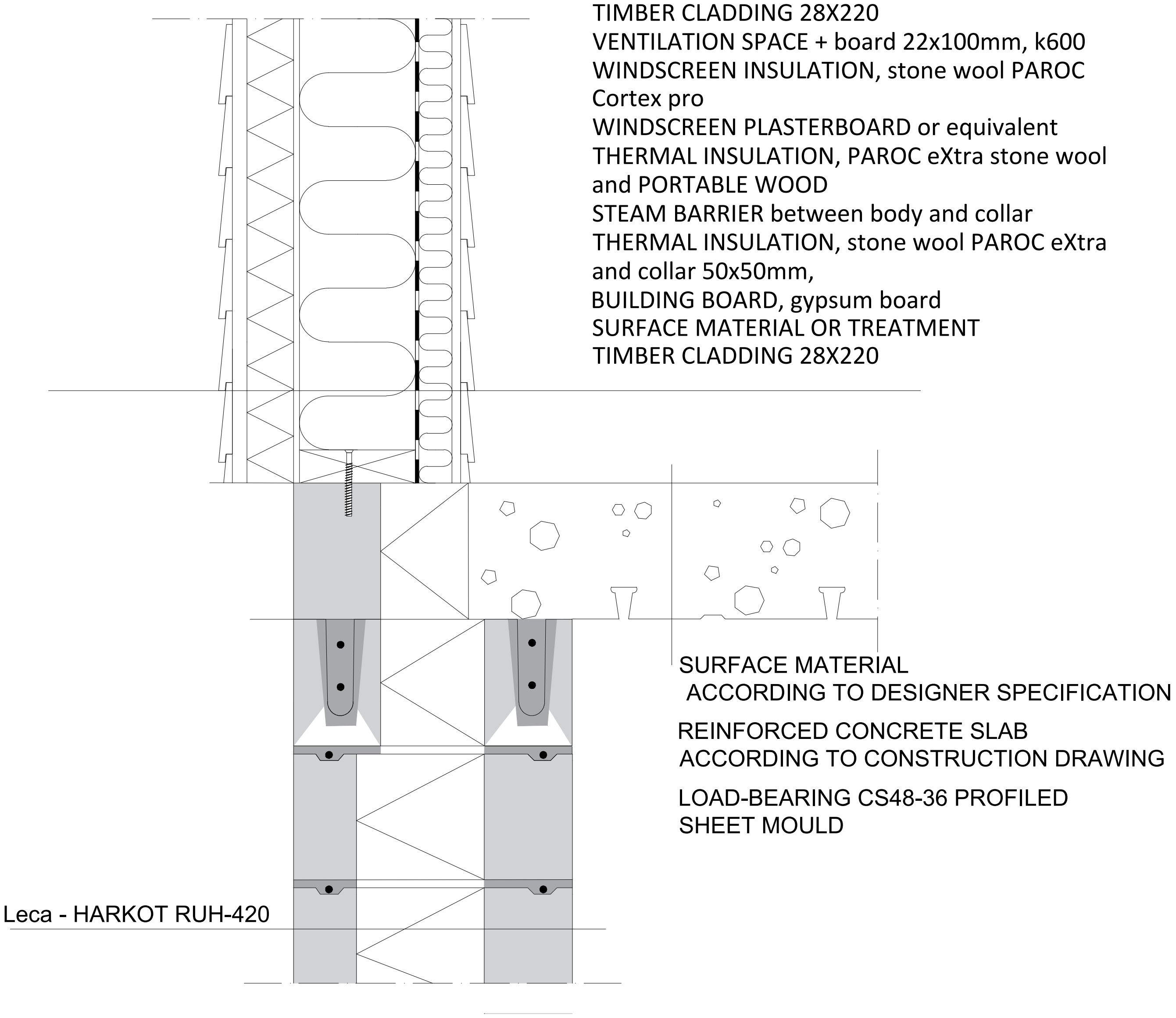

1 TIMBER PANEL 28X220
2 TIMBER STUDS 70mm cc600/ PAROC extra 70mm
3 TIMBER PANEL 28X220

FIRE CLASS EI45

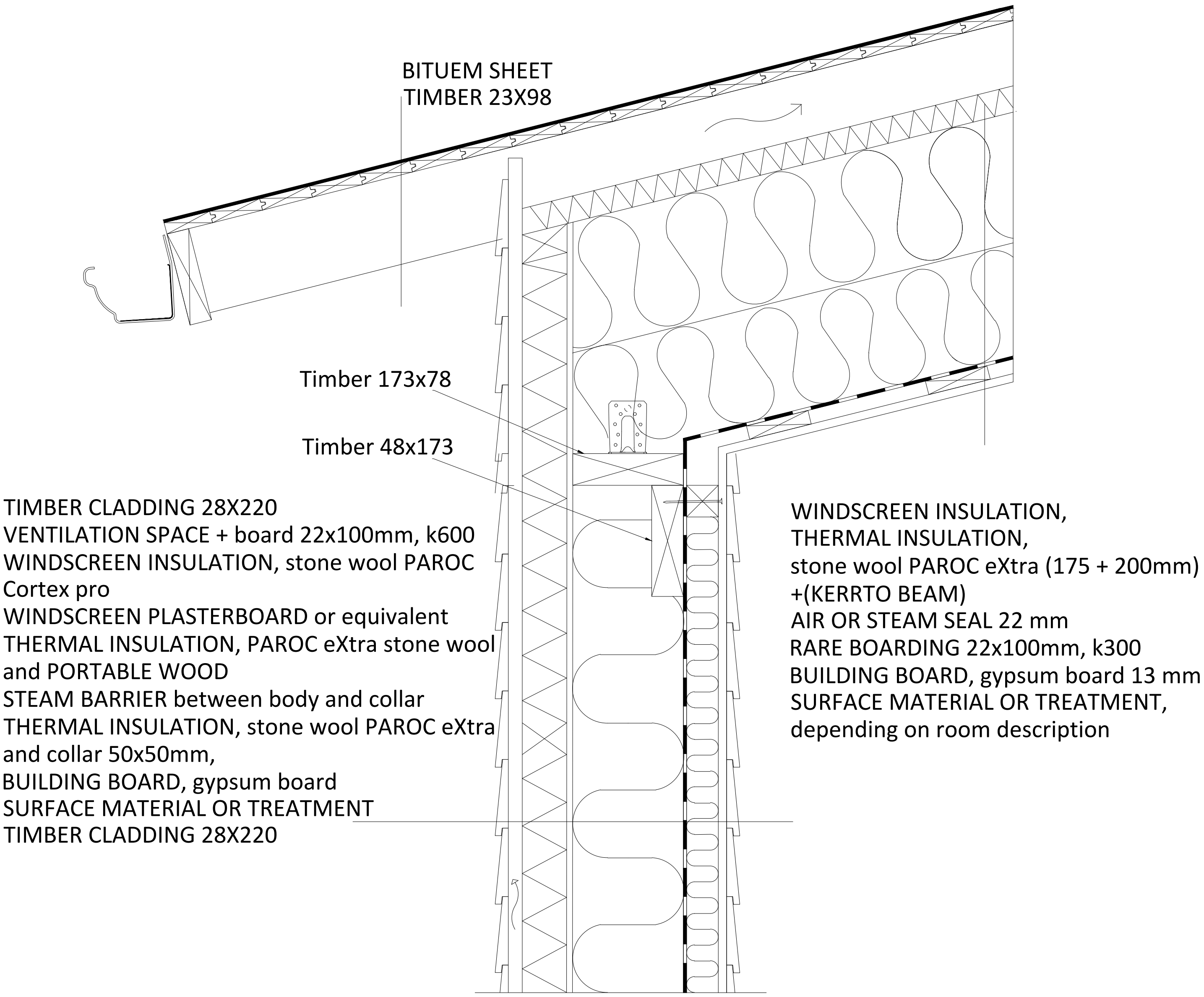
| | | |
|---|---|---|
| Two apartment house | Basement wall and floor connection | |
| Igert Loka | 8<br>13.05.2022 | VP1–US2 |



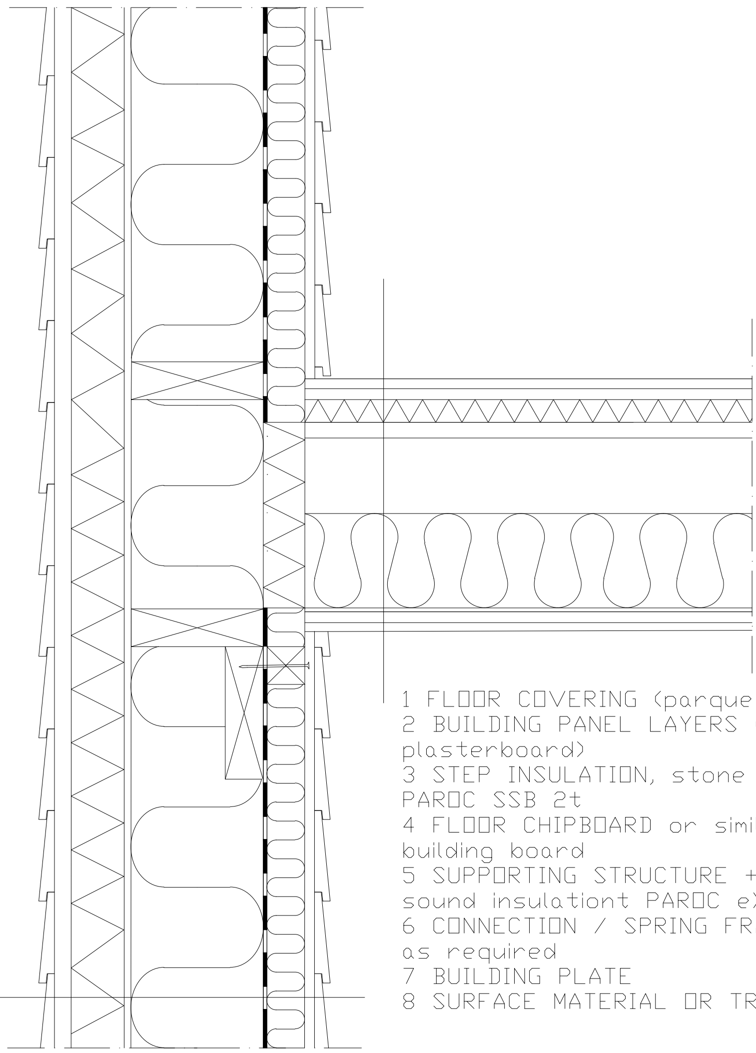
| | | |
|---|---|---|
| Two apartment house | Composite slab connection to outter walls | |
| Igert Loka | 9<br>13.05.2022 | VP1–US2–VP2 |



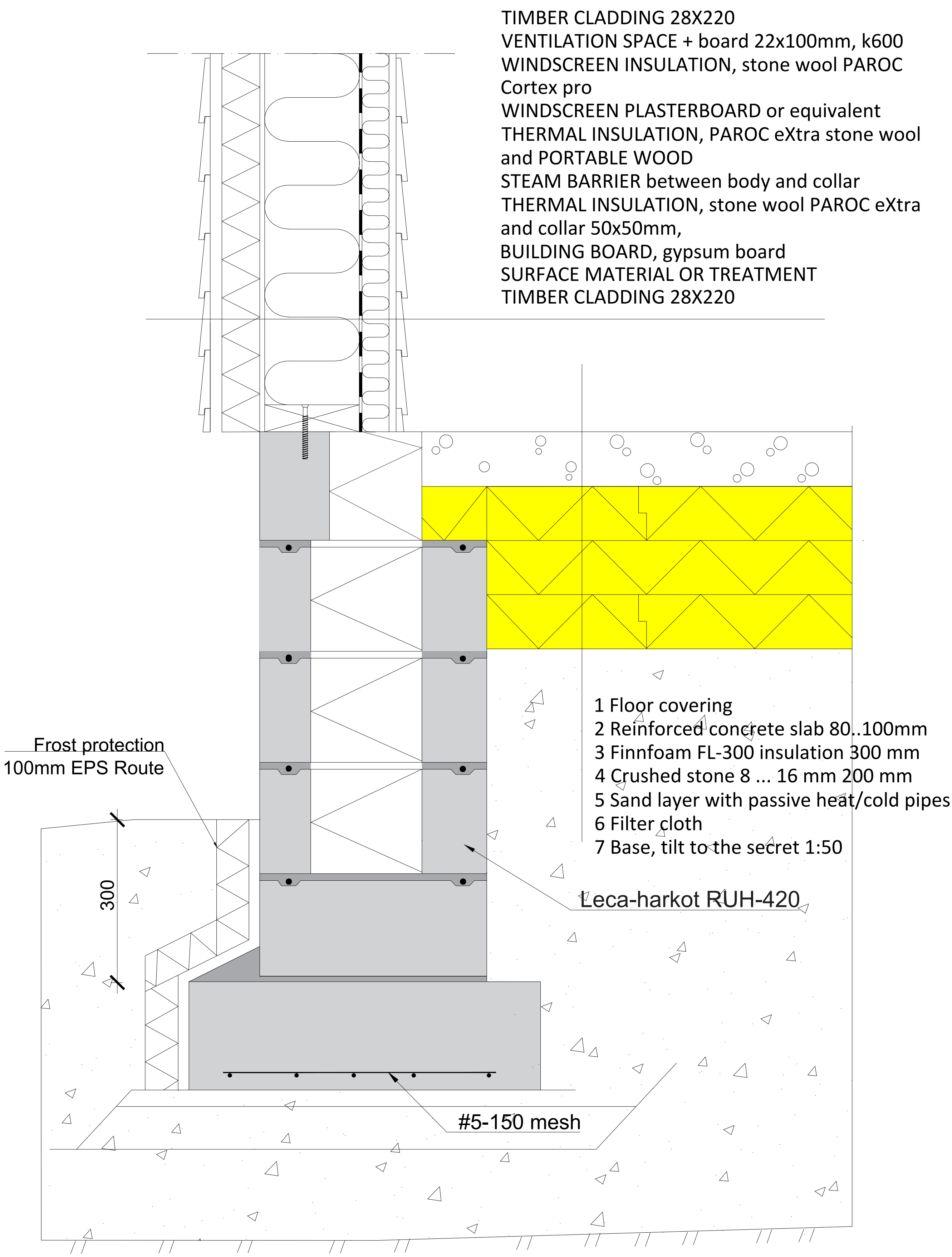
| | | |
|---|---|---|
| Two apartment house | Roof connection to outter timber wall | |
| Igert Loka | 10<br>13.05.2022 | US1–YP1 |



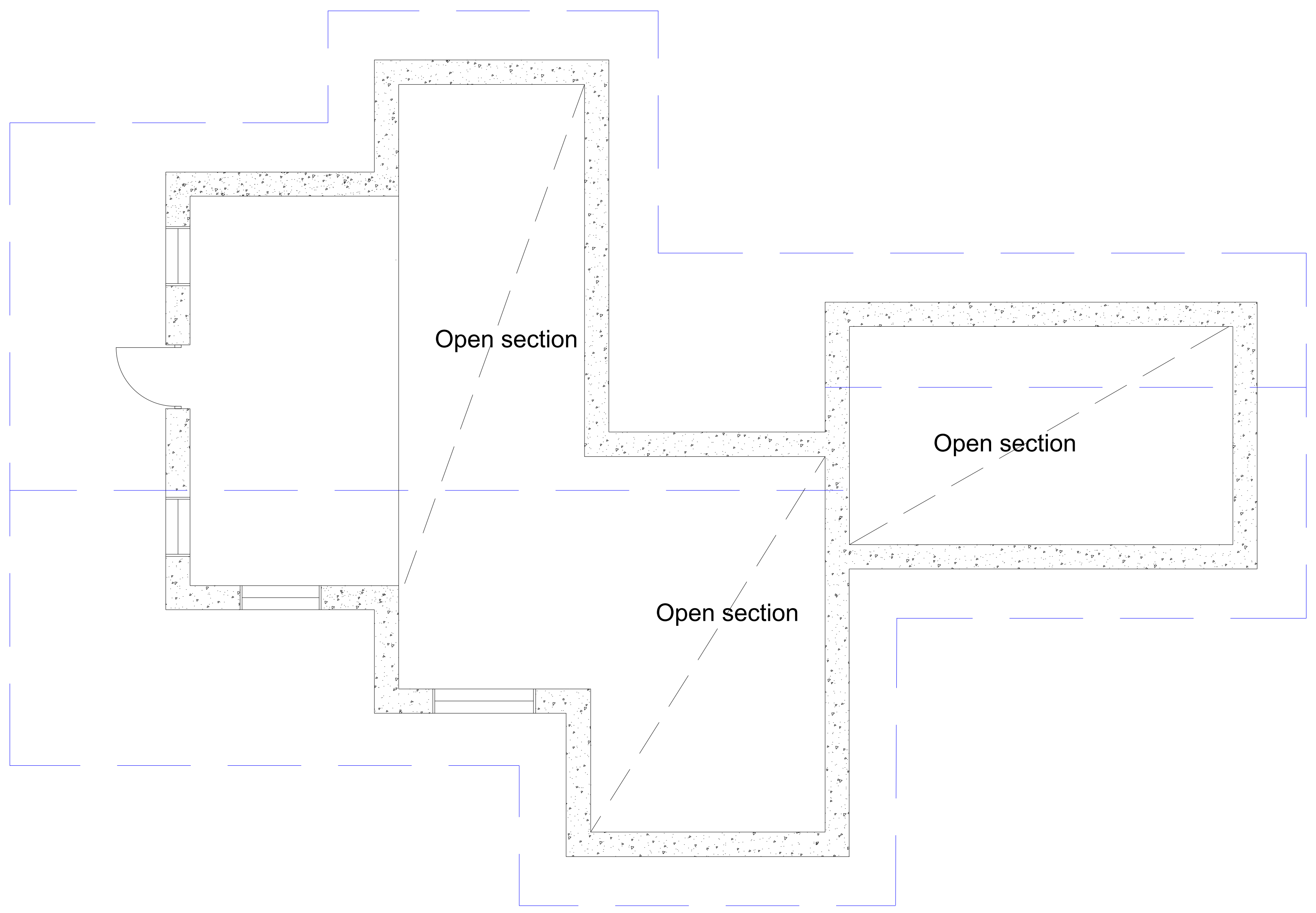
| | | |
|---|---|---|
| Two apartment house | Exterior wooden wall and intermediate floor connection | |
| Igert Loka | 3<br>13.05.2022 | US1–VP3 |



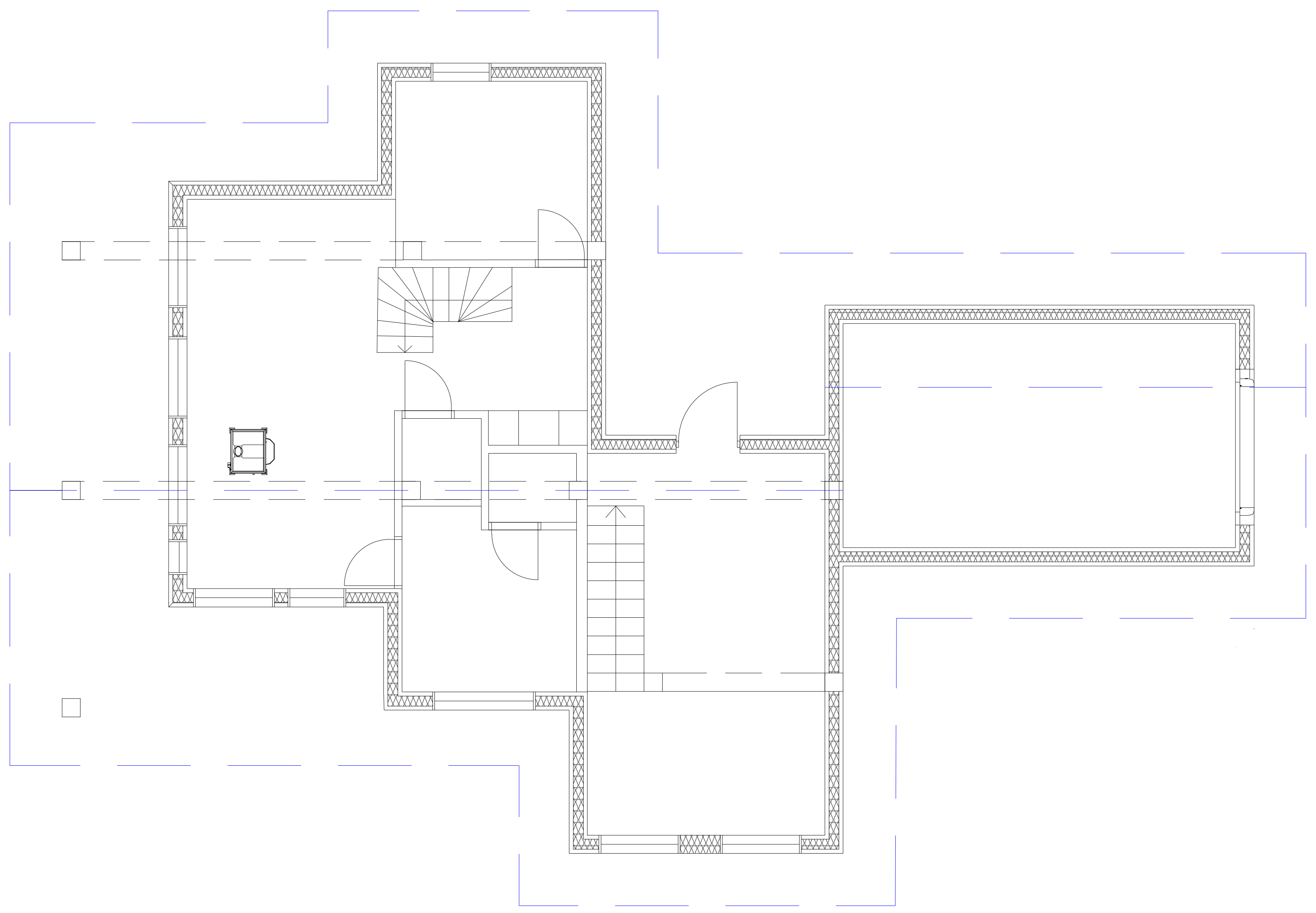
| Two apartment house | First floor and exterior walls connection | |
|---|---|---|
| Igert Loka | 3<br>13.05.2022 | VP1–US2–VP1 |



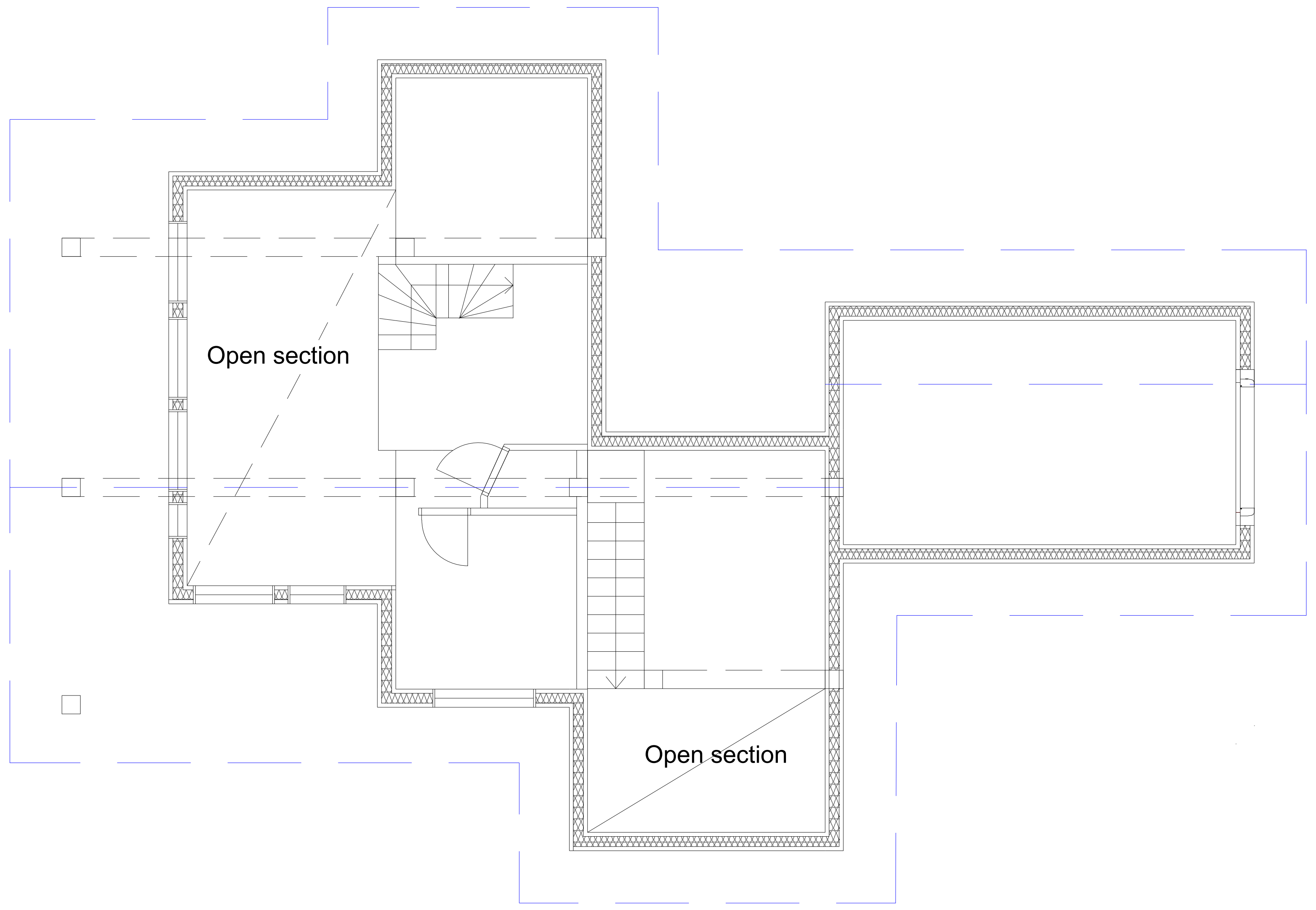
**Appendix 12: Floor layout plan**




BASE FLOOR



FIRST FLOOR




SECOND FLOOR

| | | | |
|---|---|---|---|
| K.OSA 40 | KORTTELI/TILA 116 | TONTTI/RNo 4 | RAKENNUSLUVAN TUNNUS |
| RAKENNUSTOIMENPIDE NEW BUILDING | | | PIIRUSTUSLAJI MASTER DRAWING — JUOKS.No 2 |
| RAKENNUSKOHTEEN NIMI JA OSOITE TWO APARTMENT HOUSE Kolari | | | PIIRUSTUKSEN SISÄLTÖ FLOOR LAYOUT 1:100, CHIMNEY 1:20 — MITTAKAAVA |
| IGERT LOKA | | | SUUNN.ALA AR — TYÖ No — PIIR.No — MUUTOS |
| | | | PÄIVÄYS 13.05.2022 — YHT.HENK. — TIEDOSTO — PIIRTÄJÄ |