Engineering Principles and Design Procedures of a Detached House in Finland

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Bachelor's thesis

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ABSTRACT

Degree Programme in Construction Engineering
Visamäki

Author Md. Zahid Hasan Rubel **Year** 2018

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Supervisor(s) Tomi Karppinen

TIIVISTELMÄ

Tämän opinnäytetyön tavoitteena oli suunnitella omakotitalo Suomessa. Yleisesti rakennesuunnittelu tulee tutuksi ja helpommaksi suunnittelukokemuksen myötä, mutta aloitteleville suunnittelijoille työ on vaikeampaa ja hitaampaa. Tämä työ toimii suunnitteluoppaana paikallisille ja kansainvälisille toimijoille.

Työn alkuvaiheeseen on koottu tietoa omakotitalon eri suunnitteluvaiheista ja niihin liittyviä laskelmia, kuten kattoristikon suunnittelu, kantava seinä, perustukset, lämmönläpäisevyys, energiatehokkuus, sekä lopuksi rakennuksen toiminta kosteutta vastaan. Työn keskivaihe käsittelee suunnitteluprosessiin liittyviä standardeja ja määräyksiä.

Eri rajatiloja on käytetty suunnittelutyön perustana. Niiden avulla rakennus suunnitellaan turvalliseksi romahdusta vastaan ja kestäväksi rakennuksen käyttötarkoitusta varten. Minkä tahansa rakenteen tärkeimpiä ominaisuuksia on sen todellinen lujuus. Rakenteellisen lujuuden tulee olla riittävän suuri kestämään joitakin hypoteettisia ylikuormitustiloja. Työssä käsitellään myös muita rajoittavia tekijöitä, kuten kustannusarvio, paloturvallisuus ja LVI-suunnittelu.

Lopuksi on saatu aikaan joukko rakennesuunnitelmia rakennuslupahakemusta varten. Tämä insinöörityö mahdollistaa aiheeseen liittyvän lisätutkimuksen tulevaisuudessa.

Avainsanat murtorajatila, käyttörajatila, kuormitusydistelmä, arkkitehtisuunnittelu, rakennesuunnittelu, energiatehokkuus

Sivut 124 sivua, joista liitteitä 8 sivua

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ABSTRACT

The purpose of this Bachelor's thesis was to design a detached house in Finland. In general, construction designing work becomes quite familiar and easier for the structural engineers with their experiences over time. In contrast, for the inexperienced, this is not so. Therefore, the work carries a significant importance to be done and plays a key role to produce a design manual for both local and international stakeholders.

At first, the thesis states the conceptual understanding based on various sources and regarding the design work. After that, different calculations for designing a roof truss, load bearing wall, foundation, thermal transmittance, energy efficiency and building behaviour against moisture are included. In addition to that, prerequisites or the standards from different rules, laws and decrees are shown.

The limit state concept used in conjunction with a partial factor method was considered as the basis for the design work in this thesis. It denotes strength design in brief, i.e. a structure must be safe against collapse and serviceable in use. The most important characteristics of any structural member are its actual strength. Therefore, the design strength for the members should be adequate to withstand certain hypothetical overload stages. However, some limitations for instance, cost estimation, fire safety, HVAC, and plumbing design were also discussed in the thesis.

As a result of the thesis, a complete set of blueprints ready to be attached to the application for a building permit were produced. This thesis provides opportunities to conduct some further research.

Keywords ultimate and serviceability limit, load combination, architectural and structural design, energy efficiency

Pages 124 pages including appendices 8 pages

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Md. Zahid Hasan Rubel
13.06.2018

CONTENTS

LIST OF ACRONYMS

AMSL	Above Mean Sea Level
BNBC	The Bangladesh National Building Code
HVAC	Heating Ventilation and Air Condition
MoE	Ministry of the Environment
NAS	National Annex to Standard
NBCF	National Building Code of Finland
SFS	Finnish Standards Association

1 INTRODUCTION

Today, the world is running by the technical advancement. The objective of the new technological introduction is to achieve work efficiency and accuracy with mass production by saving time. However, in Europe, and especially, in Finland the construction sector is playing a major role regarding its national economic growth at the moment. The regulations for construction are continuously updated over the period by adapting new technology, for example, nearly zero energy building is a new concept nowadays. Consequently, the building permission process and structural design are also being moved from the old fashion to new systems in Finland. On studying current legislation concerning the construction of any structures the whole process to design a detached house requiring architectural drawings, structural design, and energy efficiency of a building is shown in this thesis.

A structure must be safe against collapse and serviceable in use. Serviceability requires that deflections be adequately small, if any, and they should be kept to tolerable limits. Safety requires that the strength of the structure is adequate for all loads that may foreseeably act on it. Safety could be ensured by providing a carrying capacity just barely in access of the known loads, if the loads and their internal effects were known accurately and if the strength of a structure, built as designed, could be predicted accurately.

2 GOALS OF THE STUDY

The goal of this thesis is to examine and show an overall planning and design process of a detached house practiced in Finland. The aim is to make architectural drawings using Revit 2017 Software and to investigate and design the overall structural stability and building behaviour. Other objectives are to calculate the energy efficiency and to produce structural drawings using Autocad 2017 Software. The target of the study is to make a design manual and drawing portfolio in order to assist the fellow students and novice engineers for their design work.

Since, a complete design work comprises a group of effort which is not possible to complete within this short period the following aspects are not included in the study

- Electric design
- Detail fire safety design
- HVAC design
- Plumbing design
- Structural joints analysis
- Finite element analysis
- Stability of the whole building frame
- Cost estimation and project planning

The first chapter discusses the background and necessity of the study including the aim, scope and limitation of the study. The second chapter shows the location of the project and gives an overall geographical and demographic idea of the place where the project is going to be held. Different terminologies, issues and common aspects regarding this design work are shown in the third chapter depicting the detailed conceptual phenomena. The fourth chapter is composed of in detailed theories, laws and regulations incorporated to this project. An engineer should consider all the criteria mentioned here in order to complete a design work. The basic requirements and the theme work for this design are described in in the fifth chapter. This plays a very important role whether it contains architectural design portfolio which are the main blueprints to place for a building permit application to the city authority. The detailed work including calculation, structural design and drawings for each structural elements of the building are presented in the sixth chapter. All the assumptions are verified here with the theory in order to get the structural stability and right behaviour. The final chapter contains a conclusion including the overall findings and directs the possible further research work based on this study.

3 PROFILE OF THE STUDY AREA

The building site is located at Rapamäki, especially, in Hongistonkuja 4, 13500 Hämeenlinna which is the heart of a historical province of Häme in the south of Finland. The altitude of the site area is about 109 m from the sea level including 60.980 latitude and 24.410 longitude (Elevation map n.d.). Distances from the other major cities are 100, 80, 80 and 144 km from Helsinki, Tampere, Lahti and Turku respectively. In addition, the area of Hämeenlinna city is 1785.83 km2 including 245.77 km2 lakes with the population of 67845. However, the area is not near by the sea. (Hämeenlinna n.d.). Figures 1 and 2 below show the map and location of Hämeenlinna city.

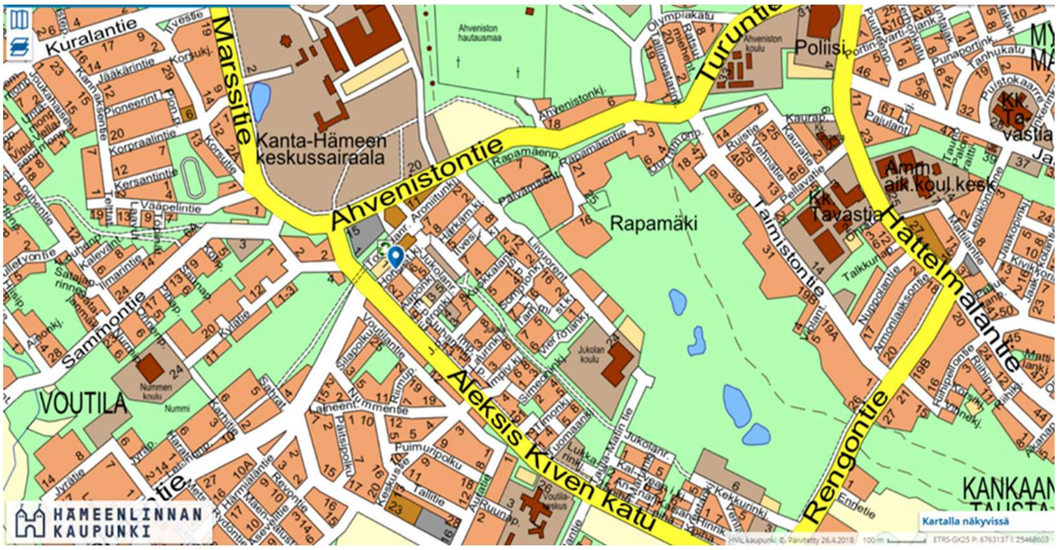


Figure 1. City map of Hämeenlinna (Hämeenlinna n.d.).

Figure 2.   Location map of Hämeenlinna (Liiteri n.d.).

4 CONCEPT AND THEORY

4.1 Building materials

Plenty of materials around the world have been used for building constructions. But among them, the following two i.e. concrete and timber are used for the project. They will be discussed below.

4.1.1 Concrete

Concrete is a mixture of portland cement or any other hydraulic cement, fine aggregate, coarse aggregate, and water, with or without admixtures. Concrete today is a sophisticated material to which exotic (foreign) constituents can be added and, with computer-controlled batching, can produce a product capable of achieving 350 MPa compressive strength. Moreover, in order to gain more tensile strength, reinforcements are added to the concrete structures. On the other hand, because of its high strength, it is used extensively for construction of infrastructures as well as structures. (Civil engineering terms n.d.).

4.1.2 Timber

Timber denotes wood which is suitable for building or carpentry and for various engineering and other purposes. Timber or wood as a building material possesses a number of valuable properties, such as low heat conductivity, amenability to mechanical working, low bulk density and relatively high strength. Timber has been a very important structural member from time immemorial. It has been extensively used as beams, columns and plates in construction in a variety of situations, such as foundation, flooring, stairs and roofing. (Gopi 2009).

4.2 Building elements

The main components of any buildings which help to stabilize and build the structure are normally known as building elements. Some major elements of buildings are defined below.

4.2.1 Roof and truss

A roof is the structure that represents the upper cover of a building. A roof is a part of building envelopes that provides shelter and protection from the weather and animals. Truss is a roof member that usually locates under the roof materials. In engineering, a truss is a structure that consists of two-force members only, where the members are organized so that the assemblage as a whole behaves as a single object. A roof truss is a structural framework designed to bridge the space above a room and to provide support for a roof. Trusses usually occur at regular intervals, linked by longitudinal timbers such as purlins. The space between each truss is known as a bay. (GTZ, UNDP & ISDR 2008). Figure 3 shows a picture of a timber roof truss.

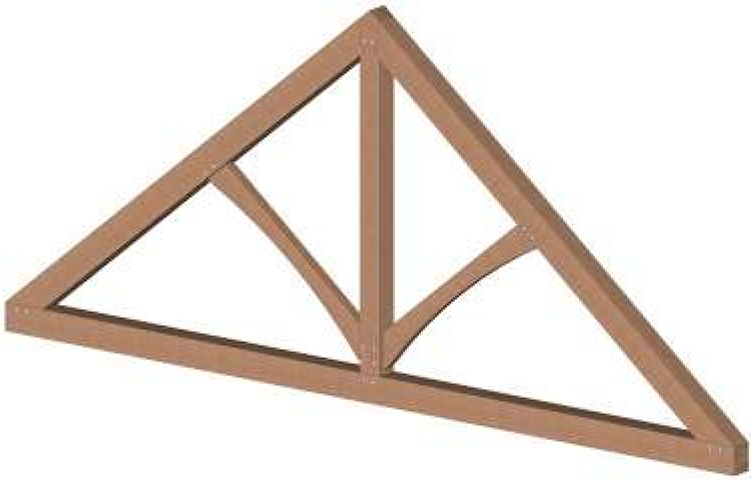


Figure 3. Timber roof truss (Vermont timber works 2011).

4.2.2 Floor and hollow core slab

The lower surface of the room is termed as floor. It could be made of different constructions materials, for example, timber, and concrete and so on. A hollow core slab for floor or roof is a modern technology used in building construction industry now a day. A hollow core slab is a precast pre-stressed concrete member with continuous voids provided to reduce weight and cost. They are primarily used as a floor deck system in residential and commercial buildings as well as in parking structures because they are economical, have good fire resistance and sound insulation properties, and are capable of spanning long distances with relatively small depths. Structurally, a hollow core slab provides the efficiency of a pre-stressed member for load capacity, span range, and deflection control. Hollow core slabs can make use of pre-stressing strands, which allow slabs with depths between 150 and 260 mm to span over 9 meters. When used in buildings, several hollow core slabs are placed next to each other to form a continuous floor system. The small gap that is left between each slab is usually filled with a non-shrink grout. To give the floor a smooth finished surface, a topping slab overlay, typically 5cm deep is poured on the top surface of the hollow core slabs. (The concrete centre n.d.). This is shown in Figure 4 below.

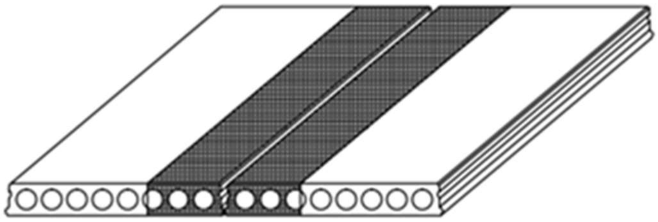


Figure 4. Hollow core slab (Parman ontelolaatastot 2013).

4.2.3 **Beam**

A beam is a structural member used for bearing loads. It is typically used for resisting vertical loads, shear forces and bending moments. It is a structural element that primarily resists loads applied laterally to the axis of a beam. Its mode of deflection is primarily by bending. The loads applied to the beam result in reaction forces at the beam's support points. (Darwin, Dolan & Nilson 2016). It could be in various shapes (e.g. Rectangular, circular etc.), designs (e.g. I-beam, H-Beam, T-beam etc) or support systems (e.g. Pin, rigid, cantilever etc.). An example of a timber beam is shown in Figure 5.

Figure 5. Timber Beam (Home improvement n.d.).

4.2.4 **Column**

A column can be defined as a vertical structural member designed to transmit a compressive load. A column transmits the load from the ceiling/roof slab and beam, including its own weight to the foundation. Hence, it should be realized that the failure of a column results in the collapse of the entire structure. The design of a column should, therefore, receive importance. Columns are mostly constructed from concrete; apart from that materials such as Wood, Steel, Fibre-reinforced polymer, Cellular PVC, and Aluminum, too, are used. The type of material is decided on the scale, cost and application of the construction. A picture of a column is shown below in Figure 6.

Figure 6. Concrete column (Houzz n.d.).

4.2.5 **Building wall**

A building wall serves a number of functions, ranging from separation of space in a building to restraining earth adjacent to a building or building site. Walls are of several types: Panel walls, curtain walls, partition walls, shear walls and so on. Generally, almost every wall could be designed either as load-bearing or non-load bearing. (Darwin, Dolan & Nilson 2016). Figure 7 shows a picture of building wall.

Figure 7. Building wall (Double fresh 2010).

4.2.6 **Foundation**

The foundation is the part of the structure that is usually placed below the surface of the ground and that transmits the load to the underlying soil or rock. Two essential requirements in the design of foundation are that the total settlement of the structure be limited to a tolerably small amount and that differential settlement of the various parts of the structure be eliminated as nearly as possible. Foundations could be individual or spread. Besides, a number of sub-division belongs to the foundation category. (Darwin, Dolan & Nilson 2016). Figure 8 shows a building foundation.

Figure 8. Building Foundation (Homes 4 India n.d.).

4.3 **Structural Mechanics and engineering principles**

Engineering principles refers to the rules, ideas and concepts regarding construction of the structures and the structural mechanics compute building stability regarding building force, stressed, formation, deflection and service of uses. Major terms of these issues are discussed below

4.3.1 **Stress**

Stress is force per unit area and is usually expressed in pounds per square inch. Stress is a measure of:

$$\frac{Applied\ force\ on\ a\ material}{Area\ over\ which\ that\ force\ is\ applied}$$

Normal stress is defined as: $\sigma = \frac{F}{A}$

In the SI, the unit of stress is the Pascal (Pa), the newton per meter squared (N/m^2). Stresses are of three types: Tensile, Compressive and

Shearing stress. If the stress tends to stretch or lengthen the material, it is called tensile stress. Tensile stresses always act at right-angles to (normal to) the area being considered. If the stress tends to compress or shorten the material, it is called compressive stress. Compressive stresses always act at right-angles to (normal to) the area being considered. If the stress tends to shear the material then it is Shearing stress. Shearing stresses are always in the plane of the area (at right-angles to compressive or tensile stresses). (University of Victoria n.d.).

4.3.2 **Strain**

Strain is the amount by which a dimension of a body changes when the body is subjected to a load, divided by the original value of the dimension. Strain is a measure of:

$$\frac{Elongation\ of\ a\ material\ due\ to\ an\ applied\ force}{The\ original\ length\ of\ the\ material}$$

Normal strain is defined as: $\varepsilon = \frac{\Delta L}{L_0}$

The relationship between stress and strain is defined by Hooke's Law as shown in Figure 9 below

$$\sigma = \varepsilon E$$

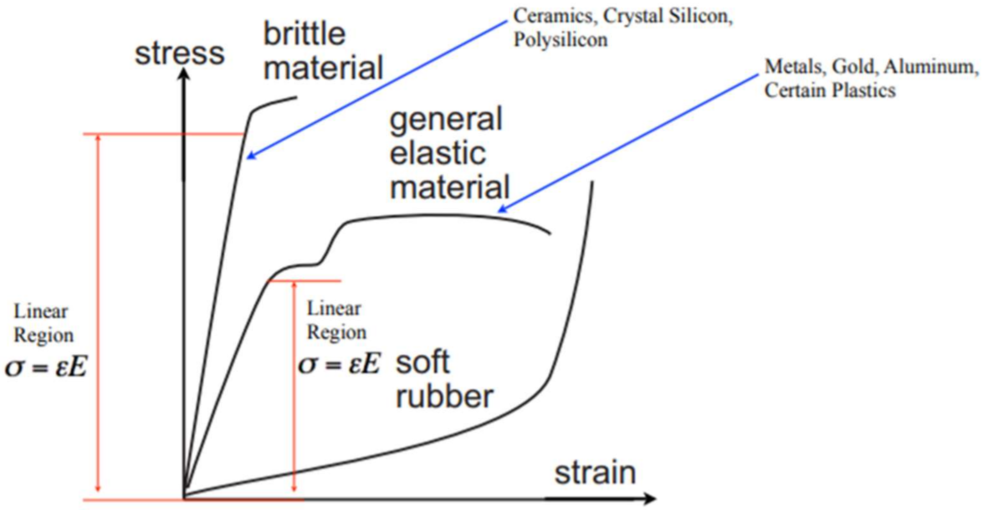


Figure 9. Stress-strain relationship (Nong Lam University n.d.)

4.3.3 **Elasticity**

Elasticity is an ability of material to get back after removing the courses of changes (for example load) into the former (original) state. If the external forces producing deformation do not exceed a certain limit, the deformation disappears with the removal of the forces. Thus, the elastic behavior implies the absence of any permanent deformation. (Technical University of Ostrava n.d.).

4.3.4 **Plasticity**

Plasticity is an ability of material to deform without any rupture in a non-returnable way. If the external forces producing deformation do not exceed a certain limit, the deformation disappears with the removal of the forces. Thus, the elastic behavior implies the absence of any permanent deformation. Elasticity has been developed following the great achievement of Newton in stating the laws of motion, ''After removing the load permanent deformations stay''. It is used in composite steel-concrete structures. (Technical University of Ostrava n.d.).

4.3.5 **Tensile strength**

Tensile strength has been measured in terms of the modulus of rupture, the computed flexible tensile stress at which a test beams of plain concrete fractures for many years. The nominal stress is computed on the assumption that concrete is an elastic material, and bending stress is localized at the outermost surface. It is apt to be larger than the strength of concrete in uniform axial tension. Tensile strength is the maximum amount of tensile stress that a material can take before failure. There are three definitions of tensile strength. Yield strength that refers to the stress at which material strain changes from elastic deformation to plastic deformation, causing it to deform permanently. Secondly, ultimate strength shows the maximum stress a material can withstand. Finally, breaking strength is the stress coordinates on the stress-strain curve at the point of rupture. (Civil engineering n.d.).

4.3.6 **Compressive strength**

Compressive strength or compression strength is the capacity of a material or structure to withstand loads tending to reduce size, as opposed to tensile strength, which withstands loads tending to elongate. In other words, compressive strength resists compression (being pushed together), whereas tensile strength resists tension (being pulled apart). In the study of strength of materials, tensile strength, compressive strength, and shear strength can be analyzed independently. (Wikipedia n.d.).

4.3.7 **Axial force**

Axial force is the compression or tension force acting in a member. If the axial force acts through the centroid of the member it is called concentric loading. When the loading on such a member is on a plane the same as the member itself, it represents a two-dimensional (planar) case. In such cases, the internal forces also lie on the same plane. The internal forces on any cross-section can be expressed with two orthogonal force components and one moment in the plane of loading. We can align x - axis along the centroidal axis of the member and we can also align one of the forces, along this centroidal axis (along the primary dimension). Then this internal force will be known as the axial force. (Wikiengineer n.d.).

4.3.8 Shear force

Shear force is the force that tends to separate the member. Shear Forces occur when two parallel forces act out of alignment with each other. The tangential act of a force with the axial force is known as shear force. Then the centroidal axis of a member works along x-axis, then F_x is the axial force and F_y and F_z are the two shear forces of that member. (NPTEL n.d.). Figure 10 shows the relationship between different figures and moments.

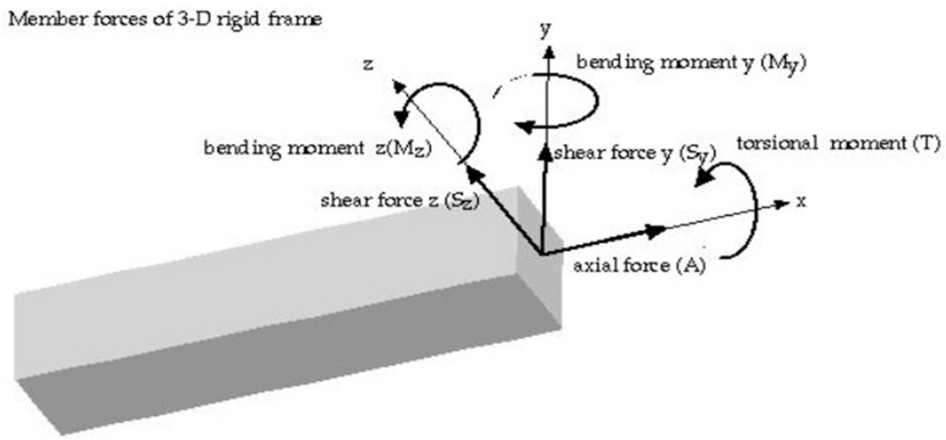


Figure 10. Relationship between different forces and moments (NPTEL n.d.).

4.3.9 Characteristic load

Characteristic loads are the ones that have 95 % probability of not being exceeded during the service life of a material or the maximum load that it could take during its lifespan. The characteristic load is the ultimate load that is liable to come on structure during its lifetime. (Civil Engineering blog 2017)

4.3.10 Design load

Loads are a primary consideration in any building design because they define the nature and magnitude of hazards or external forces that a building must resist to provide reasonable performance (i.e., safety and serviceability) throughout the structure’s useful life. The anticipated loads are influenced by a building’s intended use (occupancy and function), configuration (size and shape), and location (climate and site conditions). Ultimately, the type and magnitude of design loads affect critical decisions such as material selection, construction details, and architectural configuration. So, design load is the combination of loads such as dead loads, live loads, soil lateral loads, wind loads, snow loads, earthquake loads etc. (Hud User n.d.).

4.3.11 Ultimate limit state

The ultimate limit state is the design for the safety of a structure and its users by limiting the stress that materials experience. Design for the ULS represents a defined process that is aimed at ensuring the probability of collapse of a building (and therefore the risk to human life) is at an acceptable level. In order to comply with engineering demands for strength and stability under design loads, ULS must be fulfilled as an established condition. The ULS is a purely elastic condition, usually located at the upper part of its elastic zone (approximately 15% lower than the elastic limit). This is in contrast to the ultimate state (US) which involves excessive deformations approaching a structural collapse and is located deeply within the plastic zone. (Designing building wiki 2018).

4.3.12 Service limit state

The service limit state is the design to ensure that a structure is comfortable and useable. This includes vibrations and deflections (movements), as well as cracking and durability. The SLS represents a level of stress or strain within the building below which there is a high expectation the building can continue to be used as originally intended without repair. These are the conditions that are not strength-based but still may render the structure unsuitable for its intended use, for example, it may cause occupant discomfort under routine conditions. A structure must remain functional for its intended use subject to routine loading in order to satisfy SLS criterion. (Designing building wiki 2018).

4.3.13 Shrinkage and creep

If any material is exposed to air, it starts to evaporate the moisture it contains inside. As it dries over time, it starts to shrink in length, width, height or volume, which in further termed as shrinkage. Creep mostly found in Concrete is defined as deformation of structure under sustained load. Basically, long term pressure or stress on concrete can make it change shape. This deformation usually occurs in the direction the force is being applied, e.g. a concrete column getting more compressed, or a beam bending. Creep does not necessarily cause concrete to fail or break apart. Creep is factored in when concrete structures are designed. The amount of creep that the concrete undergoes is dependent upon the magnitude of the sustained loading, the age and strength of the concrete when the stress is applied and the total amount of time that the concrete is stressed. (Penn State College of Engineering n.d.).

4.3.14 **Deflection**

Deflection means the state of deformation of a material mostly the beam in any structures from its original shape under the work of a force or load or weight. One of the most important applications of beam deflection is to obtain equations with which it is possible to determine the accurate values of beam deflections in many practical cases. Deflections are also used in the analysis of statically indeterminate beams. Several methods are available for determining beam deflections such as Direct Integration Method, Area moment Method, Conjugate Beam Method, Method of Superposition and so on. (Civil Engineering n.d.).

4.3.15 **Crack control**

Cracks usually occur in concrete structures. When concrete is subjected to shrinkage, a need for deformation will occur. If this deformation is restrained, restraint forces will appear. When a concrete member is cast against an existing concrete member restraint forces may occur due to different needs of movement, which may lead to cracking. Even though cracks are natural in reinforced concrete during the service state, they can cause durability problems if they become too large. Guidance of how to design restrained members with regards to crack widths is insufficient in codes used today. (CHALMERS n.d.).

4.3.16 **Slenderness**

Slenderness of an element gives a measure of the load at which a section will yield locally, compared to the load at which the element will buckle. Slenderness is related to the behaviour of an element, like a strut, acting as a whole body. In other words, slenderness is the quotient between the width of a building and its height. In structural engineering, slenderness is a measure of the propensity of a column to buckle. (The chatti guide to structural engineer, 2012).

4.3.17 **Buckling**

Buckling is a mathematical instability that leads to a failure mode. When a structure is subjected to compressive stress, buckling may occur. Buckling is characterized by a sudden sideways deflection of a structural member. This may occur even though the stresses that develop in the structure are well below those needed to cause a failure of the material of which the structure is composed of. As an applied load is increased on a member, such as a column, it will ultimately become large enough to cause the member to become unstable and it is said to have buckled. Mathematically, buckling is a bifurcation problem. At a certain load level, there is more than one solution. (Sönnerlind 2014).

4.3.18 **Bulk density of soil**

Bulk density of soil is defined as the ratio of the mass of dry solids to the bulk volume of the soil occupied by those dry solids. Bulk density of the soil is an important site characterization parameter since it changes for a given soil. It varies with structural condition of the soil, particularly that related to packing. (USDA n.d.). Bulk density is an indicator of soil compaction and soil health. It affects infiltration, rooting depth/restrictions, available water capacity, soil porosity, plant nutrient availability, and soil microorganism activity, which influence key soil processes and productivity. It is the weight of dry soil per unit of volume typically expressed in grams/cm^3. (ORNL DAAC 2018).

4.3.19 **Thermal transmittance**

Thermal transmittance, also known as U-value, is the rate of transfer of heat (in watts) through one square meter of a structure, divided by the difference in temperature across the structure. It is expressed in watts per square meter Kelvin, W/m^{2}K. Well-insulated parts of a building have a low thermal transmittance whereas poorly insulated parts of a building have a high thermal transmittance. Losses due to thermal radiation, thermal convection and thermal conduction are taken into account in the U-value. (The concrete society n.d.).

4.3.20 **Insulation**

Thermal insulation is an important technology to reduce energy consumption in buildings by preventing heat gain/loss through the building envelope. Thermal insulation is a construction material with low thermal conductivity, often less than 0.1W/mK. These materials have no other purpose than to save energy and protect and provide comfort to occupants. One of the most important and cost-effective energy saving materials in building construction is the insulation. Insulation keeps buildings warm in winter and cool in summer. Generally, insulation is installed between the framing members in the home, for instance, walls, ceilings, floors around the perimeter, basements, attics and even interior rooms of the home (Aalto University n.d.). Figure 11 shows the insulation provision of a building.

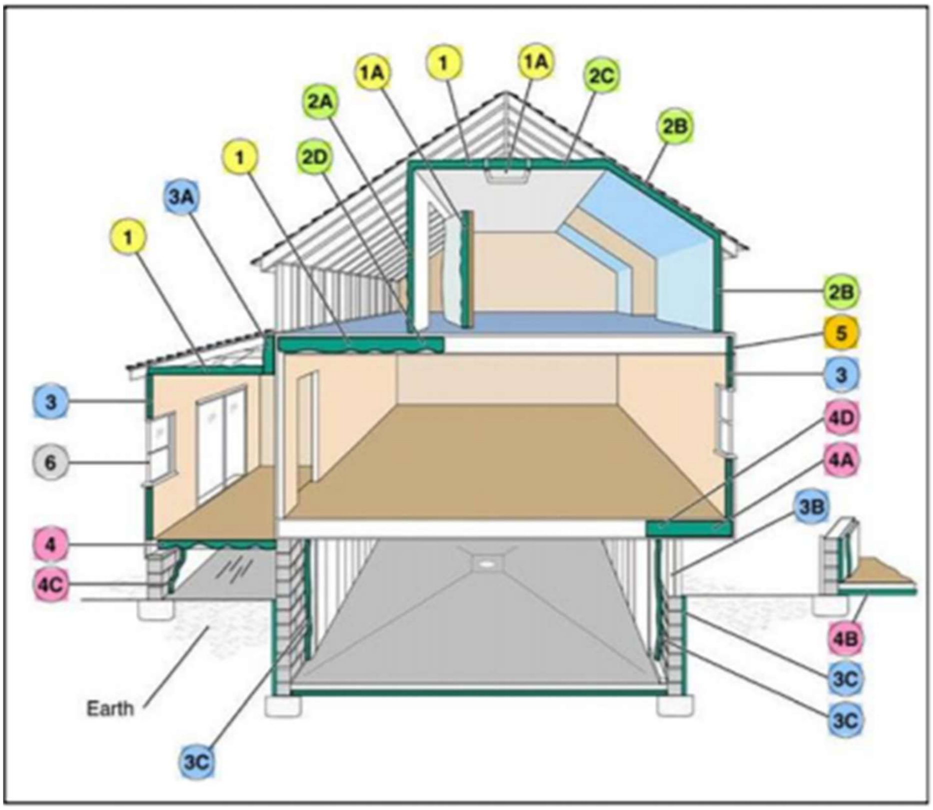


Figure 11. Insulation options of a building (Aalto University n.d.).

4.3.21 Ventilation gap

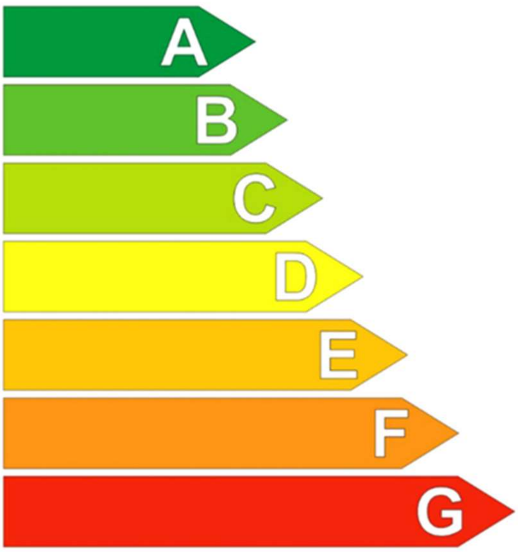
Air gaps or ventilation gaps are often included in the walls of structures such as buildings, ships, refrigerators and ovens, for the thermal insulation which they provide. The insulation of an air gap is dependent on the mean temperature of its bounding surfaces and the best effect is achieved when the gap is at the position where it is at the lowest possible mean temperature. The value of the insulation is afforded by air gaps under various practical conditions. For an air gap to improve the thermal performance of a building element, it requires the addition of a low emittance surface to one or both sides of the air gap. (Passipedia n.d.).

4.3.22 Heat loss

Heat loss is a discipline of thermal engineering that concerns the generation, use, conversion, and exchange of thermal energy (heat) between physical systems. Heat transfer is classified into various mechanisms, such as thermal conduction, thermal convection, thermal radiation, and transfer of energy by phase changes. (The engineering toolbox n.d.).

4.3.23 **Energy efficiency**

Energy efficiency is the ratio between the useful output and input of an energy conversion process. Energy efficiency is defined as the use of energy in an optimum manner to achieve the same service that could have been achieved using a common less efficient manner. Energy efficiency is the practice of reducing the energy requirements while achieving the required energy output. Energy efficiency may also be stated as efficient energy in use. (Corrosionpedia n.d.). In Finland, efforts have been made for decades to produce goods and services with as little energy as possible. The E-value is needed for building permits and statutory energy performance certificates. It can be used to optimize design solutions and it can be used in the retail and the renting of buildings. The E-value is defined by the annual consumption of purchased energy of a building in its intended use. (Green building council Finland n.d.). Categories of energy efficiencies of buildings in Finland is shown in Figure 12.



Energy category	Annual value KWh/m^2
A	≤ 90
B	≤ 170
C	≤ 240
D	≤ 280
E	≤ 340
F	≤ 390
G	≥ 390

Figure 12. Categories of energy efficiency of building structures in Finland (Vänskä 2016)

5 DESIGN MANUAL TO FOLLOW

The design manual provides guidelines, procedures and method of structural design for the construction of any structures.

5.1 Load combination

Load combination suggests the formation of different actions together such as permanent, variable or accidental load. Permanent actions (G) includes self-weight of structures, fixed equipment and so on. Variable actions (Q) refer to imposed loads on building floors, beams and roofs. It also adds wind and snow load. Lastly, explosions, impact from vehicles and so on belong to accidental actions (A). Besides, actions shall also be classified by their origin, nature and spatial variation. (SFS EN 1990+A1+AC. Eurocode, 81). However, the equation for load combination effect found in the mentioned Eurocode is shown below and rights after that, definition of different types of loads are described

$$E = \sum_{j=1} \gamma_{G,j} G_{k,j}{}'' + {}''\gamma_P P'' + {}''\gamma_{Q,1} Q_{k,1}{}'' + {}'' \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$

Where,

$G_{k,j}$	Characteristic value of permanent action j
$Q_{k,1}$	Characteristic value of the leading variable action 1
$Q_{k,i}$	Characteristic value of the accompanying variable action i
P	Relevant representative value of a prestressing action
$\gamma_{G,j}$	Partial factor for permanent action j
$\gamma_{Q,1}$	Partial factor for leading variable action 1
$\gamma_{Q,i}$	Partial factor for variable action i
γ_P	Partial factor for prestressing action
$\psi_{0,i}$	Factor for combination value of variable action i
''+''	Implies ''to be combined with''
∑	Implies ''the combined effect of''

According to SFS EN 1990+A1+AC. Eurocode, 87, the values of ψ factors should be specified and therefore, it is shown in Table 1 below.

Table 1. Recommended values of ψ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A: domestic, residential areas	0,7	0,5	0,3
Category B: office areas	0,7	0,5	0,3
Category C: congregation areas	0,7	0,7	0,6
Category D: shopping areas	0,7	0,7	0,6
Category E: storage areas	1,0	0,9	0,8
Category F: traffic area,			
vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G: traffic area,	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. [*] For countries not mentioned below, see relevant local conditions.			

Nevertheless, in this thesis the following formula is used for calculating the combination effect of loads

$$E = \sum_{j=1} \gamma_{G,j} G_{k,j}{}'' + {}''\gamma_{Q,i} Q_{k,i}$$

According to SFS EN 1990+A1+AC. Eurocode, 89, the values of γ factors should be specified and therefore, it is shown in Table 2 below.

Table 2. Design values of γ factors for buildings

Persistent and transient design situations	Permanent actions		Leading variable action (*)	Accompanying variable actions	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	$\gamma_{G,j,sup} G_{k,j,sup}$	$\gamma_{G,j,inf} G_{k,j,inf}$	$\gamma_{Q,1} Q_{k,1}$		$\gamma_{Q,i} \psi_{0,i} Q_{k,i}$

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

NOTE 1 The γ values may be set by the National annex. The recommended set of values for γ are:

$\gamma_{Gj,sup} = 1{,}10$
$\gamma_{Gj,inf} = 0{,}90$
$\gamma_{Q,1} = 1{,}50$ where unfavourable (0 where favourable)
$\gamma_{Q,i} = 1{,}50$ where unfavourable (0 where favourable)

NOTE 2 In cases where the verification of static equilibrium also involves the resistance of structural members, as an alternative to two separate verifications based on Tables A1.2(A) and A1.2(B), a combined verification, based on Table A1.2(A), may be adopted, if allowed by the National annex, with the following set of recommended values. The recommended values may be altered by the National annex.

$\gamma_{Gj,sup} = 1{,}35$
$\gamma_{Gj,inf} = 1{,}15$
$\gamma_{Q,1} = 1{,}50$ where unfavourable (0 where favourable)
$\gamma_{Q,i} = 1{,}50$ where unfavourable (0 where favourable)

provided that applying $\gamma_{G,j,inf} = 1{,}00$ both to the favourable part and to the unfavourable part of permanent actions does not give a more unfavourable effect. <AC|

5.1.1 Dead load

Dead loads are those which are constant in magnitude and fixed in location throughout the lifetime of the structure. Major part of the dead load is the weight of the structure itself. Dead load is calculated with good accuracy from the design configuration, dimension of the structure, and density of the material. Floor fill, finish floors, and plastered ceilings are included as dead loads for buildings. Wearing surfaces, sidewalks, and curbs are included as dead loads for bridges. (Darwin, Dolan & Nilson 2016).

5.1.2 Live load

Live loads are occupancy loads in buildings and traffic loads on bridges. Their magnitude and distribution at any given time is uncertain. Their maximum intensities throughout the lifetime of the structure are unknown with precision. They may be either fully or partially in place or not present at all. (Darwin, Dolan & Nilson 2016).

5.1.3 Environmental loads

Environmental load consists mainly of snow loads, wind pressure and suction, earthquake load effects, soil pressure on subsurface portions of structures, loads from possible ponding of rain water on flat surfaces, and forces caused by temperature differentials. Environmental loads at any given time are uncertain in both magnitude and distribution. (Darwin, Dolan & Nilson 2016).

The values of imposed loads as mentioned in page 2, NAS. EN 1991-1-1. Eurocode 1, are shown in Table 3.

Table 3. Values of imposed load

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

5.2 Snow load

Snow load is caused by the self-weight of the accumulated snow and ice acting downward force on a building's roof. Factors like the shape of the roof, thermal properties, nearby buildings or surrounding terrain can affect the natural snow load (SFS EN 1991-1-3. Eurocode 1, 17-18). Values of C_e for different topographies are shown in Table 4.

Table 4. Values of C_e for different topographies

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.	

5.2.1 Monopitch roof

A single sloped roof surface pitched in two different directions is generally named as a mono-pitched or gabled roof. The lowest coefficient for such roof should not be reduced below 0.8.

Figure 13 below shows how the snow load shape coefficient is determined and Table 5 shows the conditions for considering snow load coefficient.

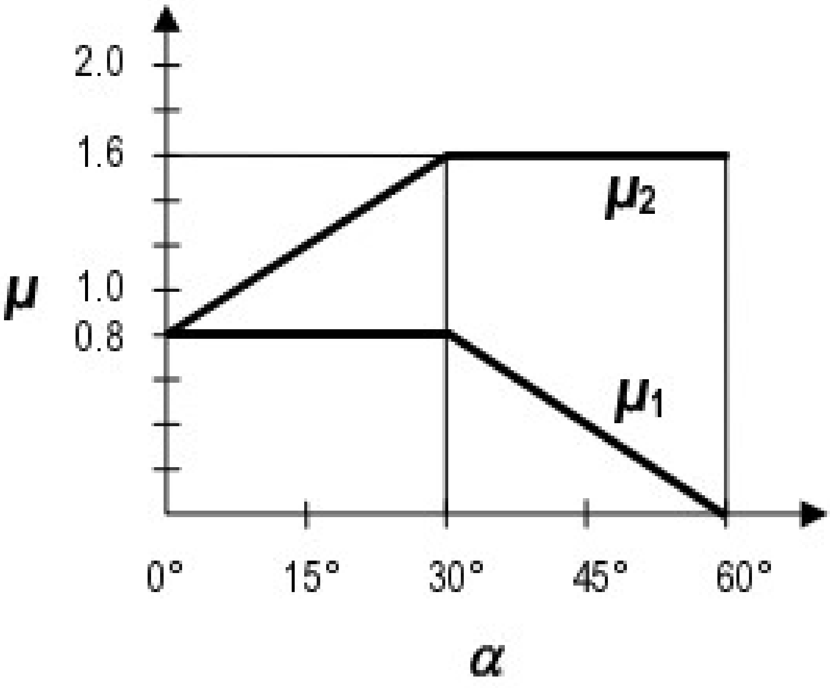


Figure 13. Snow load shape coefficient

Table 5. Snow load coefficient

Angle of pitch of roof α	$0° \leq \alpha \leq 30°$	$30° < \alpha < 60°$	$\alpha \geq 60°$
$\mu_1(\alpha)$	$\mu_1(0°) \geq 0{,}8$	$\mu_1(0°)\frac{(60° - \alpha)}{30°}$	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	--

5.2.2 **Undrifted snow load on the roof**

Undrifted snow load on the roof is defined as ''load arrangement which describes the uniformly distributed snow load on the roof, affected only by the shape of the roof, before any redistribution of snow due to other climatic actions.'' (SFS EN 1991 Eurocode 1. Part 1-3, 10.).

5.2.3 **Drifted snow load on the roof**

Drifted snow load means exactly opposite to the undrafted. The bulk weight density of snow varies on the ground. Generally, it depends on snow cover, duration, location, climate, altitude and so on (SFS EN 1991-

1-3. Eurocode 1, 10, 22). Figure 14 shows the snow load arrangement for both undrafted and drifted snow load.

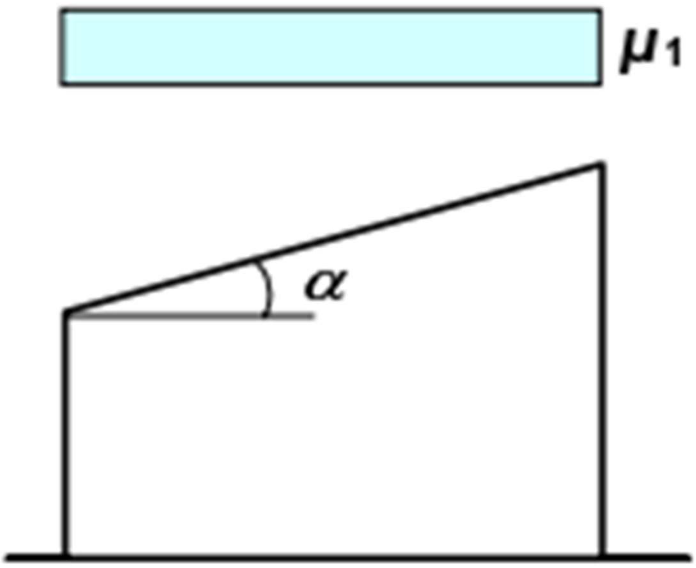


Figure 14. Undrifted and drifted snow load arrangement on monopitch roof

5.3 Wind load

The mean wind force based on the mean wind speed and fluctuating wind force based on a fluctuating flow field act on a building. The effect of fluctuating wind force on a building or part thereof depends on the characteristics of fluctuating wind force and on the size and vibration characteristics of the building or part thereof. Each wind load is determined by a probabilistic-statistical method based on the concept of equivalent static wind load, on the assumption that structural frames and components/cladding behave elastically in strong wind. In the below, Table 6 lists the mean bulk weight density of snow (SFS EN 1991-1-3. Eurocode 1, 55).

Table 6. Mean bulk weight density of snow

Type of snow	Bulk weight density [kN/m^3]
Fresh	1,0
Settled (several hours or days after its fall)	2,0
Old (several weeks or months after its fall)	2,5 - 3,5
Wet	4,0

Figure 15 shows a picture of snow load on the ground in Finland (NAS. EN 1991-1-3. Eurocode 1, 3), Table 7 shows various terrain categories and parameters (SFS EN 1991-1-4. Eurocode 1, 20).

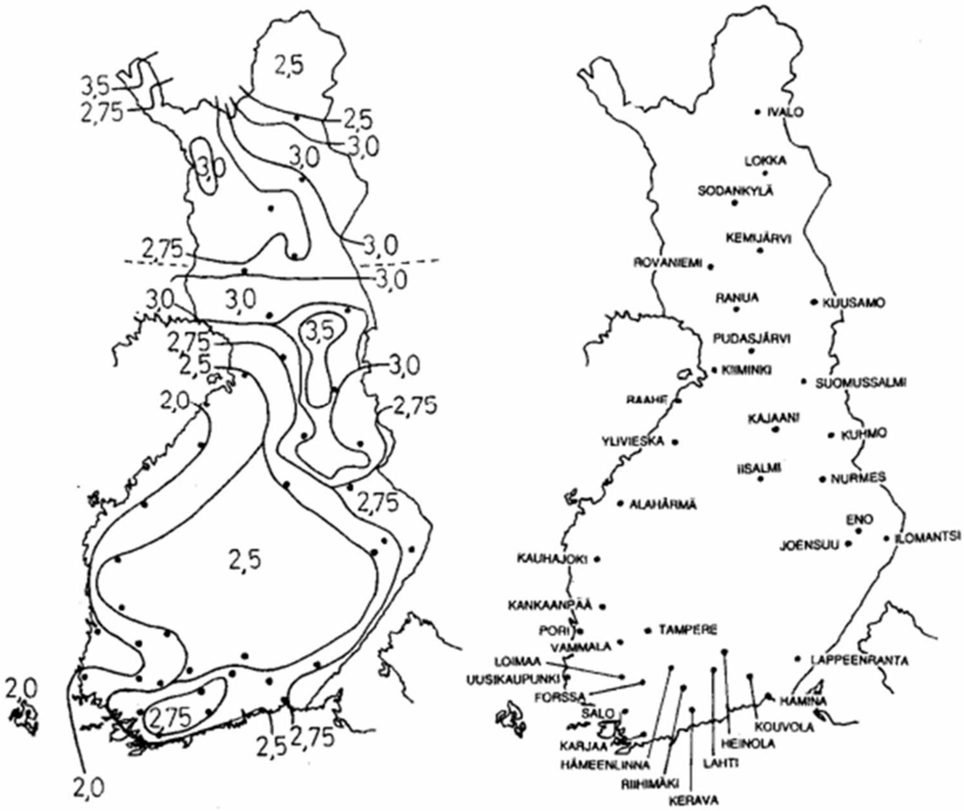


Figure 15. Snow load map on the ground in Finland

Table 7. Terrain categories and parameters

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

Figure 16 illustrates the exposure factor for c_0 and k_1 (SFS EN 1991-1-4. Eurocode 1, 23) and Figure 17 tells how wind pressures act on different surfaces of a building (SFS EN 1991-1-4. Eurocode 1, 25).

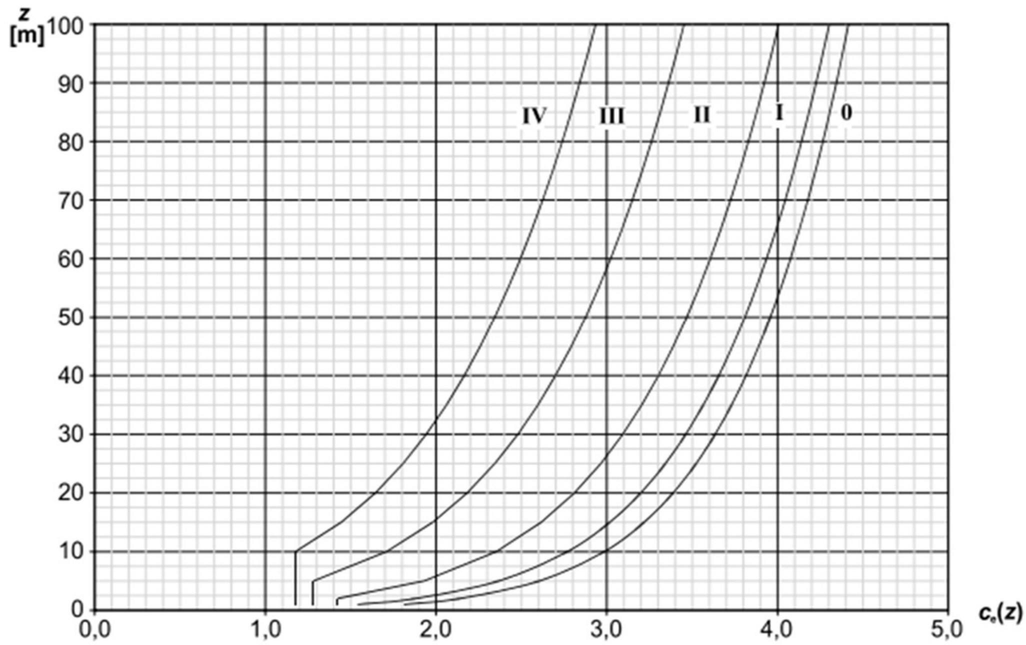


Figure 16. Illustrations of the exposure factor for c_o=1; k_1=1

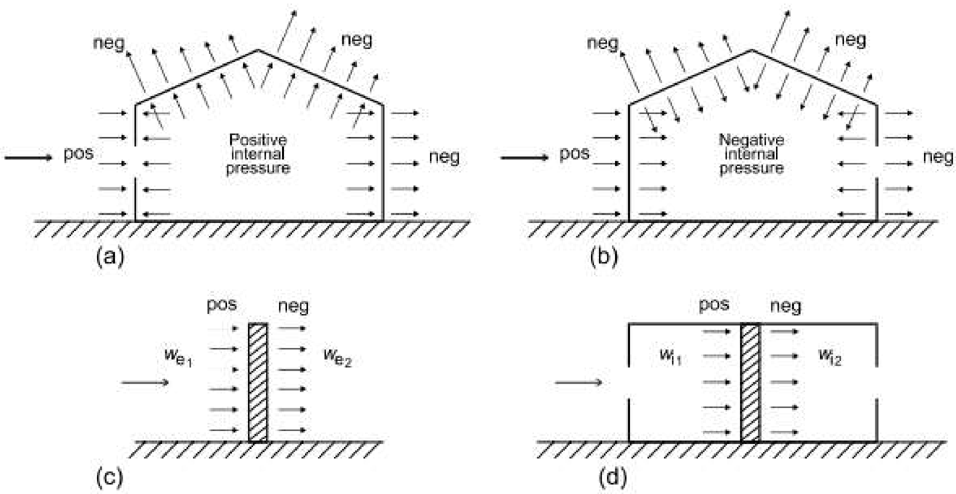


Figure 17. Pressure on surfaces

5.3.1 **Basic values of wind velocity**

NAS. EN 1991-1-4. Eurocoede 1, 2 states that in Finland, the following values are used for the fundamental value of the basic wind velocity $v_{b,0}$:

- Mainland in the entire country, $v_{b,0}$ = 21 m/s
- Sea areas: open sea, scattered islands out in the open sea, $v_{b,0}$ = 22 m/s
- In Lappland: at the top of mountains, $v_{b,0}$ = 26 m/s
- In Lappland: at the bottom of mountains, $v_{b,0}$ = 21 m/s

5.3.2 Determination of c_sc_d

As per (SFS EN 1991-1-4. Eurocode 1, 28), c_sc_d =1 if

- Buildings height ≤ 15 m
- Frequency of facade and roof elements > 5 Hz
- Framed buildings height < 100 m with structural wall height < 4 times in wind depth
- Cross sectional height of a circular chimney <60 m and < 6.5 times of the diameter

However, as stated in SFS EN 1991-1-4. Eurocode 1, 66, measuring process of force coefficient factor is shown in Figure 18.

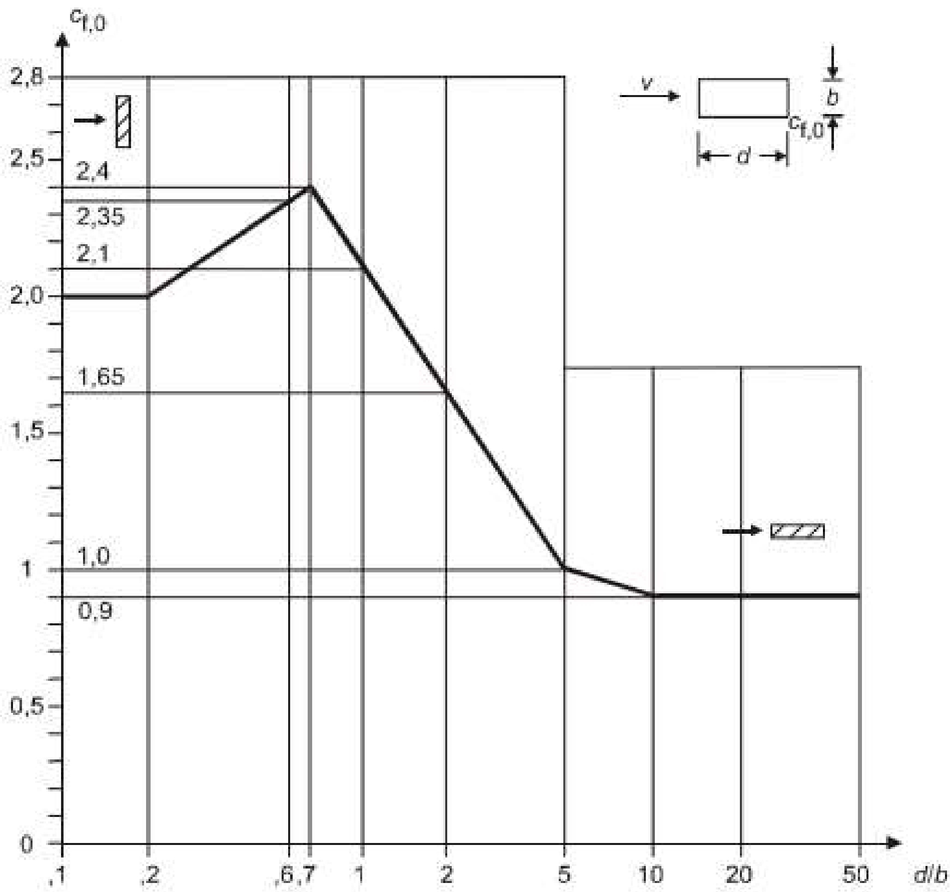


Figure 18. Force coefficient $c_{f,10}$ of rectangular sections with sharp corners and without free end flow

Figure 19 shows the procedure of determining force coefficient factor (SFS EN 1991-1-4. Eurocode 1, 66). Again, regarding SFS EN 1991-1-4. Eurocode 1, 80, Table 8 points out the values of slenderness, λ.

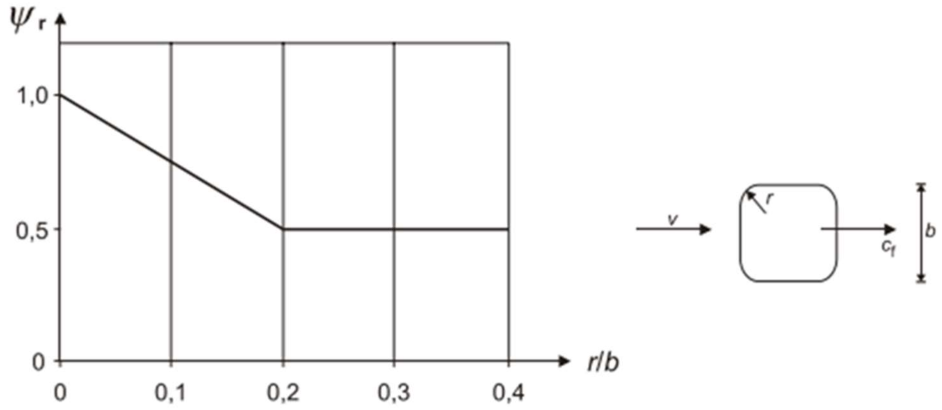


Figure 19. Reduction factor Ψ_r for a square cross-section with rounded corners

Table 8. Recommended values of λ for cylinders, polygonal sections, rectangular sections, sharp edged structural sections and lattice structure

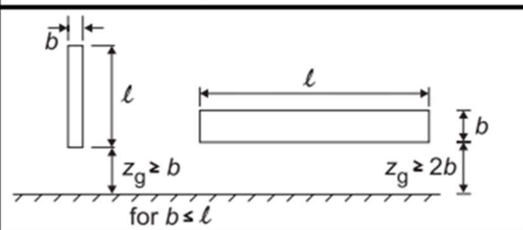
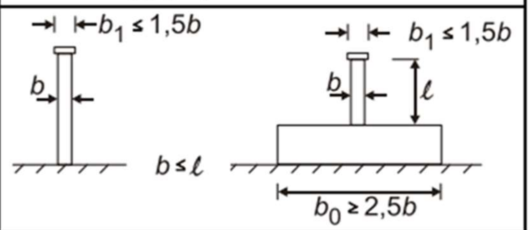
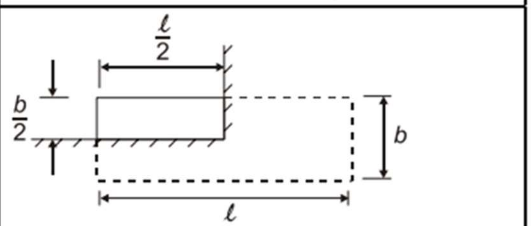
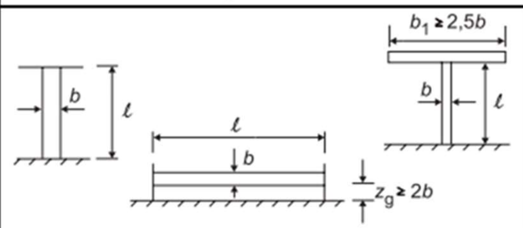
No.	Position of the structure, wind normal to the plane of the page	Effective slenderness λ
1		For polygonal, rectangular and sharp edged sections and lattice structures: for $\ell \geq 50$ m, $\lambda = 1{,}4\ \ell/b$ or $\lambda = 70$, whichever is smaller for $\ell < 15$ m, $\lambda = 2\ \ell/b$ or $\lambda = 70$, whichever is smaller For circular cylinders: for $\ell \geq 50$, $\lambda = 0{,}7\ \ell/b$ or $\lambda = 70$, whichever is smaller for $\ell < 15$ m, $\lambda = \ell/b$ or $\lambda = 70$, whichever is smaller For intermediate values of ℓ, linear interpolation should be used
2		
3		
4		for $\ell \geq 50$ m, $\lambda = 0{,}7\ \ell/b$ or $\lambda = 70$, whichever is larger for $\ell < 15$ m, $\lambda = \ell/b$ or $\lambda = 70$, whichever is larger For intermediate values of ℓ, linear interpolation should be used

Figure 20 shows Indicative values of the end-effect factor ψ_λ as a function of solidity ratio ϕ versus slenderness λ (SFS EN 1991-1-4. Eurocode 1, 80 & 81).

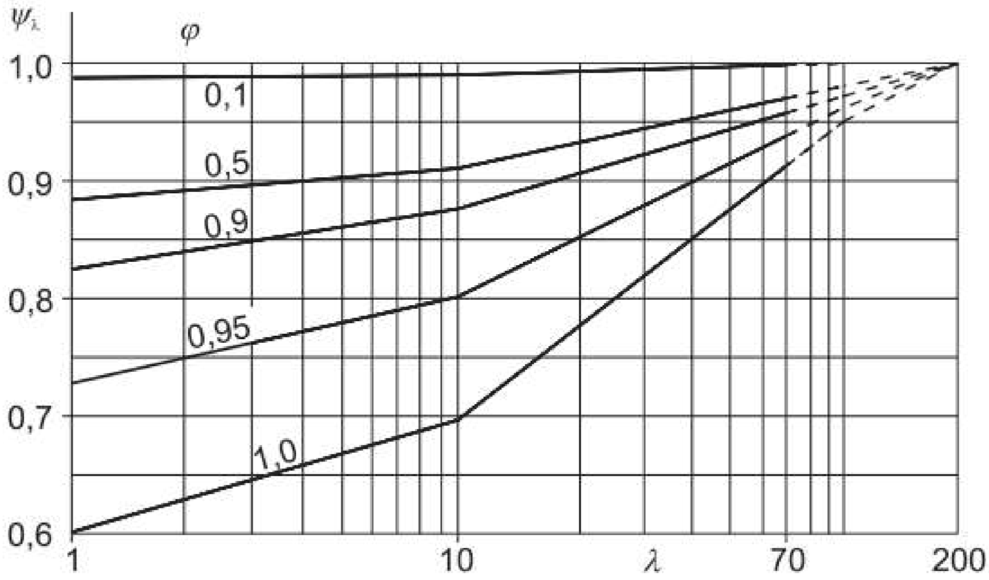


Figure 20. Indicative values of the end-effect factor Ψ_λ as a function of solidity ratio φ versus slenderness λ

5.3.3 **Pressure coefficients for buildings**

The external pressure coefficients generally depend on the size e.g. for loaded areas *A* of 1 m^2 and 10 m^2 for the appropriate building configurations as $c_{pe,1}$, for local coefficients, and $c_{pe,10}$, for overall coefficients, respectively (SFS EN 1991-1-4. Eurocode 1, 33). Figure 21 shows the mathematical ways of calculating the local and overall coefficient.

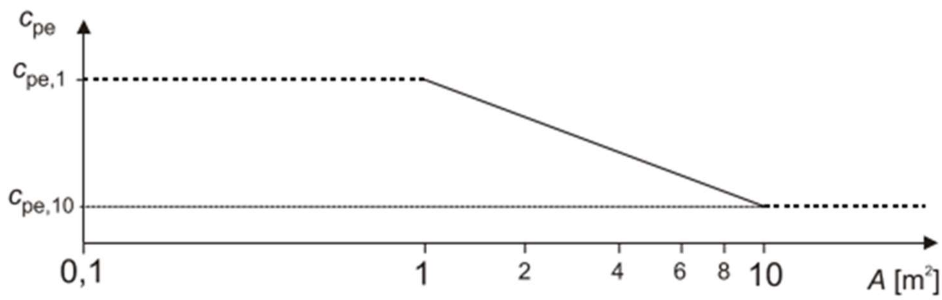


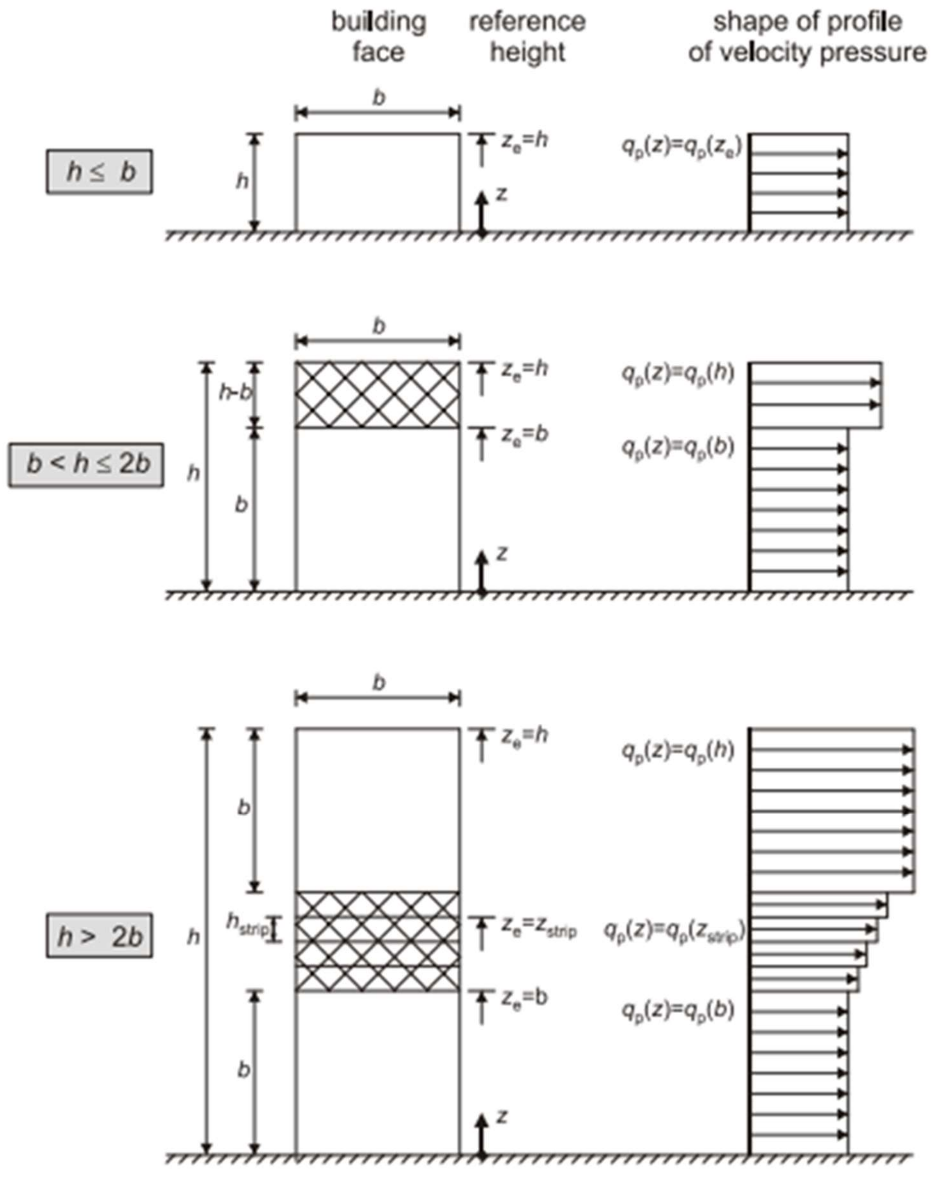
Figure 21. Local and overall external pressure coefficient

5.3.4 **Vertical walls of rectangular plan buildings**

The reference heights, z_e for windward walls of a rectangular plan building depends on the aspect ratio h/b and are always the upper heights of the different parts of the walls and also follow the given conditions given below (SFS EN 1991-1-4. Eurocode 1, 34):

- A building is considered one part when $h \leq b$
- A building is considered two parts when $b < h \leq 2b$
- A building is considered multiple parts when $h > 2b$

Figure 22 depicts how to assume the reference height corresponding to the original height of a vertical wall (SFS EN 1991-1-4. Eurocode 1, 35).



NOTE The velocity pressure should be assumed to be uniform over each horizontal strip considered.

Figure 22. Conditions for reference height of vertical walls

Figure 23 shows the guidelines for choosing elevation corresponding to the width and height of a building and Table 9 provides the recommended values of external pressure coefficient for vertical walls of rectangular plan buildings, adapted from (SFS EN 1991-1-4. Eurocode 1, 36 & 37).

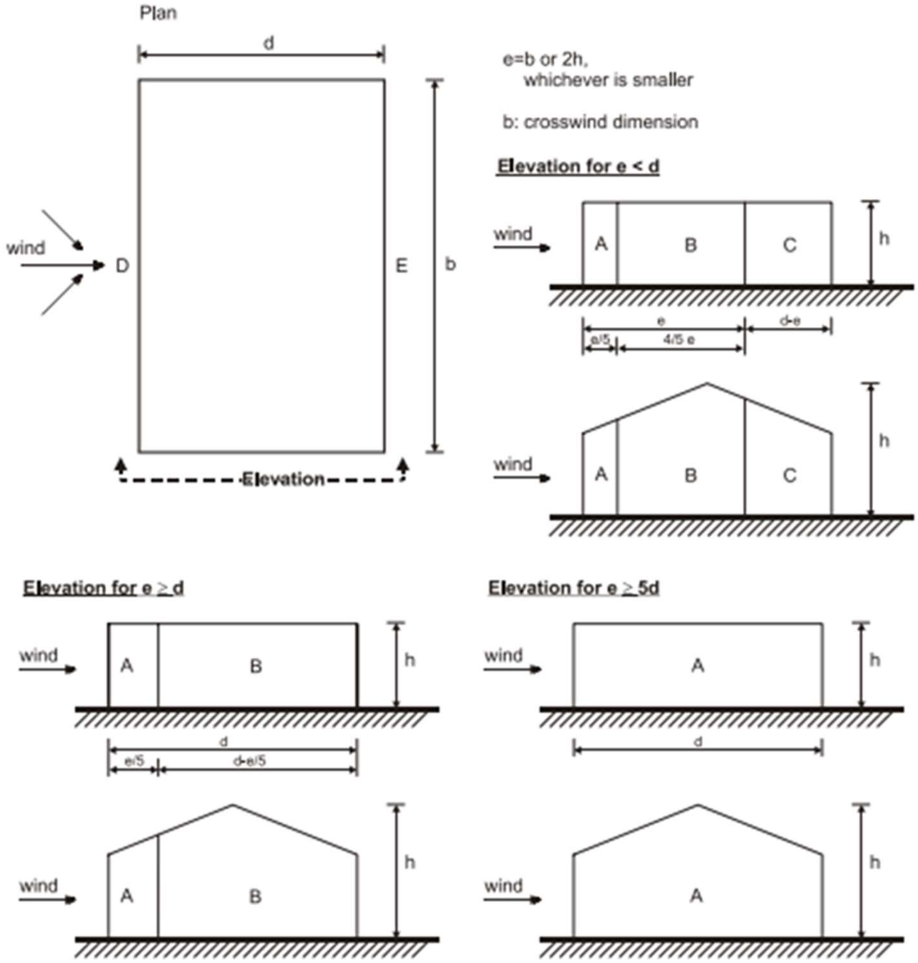


Figure 23. Key for vertical walls

Table 9. Recommended values of external pressure coefficients for vertical walls of rectangular plan buildings

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

Wind direction and angle of pitch are the key roles for determining wind load. In the below, Figure 24 shows some key features for a mono pitch roof (SFS EN 1991-1-4. Eurocode 1, 41).

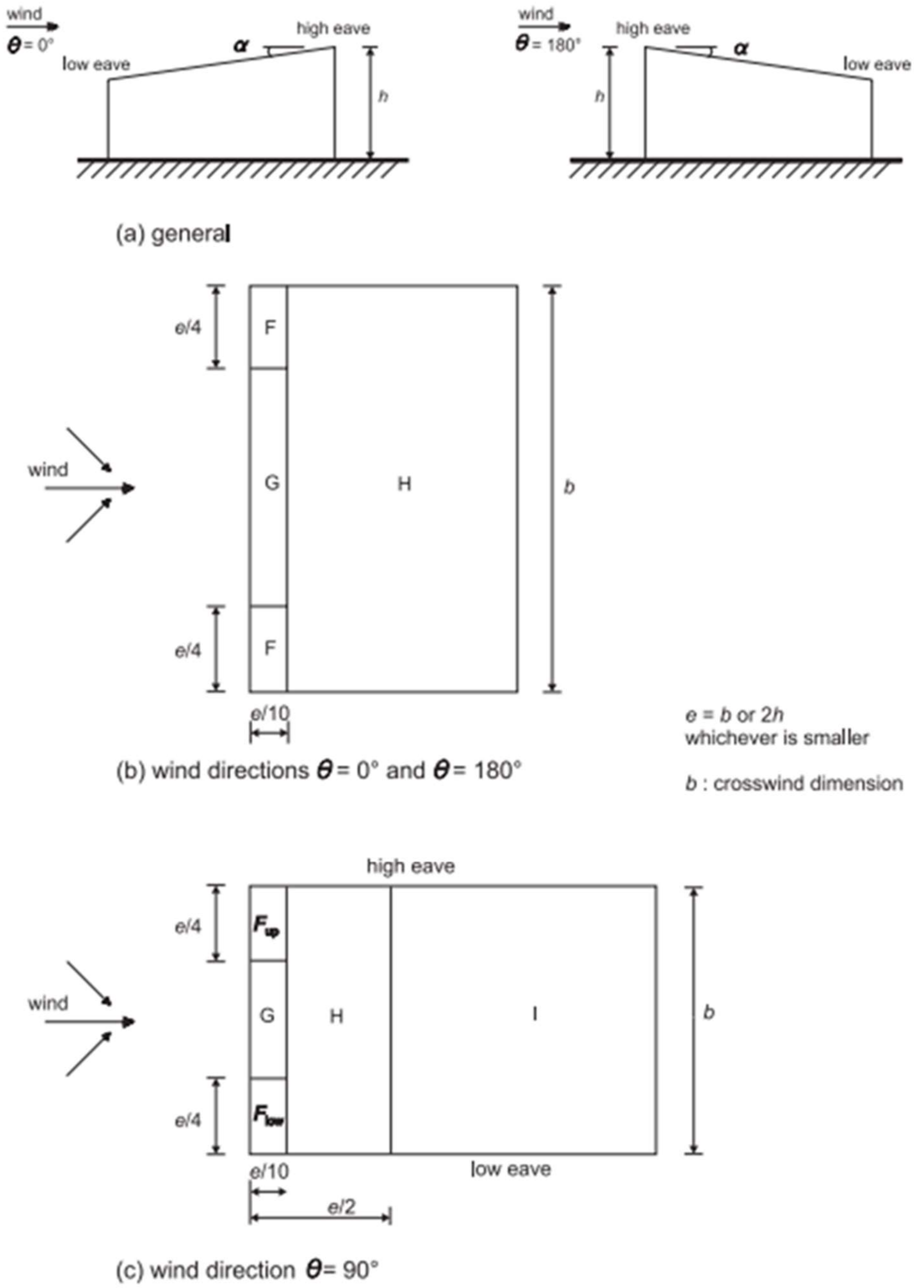


Figure 24. Key for monopitch roof

Tables 10 and 11 show the pressure coefficient for monopitch roof, free standing and parapets (SFS EN 1991-1-4. Eurocode 1, 40).

Table 10. External pressure coefficient for monopitch roof

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

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

Table 11. Recommended pressure coefficients $c_{p,net}$ for free-standing and parapets

Solidity	Zone		A	B	C	D
φ = 1	Without return corners	$\ell/h \leq 3$	2,3	1,4	1,2	1,2
		$\ell/h = 5$	2,9	1,8	1,4	1,2
		$\ell/h \geq 10$	3,4	2,1	1,7	1,2
	with return corners of length $\geq h^{a}$		2,1	1,8	1,4	1,2
φ = 0,8			1,2	1,2	1,2	1,2

Figure 25 illustrates different zones of free standing walls and parapets and Table 12 states the frictional coefficient for different structural members and (SFS EN 1991-1-4. Eurocode 1, 41).

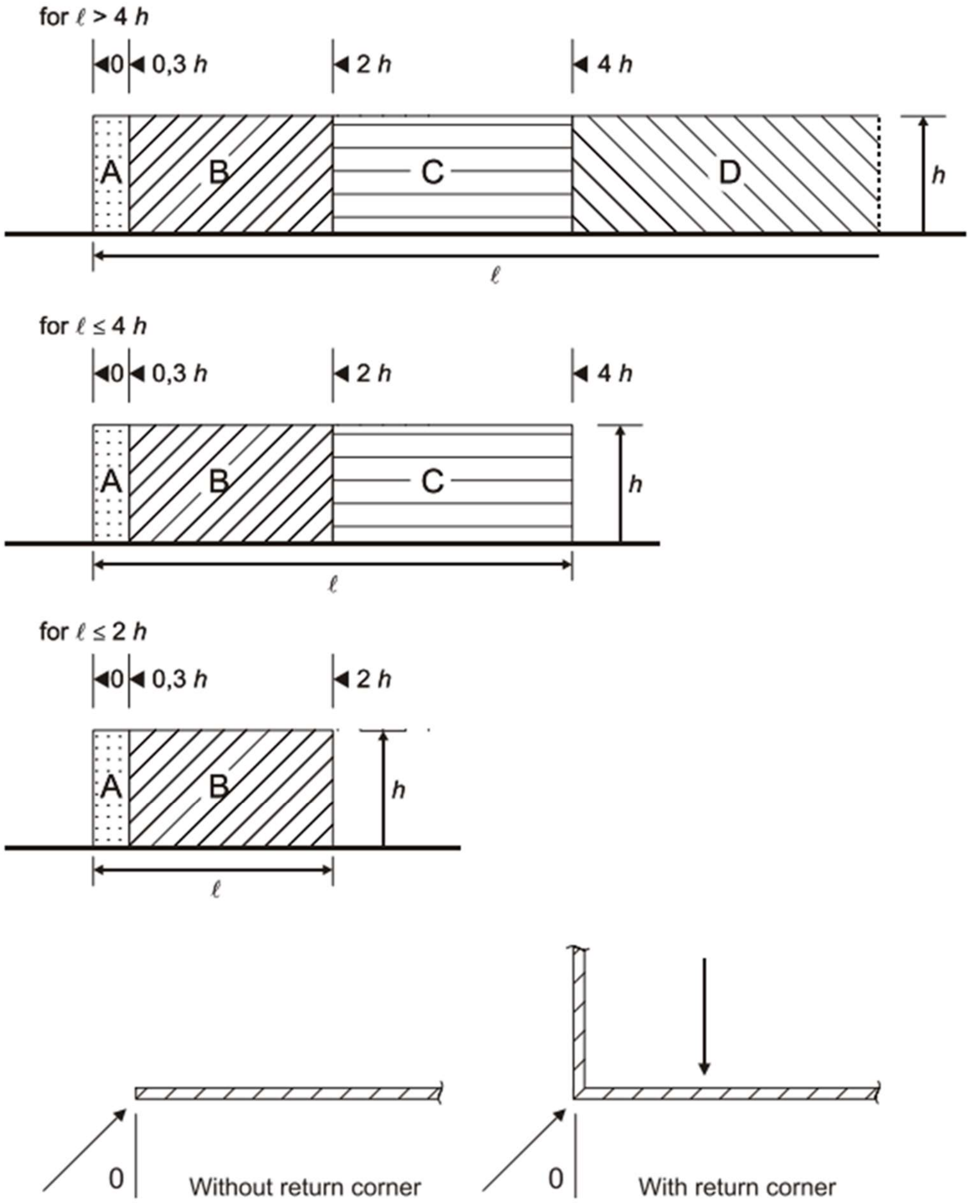


Figure 25. Key to zones of free-standing walls and parapets

Table 12. Frictional coefficients c_{fr} for walls, parapets and roof structures

Surface	**Friction coefficient c_{fr}**
Smooth (i.e. steel, smooth concrete)	0,01
Rough (i.e. rough concrete, tar-boards)	0,02
very rough (i.e. ripples, ribs, folds)	0,04

Figure 26 shows the picture of the reference area of friction, adapted from (SFS EN 1991-1-4. Eurocode 1, 41).

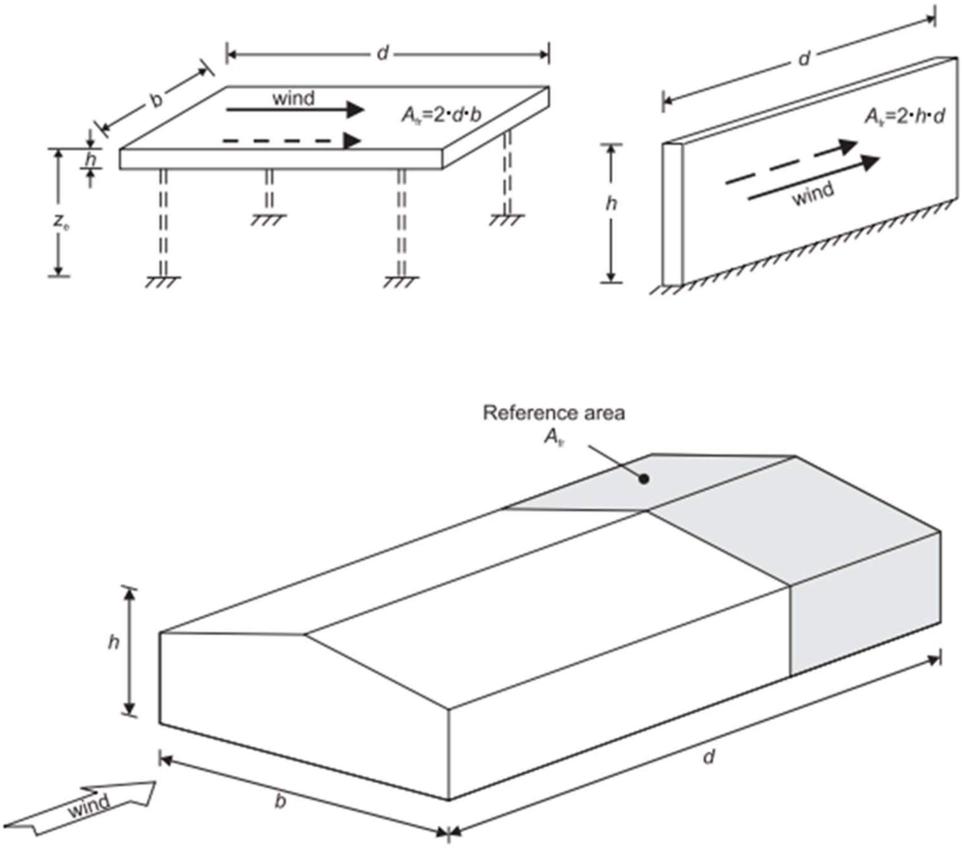


Figure 26. Reference area of friction

5.4 Timber truss design

Load duration class, loading pattern, service class, partial factors of materials, limiting value for deflections, strength class and characteristics value, modification factor, deformation factor of timber and so on are usually taken into account for limit state design of the timbers structure. Table 13 shows the load duration classes of timber (SFS EN 1995-1-1. Eurocode 5, 21).

Table 13. Load duration classes of timber

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	

Table 14 shows the requirement of loading corresponding to the classes (NAS. SFS EN 1995-1-1. Eurocode 5, 2). Tables 15 and 16 show the examples of service classes of structures and partial factor of material properties respectively, adapted from (NAS. SFS EN 1995-1-1. Eurocode 5, 2 & 3).

Table 14. Example of load duration assignment

Load-duration class	Loading
Permanent	Self-weight Machinery, equipment and lightweight partition walls fixed permanently to the structure Earth pressure
Long-term	Storage loads (category E) Water tank load
Medium-term	Snow Uniformly distributed imposed loads on floors and balconies in categories A - D Imposed loads on garages and trafficable areas (categories F and G) Actions due to moisture variation
Short-term	Imposed loads on stairs Concentrated imposed load (Q_k) Horizontal loads on partition walls and parapets Maintenance load or load caused by persons on a roof (category H) Vehicle loads in category E Actions due to transport vehicles Installation loads
Instantaneous	Wind Accidental action

Table 15. : Example of Service classes of structures

Service Classes	Conditions
Service Class 1	Structures with thermal insulation in heated room
Service Class 2	Dry timber outside. Structure should be in covered and ventilated space as well as be well protected from getting wet
Service Class 3	Structure exposed to weather or in a damp space outside

Table 16. Partial factors γ_M for material properties

Fundamental combinations:	
Solid timber and round timber generally	1,4
Coniferous sawn timber in strength class ≥ C35	1,25
Glued laminated timber, LVL	1,2
Wood-based panels	1,25
Connections	*)
Punched metal plate fasteners:	
- anchorage strength	1,25
- plate (steel) strength	1,1
Accidental combinations	1,0

Table 17 shows the value of deflection limit, adapted from (NAS. SFS EN 1995-1-1. Eurocode 5, 4). Moreover, as per EN 338, Strength classes and different characteristic values of timber are given in Table 18 (Ma 2018, 32)

Table 17. Limiting value for deflections

Structure	w_{inst} 1)	$w_{net,fin}$	w_{fin} 2)
Main girders	$\ell/400$	$\ell/300$	$\ell/200$
Purlins and other secondary girders	-	$\ell/200$	$\ell/150$
Horizontal deflection of the building	-	$H/300$	-

1) Applied only to floor members
2) Relates to precambered structures and curved or angled structures between supports

Table 18. Strength classes and characteristic value of timber

		C14	C16	C18	C20	C22	C24	C27	C30	C35	C40	C45	C50
in MPa													
Bending	$f_{m,k}$	14	16	18	20	22	24	27	30	35	40	45	50
Tension parallel	$f_{t,0,k}$	8	10	11	12	13	14	16	18	21	24	27	30
Tension perp.	$f_{t,90,k}$	0,4	0,5	0,5	0,5	0,5	0,5	0,6	0,6	0,6	0,6	0,6	0,6
Compression	$f_{c,0,k}$	16	17	18	19	20	21	22	23	25	26	27	29
Compr. perp.	$f_{c,90,k}$	2,0	2,2	2,2	2,3	2,4	2,5	2,6	2,7	2,8	2,9	3,1	3,2
Shear	$f_{v,k}$	1,7	1,8	2,0	2,2	2,4	2,5	2,8	3,0	3,4	3,8	3,8	3,8
in GPa													
Mean MOE	$E_{0,mean}$	7	8	9	9,5	10	11	11,5	12	13	14	15	16
5% MOE	$E_{0,05}$	4,7	5,4	6,0	6,4	6,7	7,4	7,7	8,0	8,7	9,4	10,0	10,7
Mean MOE perp.	$E_{90,mean}$	0,23	0,27	0,30	0,32	0,33	0,37	0,38	0,40	0,43	0,47	0,50	0,53
Mean shear mod.	G_{mean}	0,44	0,5	0,56	0,59	0,63	0,69	0,72	0,75	0,81	0,88	0,94	1,00
in kg/m³													
Density	ρ_k	290	310	320	330	340	350	370	380	400	420	440	460
Mean density	ρ_{mean}	350	370	380	390	410	420	450	460	480	500	520	550

NOTE: The tabulated properties are compatible with timber at a moisture content consistent with the temperature of 20°C and relative humidity of 65%. Bending and tension parallel to grain strengths are given for timber width 150 mm, tension strength perpendicular to grain for reference volume 0,01 m³.

Table 19 shows the modification factor for the duration of load and moisture and Table 19 shows the deformation factor (SFS EN 1995-1-1. Eurocode 5, 27 6 28)

Table 19. Values of k_{mod}

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

Table 20. Values of k_{def}

Material	Standard	Service class		
		1	2	3
Solid timber	EN 14081-1	0,60	0,80	2,00
Glued Laminated timber	EN 14080	0,60	0,80	2,00
LVL	EN 14374, EN 14279	0,60	0,80	2,00
Plywood	EN 636 Part 1 Part 2 Part 3	 0,80 0,80 0,80	 – 1,00 1,00	 – – 2,50
OSB	EN 300 OSB/2 OSB/3, OSB/4	 2,25 1,50	 – 2,25	 – –
Particleboard	EN 312 Part 4 Part 5 Part 6 Part 7	 2,25 2,25 1,50 1,50	 – 3,00 – 2,25	 – – – –
Fibreboard, hard	EN 622-2 HB.LA HB.HLA1, HB.HLA2	 2,25 2,25	 – 3,00	 – –
Fibreboard, medium	EN 622-3 MBH.LA1, MBH.LA2 MBH.HLS1, MBH.HLS2	 3,00 3,00	 – 4,00	 – –
Fibreboard, MDF	EN 622-5 MDF.LA MDF.HLS	 2,25 2,25	 – 3,00	 – –

5.4.1 General considerations for assembly

Frame structures shall be analysed such that the deformations of the members and joints, the influence of support eccentricities and the stiffness of the supporting structure are taken into account in the determination of the member forces and moments. Connections are generally to be assumed as pinned rotation if their deformation has no significant effect upon the distribution of member forces and moments then those would be rotationally stiffed. (SFS EN 1995-1-1. Eurocode 5, 33.).

5.4.2 Tension parallel to the grain

SFS EN 1995-1-1, Eurocode 5 discusses about tension in page number 36 in which it is recommended that the following expression needs to be satisfied

$$\sigma_{(t,0,d)} \leq f_{(t,0,d)}$$

Where,

$\sigma_{t,0,d}$	Design tensile stress along the grain
$f_{t,0,d}$	Design tensile strength along the grain

5.4.3 Tension perpendicular to the grain

The effect of the member size should be taken into account.

5.4.4 Compression parallel to the grain

SFS EN 1995-1-1, Eurocode 5 discusses about compression in page number 36 in which it is recommended that the following expression needs to be satisfied

$$\sigma_{c,0,d} \leq f_{c,0,d}$$

Where,

$\sigma_{c,0,d}$	Design compressive stress along the grain
$f_{c,0,d}$	Design compressive strength along the grain

5.4.5 Compression perpendicular to the grain

Compression perpendicular to the grain is discussed in SFS EN 1995-1-1, Eurocode 5, 36-37, in which it is recommended that the following expression needs to be satisfied

$$\sigma_{c,90,d} \leq k_{c,90}\, f_{c,90,d}$$

Where,

$\sigma_{c,90,d}$ Design compressive stress in the contact area perpendicular to the grain

$f_{c,90,d}$ Design compressive strength perpendicular to the grain

$k_{c,90}$ Factor taking into account the load configuration, possibility of splitting and degree of compressive deformation (the value is usually 1)

5.4.6 Bending

SFS EN 1995-1-1, Eurocode 5 discusses about bending in page number 41 in which it is recommended that the following expression needs to be satisfied

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

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

Where,

$\sigma_{m,y,d}$ and $\sigma_{m,z,d}$ Corresponding design bending stress along the axes

$f_{m,y,d}$ and $f_{m,z,d}$ Corresponding design bending strength along the axes

k_m Factor (0.7 for rectangular and 1 for other cross sections)

5.4.7 Shear

SFS EN 1995-1-1, Eurocode 5 discusses about shear in page number 41 in which it is recommended that the following expression needs to be satisfied

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

Where,

τ_d Design shear stress

$f_{v,d}$ Design shear strength for the actual condition

5.4.8 Combined bending and axial tension

SFS EN 1995-1-1, Eurocode 5 discusses about combined bending and axial tension in page number 43 in which it is recommended that the following expression needs to be satisfied

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

5.4.9 **Combined bending and axial compression**

SFS EN 1995-1-1, Eurocode 5 discusses about combined bending and axial compression in page number 43 in which it is recommended that the following expression needs to be satisfied

$$\left(\frac{\sigma_{t,0,d}}{f_{t,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\left(\frac{\sigma_{t,0,d}}{f_{t,0,d}}\right)^2 + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

5.4.10 **Columns subjected to either compression or combined compression and bending**

In this section, relative slenderness of the timber column member plays an important role to satisfy the equations. Different mathematical expression regarding compression or combined compression and bending are stated below (SFS EN 1995-1-1. Eurocode 5, 44-45).

$$\lambda_{rely} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{e,0,k}}{E_{0,05}}}$$

$$\lambda_{relz} = \frac{\lambda_z}{\pi}\sqrt{\frac{f_{e,0,k}}{E_{0,05}}}$$

Where,

$E_{0.05}$	Fifth percentile value of modulus of elasticity
$\lambda_{rel,y}$	Relative slenderness ratio corresponding to bending about y-axis ($\lambda_{rel,y} \leq 0.3\ not\ applicable\ for\ truss$)
$\lambda_{rel,z}$	Relative slenderness ratio corresponding to bending about z-axis ($\lambda_{rel,z} \leq 0.3\ not\ applicable\ for\ truss$)

$$\left(\frac{\sigma_{c,0,d}}{k_{c,y} f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\left(\frac{\sigma_{c,0,d}}{k_{c,z} f_{c,0,d}}\right)^2 + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

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

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

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

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

Where,
β_o is a factor for members within the straightness limits
$\beta_c = 0.2$ for solid timber
$\beta_c = 0.1$ for glued laminated timber and LVL

Figure 27 shows the component of deflection, adapted from (SFS EN 1991-1-4. Eurocode 1, 56) on the other hand, Table 21 shows the limiting value of beam deflections, adapted from SFS EN 1995-1-1. Eurocode 5, 56.

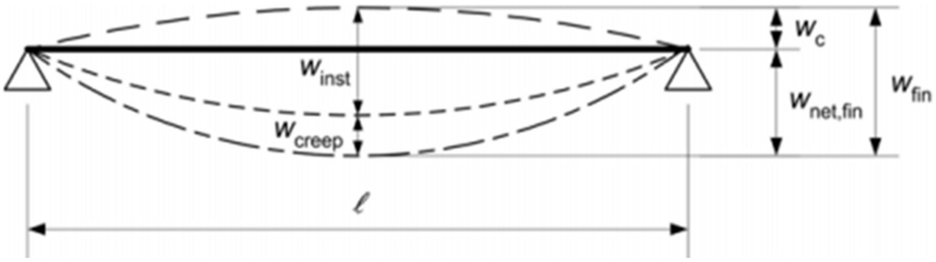


Figure 27. Component of deflection

Table 21. Limiting values for deflections of beam

	w_{inst}	$w_{net,fin}$	w_{fin}
Beam on two supports	ℓ/300 to ℓ/500	ℓ/250 to ℓ/350	ℓ/150 to ℓ/300
Cantilevering beams	ℓ/150 to ℓ/250	ℓ/125 to ℓ/175	ℓ/75 to ℓ/150

5.5 Timber connection with fastener

According to (SFS EN 1995-1-1. Eurocode 5, 59), for one row of fasteners parallel to the grain direction, the effective characteristics load-carrying capacity parallel to the row, should be taken as:

$$F_{v.ef.Rk} = n_{ef} F_{V.Rk}$$

Where,

$F_{v.ef.Rk}$	The effective characteristic load-carrying capacity of one row of fasteners
n_{ef}	The effective number of fasteners in line parallel to the grain.
$F_{V.Rk}$	The characteristic load-carrying capacity of each fastener parallel to the grain.

According to (SFS EN 1995-1-1. Eurocode 5, 61), it is important to take into account the possibility of splitting caused by the tension force component, F_{ed}sinα, perpendicular to the grain, the following shall be satisfied:

$$F_{v,Ed} \leq F_{90,Rd}$$

And

$$F_{v,Ed} = max \begin{cases} F_{v,Ed.1} \\ F_{v,Ed.2} \end{cases}$$

Where,

$F_{90,Rd}$ The design splitting capacity calculated from the characteristic splitting capacity

$F_{90,Rk}$ The characteristic splitting capacity

$F_{v,Ed.1}$, $F_{v,Ed.2}$ The design shear forces on either side of the connection.

For softwoods, the characteristic splitting capacity for the arrangement should be taken as:

$$F_{90,Rk} = 14bw\sqrt{\frac{h_c}{1-\frac{h_c}{h}}}$$

Where:

$$w = \begin{cases} max\begin{cases}\left(\frac{w_{pl}}{100}\right)^{0.35} \\ 1\end{cases} & For\ punched\ metal\ plate\ fasteners \\ 1 & For\ all\ other\ fasterners \end{cases}$$

And

$F_{90,Rk}$ The characteristic splitting capacity, in N

w A modification factor

h_c The load edge distance to the center of the most distant fasteners or to the edge of the punched metal fasteners in mm

h The member height in mm

w_{pl} The width of the punched metal plate fastener parallel to the grain, in mm

5.5.1 **Timber to timber connection**

According to SFS EN 1995-1-1. Eurocode 5, 62, the characteristic capacity for nails, staples, bolts, dowels and screw per shear plane per fastener should be taken as the minimum value found in the following expressions:

For fasteners in single shear:

$$F_{v,Rk} = min\begin{cases} f_{h.1.k}t_1d \\ f_{h.2.k}t_2d \\ \frac{f_{h.1.k}t_1d}{1+\beta}\left[\sqrt{\beta+2\beta^2\left[1+\frac{t_2}{t_1}+\left(\frac{t_2}{t_1}\right)^2\right]+\beta^3\left(\frac{t_2}{t_1}\right)^2}-\beta\left(1+\frac{t_2}{t_1}\right)\right]+\frac{F_{ax,Rk}}{4} \end{cases}$$

$$F_{v,Rk} = min \begin{cases} 1.05\dfrac{f_{h.1.k}t_1 d}{1+\beta}\left[\sqrt{2\beta(1+\beta)+\dfrac{4\beta(2+\beta)M_{y,Rk}}{f_{h.1.k}t_2^2}}-\beta\right]+\dfrac{F_{ax,Rk}}{4} \\ 1.05\dfrac{f_{h.1.k}t_2 d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta)+\dfrac{4\beta(1+2\beta)M_{y,Rk}}{f_{h.1.k}t_2^2}}-\beta\right]+\dfrac{F_{ax,Rk}}{4} \\ 1.15\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk}f_{h.1.k}d}+\dfrac{F_{ax,Rk}}{4} \end{cases}$$

For fasteners in double shear:

$$F_{v,Rk} = min \begin{cases} f_{h.1.k}t_1 d \\ 0.5 f_{h.1.k}t_2 d \end{cases}$$

$$F_{v,Rk} = min \begin{cases} 1.05\dfrac{f_{h.1.k}t_1 d}{1+\beta}\left[\sqrt{2\beta(1+\beta)+\dfrac{4\beta(2+\beta)M_{y,Rk}}{f_{h.1.k}t_2^2}}-\beta\right]+\dfrac{F_{ax,Rk}}{4} \\ 1.15\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk}f_{h.1.k}d}+\dfrac{F_{ax,Rk}}{4} \end{cases}$$

With

$$\beta = \frac{f_{h.2.k}}{f_{h.1.k}}$$

Where,

$F_{v,Rk}$	The characteristic load-carrying capacity per shear plane per fastener
t_i	The timber or board thickness or penetration depth with I either 1 or 2.
$f_{h.1.k}$	The characteristic embedment strength in timber member i
d	The fastener diameter
$M_{y,Rk}$	The characteristic fastener yield moment
β	The ratio between the embedment strength of the members
$F_{ax,Rk}$	The characteristic axial withdrawal capacity of the fastener

The first term on the right hand side is the load-carrying capacity according to the Johansen yield theory, while the second term is the contribution from the rope effect. The contribution to the load-carrying capacity due to the rope effect should be limited to the following percentage of the Johansen part: Round nails 15%, square nails 25%, other nails 50%, screws 100%, bolts 25% and dowels 0%. (SFS EN 1995-1-1. Eurocode 5, 62).

5.5.2 **Nailed connections**

If the characteristic density of the timber is greater than 500 kg/m³ and the diameter *d* of the nail exceeds 8 mm then, timber should be pre-drilled. For smooth nails the pointside penetration length should be at least 8*d* and always, at least two nails in a connection. (SFS EN 1995-1-1. Eurocode 5, 65). However, for smooth nails produced from wire with a minimum tensile strength of 600 N/mm², the following characteristic values for yield moment should be used:

$$M_{y,Rk} = \begin{cases} 0.3 f_u d^{2.6} & for\ round\ nails \\ 0.45 f_u d^{2.6} & for\ square\ nails \end{cases}$$

For nails with diameters up to 8 mm, the following characteristics embedment strengths in timber and LVL apply:

Without predrilled holes,

$$f_{h,k} = 0.082 \rho_k d^{-0.3} \ \ N/mm^2$$

With predrilled holes,

$$f_{h,k} = 0.082(1 - 0.01d)\rho_k \quad N/mm^2$$

Where,

ρ_k The characteristic timber density, in kg/m³
D The nail diameter, in mm

Figure 28 states the meaning of t_1 and t_2, adapted from (SFS EN 1991-1-4. Eurocode 1, 66) and Figure 29 expresses the overlapping situation of nails.

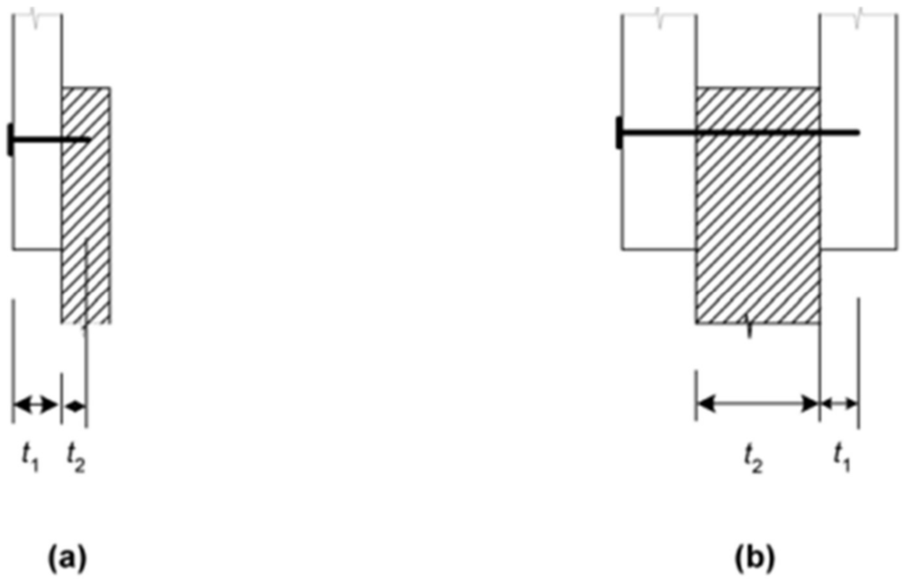


Figure 28. Definitions of t_1 and t_2 where (a) single shear connection and (b) double shear connection

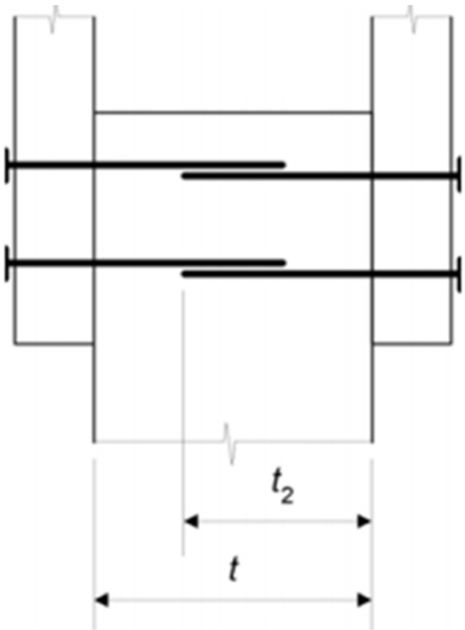


Figure 29. Overlapping nails

$$n_{ef} = n^{k_{ef}}$$

Where,

n_{ef}	The effective number of nails in the row
n	Number of nails in a row
k_{ef}	Interpolation

According to SFS EN 1995-1-1. Eurocode 5, 67, the values of the interpolation of nails are expressed below in Table 22.

Table 22. Values of k_{ef}

Spacing[a]	k_{ef}	
	Not predrilled	Predrilled
$a_1 \geq 14d$	1,0	1,0
$a_1 = 10d$	0,85	0,85
$a_1 = 7d$	0,7	0,7
$a_1 = 4d$	-	0,5
[a] For intermediate spacings, linear interpolation of k_{ef} is permitted		

Acording to SFS EN 1995-1-1. Eurocode 5, 68 & 69, minimum spacing and edge and end distances for nails are shown in Table 23 whereas, positioning of nails both perpendicular and parallel to the grain is shown in Figure 30.

Table 23. Minimum spacings and edge and end distances for nails

Spacing or distance (see Figure 8.7)	**Angle α**	**Minimum spacing or end/edge distance**		
		without predrilled holes		with predrilled holes
		$\rho_k \leq 420$ kg/m^3	420 kg/m^3 < $\rho_k \leq 500$ kg/m^3	
Spacing a_1 (parallel to grain)	$0° \leq \alpha \leq 360°$	d < 5 mm: $(5+5\lvert\cos\alpha\rvert)\,d$ $d \geq 5$ mm: $(5+7\lvert\cos\alpha\rvert)\,d$	$(7+8\lvert\cos\alpha\rvert)\,d$	$(4+\lvert\cos\alpha\rvert)\,d$
Spacing a_2 (perpendicular to grain)	$0° \leq \alpha \leq 360°$	$5d$	$7d$	$(3+\lvert\sin\alpha\rvert)\,d$
Distance $a_{3,t}$ (loaded end)	$-90° \leq \alpha \leq 90°$	$(10+5\cos\alpha)\,d$	$(15+5\cos\alpha)\,d$	$(7+5\cos\alpha)\,d$
Distance $a_{3,c}$ (unloaded end)	$90° \leq \alpha \leq 270°$	$10d$	$15d$	$7d$
Distance $a_{4,t}$ (loaded edge)	$0° \leq \alpha \leq 180°$	d < 5 mm: $(5+2\sin\alpha)\,d$ $d \geq 5$ mm: $(5+5\sin\alpha)\,d$	d < 5 mm: $(7+2\sin\alpha)\,d$ $d \geq 5$ mm: $(7+5\sin\alpha)\,d$	d < 5 mm: $(3+2\sin\alpha)\,d$ $d \geq 5$ mm: $(3+4\sin\alpha)\,d$
Distance $a_{4,c}$ (unloaded edge)	$180° \leq \alpha \leq 360°$	$5d$	$7d$	$3d$

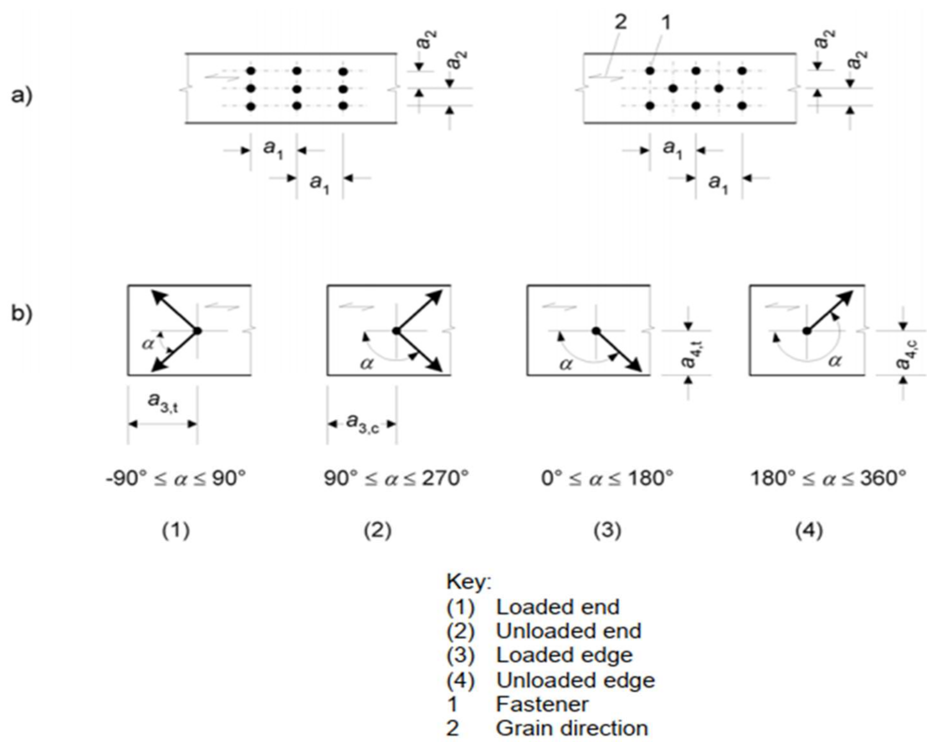


Figure 30. Spacing (a) parallel to the grain in a row and (b) perpendicular to the grain between rows (SFS EN 1991-1-4. Eurocode 1, 69)

Predrill required:

for normal timber, $t \leq max \begin{cases} 7d \\ (13d - 30)\frac{\rho_k}{400} \end{cases}$

for splitting sensitive timber, $t \leq max \begin{cases} 14d \\ (13d - 30)\frac{\rho_k}{200} \end{cases}$

Again, the 2nd formula would be applicable in every case if,

$a_4 \geq 10d$ and $\rho_k \leq 420kg/m^3$

$a_4 \geq 10d$ and $420\ kg/m^3 \leq \rho_k \leq 420\ kg/m^3$

5.5.3 Modified considerations for truss

SFS EN 1995-1-1, Eurocode 5 discusses about truss in page numbers 93-94, in which it is recommended that the following expression needs to be satisfied

$$\left(\frac{\sigma_{c,0,d}}{k_{c,y}f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m\frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 0.9$$

$$\left(\frac{\sigma_{c,0,d}}{k_{c,z}f_{c,0,d}}\right)^2 + k_m\frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 0.9$$

For the strength verification of members in compression and connections, the calculated axial forces should be increased by 10 %.When a simplified analysis is carried out for trusses which are loaded at the nodes, the tensile and compressive stress ratios as well as the connection capacity should be limited to 70 %. All joints should be capable of transferring a force $F_{r,d}$ acting in any direction within the plane of the truss. $F_{r,d}$ should be assumed to be of short-term duration, acting on timber in service class 2, with the value:

$$F_{r,d} = 1.0 + 0.1L$$

5.6 Design of partition wall

Most of the cases, partition walls are found as no-load bearing members. But here, partition walls carry the most load of the building. According to NAS. SFS EN 1992-1-1. Eurocode 2, 3, Table 24 shows partial factors of materials used in ultimate limit state.

Table 24. Partial factors of materials for ultimate limit state

Design situations	γ_C for concrete	γ_S for reinforcing steel	γ_S for prestressing steel
Persistent & Transient	1,5	1,15	1,15
Accidential	1,2	1,0	1,0

Concrete cover is necessary to save the reinforcement from the external environmental conditions. It differs from various matters. According to NAS. SFS EN 1992-1-1. Eurocode 2, 5, some standards are pre-set in this issue regard. Table 25 represents the value of minimum concrete cover.

Table 25. Value of minimum concrete cover

Requirement for value of minimum cover $c_{min,dur}$ according to exposure classes (mm)								
Criteria	Exposure Class according to Table 4.1							
	X0	XC1	XC2 XC3	XC4	XD1	XS1	XD2	XD3 XS2,3
Reinforcing steel	10	10	20	25	30	30	35	40
Prestressing steel	10	20	30	35	40	40	45	50
Design working life 100 years [1)]	+0	+0	+5	+5	+5	+5	+5	+5
Concrete strength ≥	C20/25 -5	C30/37 -5	C35/45 -5	C35/45 -5	C35/45 -5	C40/50 -5	C35/45 -5	C45/55 -5
Construction class 1 (RakMK B4)	-5	-5	-5	-5	-5	-5	-5	-5

The allowed deviation Δc_{dev} is normally 10 mm.

As mentioned earlier in the writing, concrete has the wide range of strength nowadays. The strength class and deformation characteristics for concrete are defined in SFS EN 1992-1-1. Eurocode 2, 29. Table 26 shows the strength classes for concrete.

Table 26. Strength and deformation characteristics for concrete

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	$f_{ctm}=0{,}30\times f_{ck}^{(2/3)} \le$ C50/60 $f_{ctm}=2{,}12\cdot\ln(1+(f_{cm}/10))$ > 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	$f_{ctk;0,05} = 0{,}7\times f_{ctm}$ 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	$f_{ctk;0,95} = 1{,}3\times f_{ctm}$ 95% fractile
E_{cm} (GPa)	27	29	30	31	33	34	35	36	37	38	39	41	42	44	$E_{cm} = 22[(f_{cm})/10]^{0,3}$ (f_{cm} in MPa)
ε_{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 ε_{c1} (‰) $= 0{,}7\ f_{cm}^{0,31} < 2.8$
ε_{cu1} (‰)	3,5									3,2	3,0	2,8	2,8	2,8	see Figure 3.2 for $f_{ck} \ge 50$ Mpa ε_{cu1}(‰)$=2{,}8+27[(98-f_{cm})/100]^4$
ε_{c2} (‰)	2,0									2,2	2,3	2,4	2, 5	2,6	see Figure 3.3 for $f_{ck} \ge 50$ Mpa ε_{c2}(‰)$=2{,}0+0{,}085(f_{ck}-50)^{0,53}$
ε_{cu2} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.3 for $f_{ck} \ge 50$ Mpa ε_{cu2}(‰)$=2{,}6+35[(90-f_{ck})/100]^4$
n	2,0									1,75	1,6	1,45	1,4	1,4	for $f_{ck} \ge 50$ Mpa $n=1{,}4+23{,}4[(90-f_{ck})/100]^4$
ε_{c3} (‰)	1,75									1,8	1,9	2,0	2,2	2,3	see Figure 3.4 for $f_{ck} \ge 50$ Mpa ε_{c3}(‰)$=1{,}75+0{,}55[(f_{ck}-50)/40]$
ε_{cu3} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.4 for $f_{ck} \ge 50$ Mpa ε_{cu3}(‰)$=2{,}6+35[(90-f_{ck})/100]^4$

Tables 27 and 28 show the exposure classes of concrete and their structural classifications. (SFS EN 1992-1-1. Eurocode 2, 48 & 50)

Table 27. Exposure classes related to environmental conditions ‘

Class designation	Description of the environment	Informative examples where exposure classes may occur
1 No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2 Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3 Corrosion induced by chlorides		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete components exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements Car park slabs
4 Corrosion induced by chlorides from sea water		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures
5. Freeze/Thaw Attack		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agents	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation with de-icing agents or sea water	Road and bridge decks exposed to de-icing agents Concrete surfaces exposed to direct spray containing de-icing agents and freezing Splash zone of marine structures exposed to freezing
6. Chemical attack		
XA1	Slightly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water
XA2	Moderately aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water
XA3	Highly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water

Table 28. Recommended structural classifications

Structural Class							
Criterion	**Exposure Class according to Table 4.1**						
	X0	XC1	XC2 / XC3	XC4	XD1	XD2 / XS1	XD3 / XS2 / XS3
Design Working Life of 100 years	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2
Strength Class [1) 2)]	≥ C30/37 reduce class by 1	≥ C30/37 reduce class by 1	≥ C35/45 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C45/55 reduce class by 1
Member with slab geometry (position of reinforcement not affected by construction process)	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control of the concrete production ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

5.6.1 **Creep**

The effect of creep shall be taken into account in second order analysis with due consideration of both the general conditions for creep, and the duration of different loads in the load combination considered. (SFS EN 1992-1-1. Eurocode 2, 68). The duration of loads may be taken into account in a simplified way by means of an effective creep ratio, which used together with the design load, gives a creep deformation (curvature) corresponding to the quasi-permanent load:

$$\varphi_{ref} = \varphi_{(\alpha,t_0)}.M_{0E_{qp}}/M_{0Ed}$$

Where,

$\varphi_{(\alpha,t_0)}$ The final creep coefficient

$M_{0E_{qp}}$ The first order bending moment in quasi-permanent load combination (SLS)

M_{0Ed} The first order bending moment in design load combination (ULS)

If $M_{0E_{qp}}/M_{0Ed}$ varies in a member or structure, the ratio may be calculated for the section with maximum moment, or a representative mean value may be used.

The effect of creep may be ignored, if the following conditions are met

$$\varphi_{(\alpha,t_0)} \leq 2$$
$$\lambda \leq 75$$
$$M_{0Ed}/N_{Ed} \geq h$$

Here,

M_{0Ed} The first order moment and

h The cross section depth in the corresponding direction

Tables 29 and 30 show the relationship between crack control and the thickness of rebar corresponding to bar spacing respectively (SFS EN 1992-1-1. Eurocode 2, 123).

Table 29. Maximum bar diameter ϕ_s for crack control

Steel stress[2] [MPa]	Maximum bar size [mm]		
	w_k= 0,4 mm	w_k= 0,3 mm	w_k= 0,2 mm
160	40	32	25
200	32	25	16
240	20	16	12
280	16	12	8
320	12	10	6
360	10	8	5
400	8	6	4
450	6	5	-

Table 30. Maximum bar spacing for crack control

Steel stress[2] [MPa]	Maximum bar spacing [mm]		
	w_k=0,4 mm	w_k=0,3 mm	w_k=0,2 mm
160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50
320	150	100	-
360	100	50	-

As stated about the creep and environmental condition in SFS EN 1992-1-1. Eurocode 2, 31, Figure 32 reflects the same matter here.

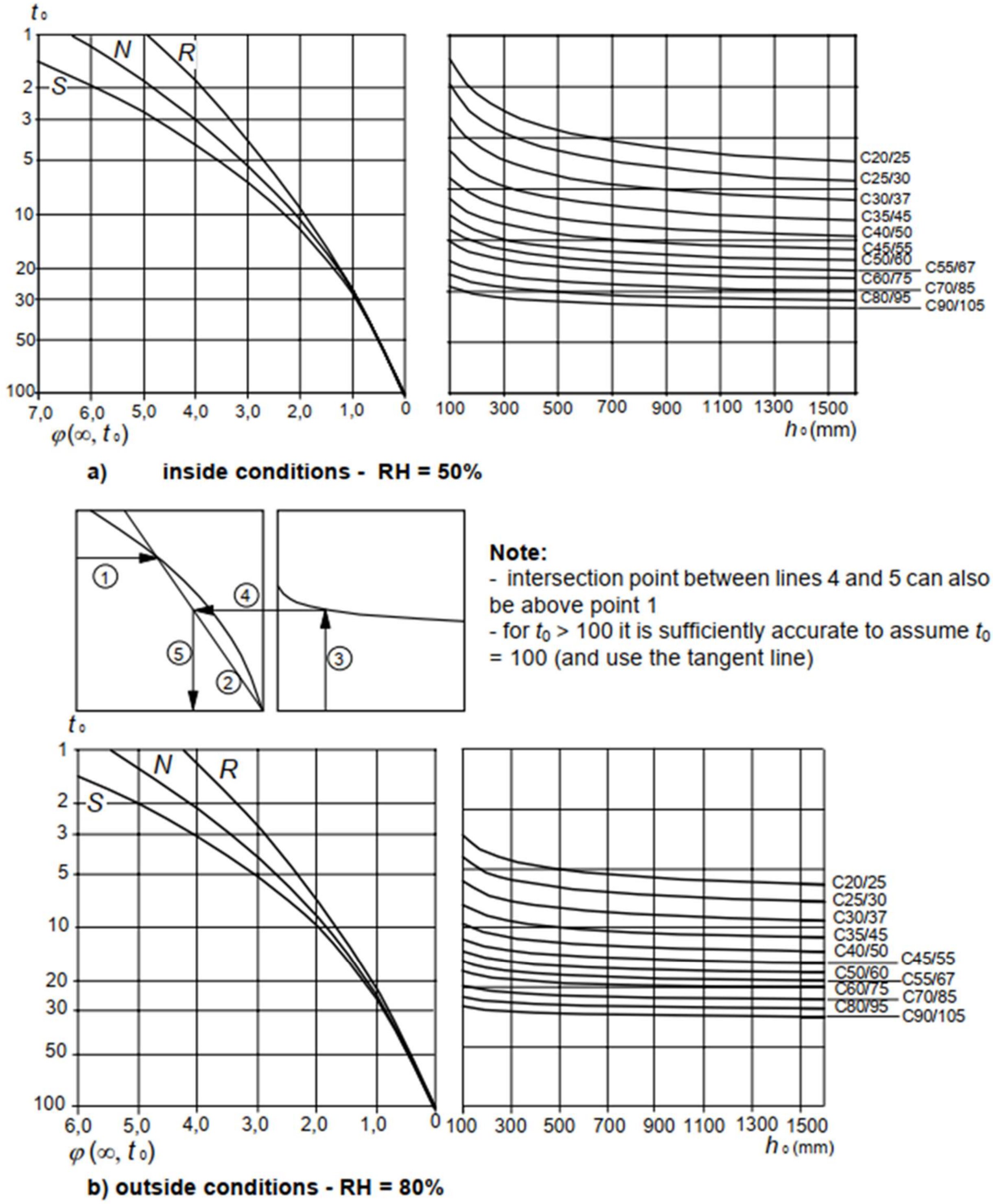


Figure 31. Method for determining the creep coefficient $\varphi(\infty, t_0)$ for concrete under normal environmental conditions

Where,

h_0	The notional size of the cross section in mm
t_0	The age of the concrete at time of loading in days

5.6.2 **Ultimate limit state**

Design resistance to bending and axial force In the case of walls, subject to the provision of adequate construction details and curing the imposed deformations due to the temperature or shrinkage may be ignored. The stress-strain relations for plain concrete should be taken into account. (SFS EN 1992-1-1. Eurocode 2, 194). Table 32 shows the characteristic of plain walls and the axial resistance of a rectangular cross-section with a uniaxial eccentricity in the direction of h_w may be taken as:

$$N_{Rd} = \eta f_{cd} \times b \times h_w \times (1 - 2e/h_w)$$

Where,

ηf_{cd}	The design effective compressive strength
B	The overall width of the cross-section
h_w	The overall depth of the cross-section
E	The eccentricity of N_{Ed} in the direction of h_w

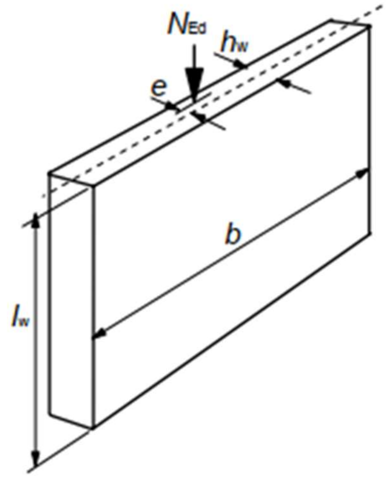


Figure 32. Notation for plain walls

5.6.3 **Slenderness of wall for buckling**

According to SFS EN 1992-1-1. Eurocode 2, 195, the slenderness of a column or wall is given by

$$\lambda = {l_0}/{i}$$

Where,

i	The minimum radius of gyration
l_0	The effective length of the member which can be assumed to be

$$l_0 = \beta . l_w$$

Where,

l_w	The clear height of the member
β	Coefficient which depends on the support conditions: For normal columns β=1 should in general be assumed. For cantilever columns or walls β=2

The slenderness of a wall in plain concrete cast in situ should generally not exceed

$$\lambda = 86 \; i.e. \frac{l_0}{h_w} = 25$$

Figure 33 states the effective lengths during buckling and Table 31 shows different values of in various conditions respectively. (SFS EN 1992-1-1. Eurocode 2, 66 & 195).

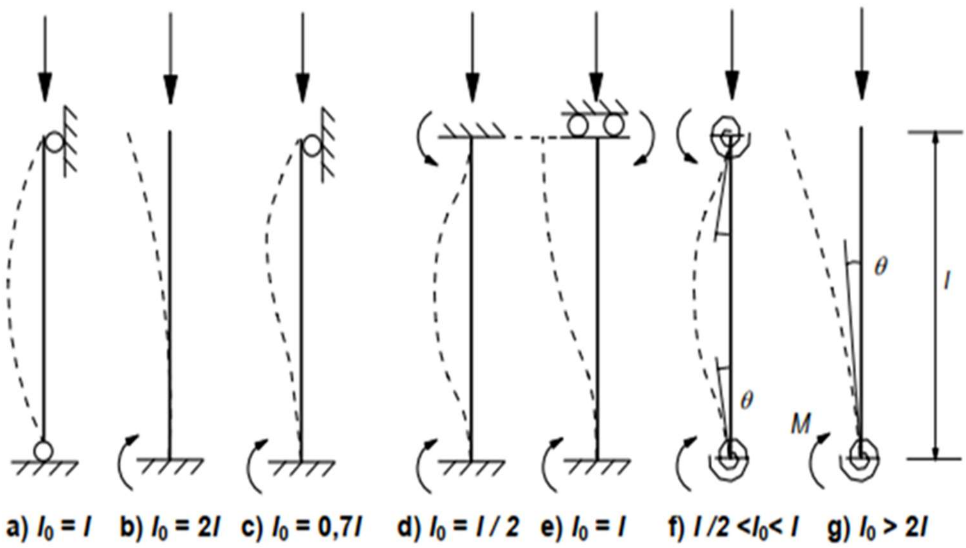
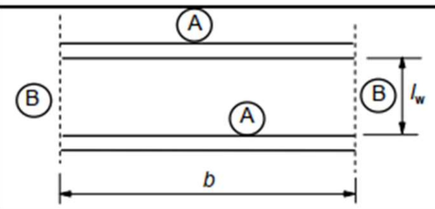
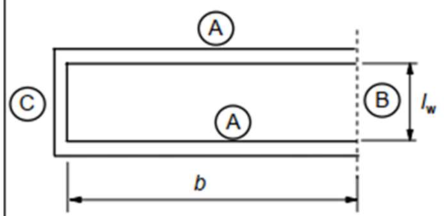
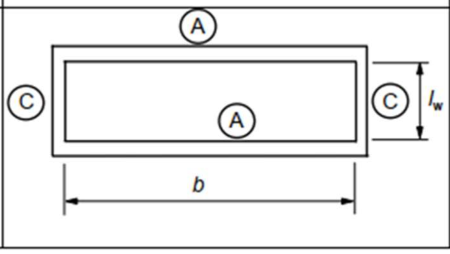


Figure 33. Buckling modes and effective lengths for isolated members

Table 31. Values of β for different edge conditions

Lateral restraint	Sketch	Expression	Factor β	
along two edges			β = 1,0 for any ratio of l_w/b	
Along three edges		$\beta = \frac{1}{1+\left(\frac{l_w}{3b}\right)^2}$	b/l_w	β
			0,2	0,26
			0,4	0,59
			0,6	0,76
			0,8	0,85
			1,0	0,90
			1,5	0,95
			2,0	0,97
			5,0	1,00
Along four edges		If $b \geq l_w$ $\beta = \frac{1}{1+\left(\frac{l_w}{b}\right)^2}$ If $b < l_w$ $\beta = \frac{b}{2l_w}$	b/l_w	β
			0,2	0,10
			0,4	0,20
			0,6	0,30
			0,8	0,40
			1,0	0,50
			1,5	0,69
			2,0	0,80
			5,0	0,96

A - Floor slab B - Free edge C - Transverse wall

5.6.4 **Simplified design method for walls and columns**

According to SFS EN 1992-1-1. Eurocode 2, 197, in absence of a more rigorous approach, the design resistance in terms of axial force for a slender wall or column in plain concrete may be calculated as follows:

$$N_{Rd} = \eta f_{cd} \times b \times h_w \times f_{cd} \times \emptyset$$

Where,

N_{Rd}	The axial resistance
b	The overall width of the cross-section
h_w	The overall depth of the cross-section
$\emptyset$	Factor taking into account eccentricity, including second order effects and normal effects of creep

For braced members, the factor Ø may be taken as:

$$\emptyset = (1.14 \times (1 - 2e_{tot}/h_w) - 0.02 \times l_c/h_w) \leq (1 - 2e_{tot}/h_w)$$

Again,

$$e_{tot} = e_0 + e_i$$

Where,

e_0	The first order eccentricity including, where relevant, the effects of floors (e.g. possible clamping moments transmitted to the wall from a slab) and horizontal actions
e_i	The additional eccentricity covering the effects of geometrical imperfections

5.7 **Foundation engineering**

According to lecture notes by Mustonen, 2016, bulk density of soil in general is 2.65g/cm^3 = 26 KN/m^3. All the equations regarding foundation engineering mentioned in this section are also extracted from his lectures.

Bearing capacity of soil:

$$\delta_{allowed} = \gamma_1 \times N_D \times D + {}^1\!/_2 \times \gamma_2 \times N_B \times B$$

$$\gamma_2 = \gamma_1 - 10$$

Where,

The smaller depth of the foundation is always counted to determine D.

γ_1	Bulk weight volume of soil above the footing
γ_2	Bulk weight volume of soil under the footing
D	Depth of footing from the zero plane.
B	Width of footing.
N_B/N_D	Factor of safety for friction

$$\phi_D = \tan^{-1}\left(\frac{\tan \phi_k}{F.O.S}\right)$$

F.O.S. = Factor of safety

$$N_D = \left[\tan\left(45^o + \frac{\phi_D}{2}\right)\right]^2 \times e^{\pi \tan \phi_d}$$
$$N_B = 2(N_D - 1) \times \tan \phi_D$$

Soil pressure for friction type of soil:

At rest, $P_o = \bar{\delta}(1 - \sin \phi_D)$

Active, $P_a = \bar{\delta}\left[\tan\left(45^o - \frac{\phi_D}{2}\right)\right]^2 \because C = 0$

Passive, $P_p = \bar{\delta}\left[\tan\left(45^o + \frac{\phi_D}{2}\right)\right]^2 \because C = 0$

Soil pressure for cohesion type of solid:

At rest, $P_0 = \bar{\delta} \because \phi_k = 0$

Active, $P_a = \bar{\delta} - 2C \because \phi = 0$

Passive, $P_p = \bar{\delta} + 2C \because \phi = 0$

Here,

ϕ_k Characteristic friction angle

C Cohesion

Table 32 gives information regarding friction angle (SFS EN 1997-2. Eurocode 7, 129) and Table 33 shows load bearing capacity of soil (BNBC 2006, 10667).

Table 32. Effective angle of shearing resistance

Soil type	**Grading**	**Range of I_D** %		**Effective angle of shearing resistance (φ')** °
Slightly fine-grained sand, Sand, sand-gravel	Poorly graded, ($C_U < 6$)	15–35	(loose)	30
		35–65	(medium dense)	32,5
		>65	(dense)	35
Sand, sand-gravel, gravel	Well-graded, ($6 \leq C_U \leq 15$)	15–35	(loose)	30
		35–65	(medium dense)	34
		>65	(dense)	38

Table 33. Presumptive values of bearing capacity for lightly loaded structures

Type of material	**Safe bearing capacity, kPa**
Soft rock or shale	440
Gravel, sandy gravel, silty sandy gravel, very dense and offer high resistance to penetration during excavation (soil shall include into groups GW, GP, GM, GC)	400
Sand (other than fine sand), gravelly sand, silty sand; dry (soil shall include the groups SW, SP, SM, SC)	200
Fine sand; loose and dry (soil shall include the groups SW, SP)	100

Slit, clayey slit, clayey sand, dry lumps which can be easily crushed by finger (soil shall include the groups ML, MI, SC, MH)	150
Clay, sandy clay; can be indented with strong thumb pressure (soil shall include the groups CL, CI,CH)	150
Soft clay; can be indented with modest thumb pressure (soil shall include groups CL, CI, CH)	100
Very soft clay; can be penetrated several centimeters with thumb pressure (soil shall include the groups CL, CI, CH)	50
Organic clay and peat (soil shall include the groups OI, OH, OL, Pt)	To be determined after investigation.
Fills	To be determined after investigation.

5.8 Thermal transmittance and building behaviour

The instructions given in Annex C4. NBCF 2003, 5-7, provide a method of calculating thermal transmittance (U) for building components and structures. Test results may also be utilized when the calculation is unreasonably difficult or the input information necessary for calculation is determined experimentally. An individual measuring result of thermal transmittance is valid only for the tested structure under measuring conditions. When the calculation of thermal transmittance is unreasonably difficult, it is, however, possible to estimate the thermal transmittance for a structure applicable to practical design on the basis of test results.

5.8.1 Homogeneous structure

The aim must be to take into account any inaccuracies in measuring, any practical variations of the characteristics of the structure and the materials e.g. homogeneous used in it, the effects of moisture content of the materials in accordance with the construction design and any possible irreversible changes to thermal conductivity of building materials during the service life. Over all resistance R and U value

$$R=R_1+R_2+\ldots..R_{si}+R_{se}$$

Where,

$$R_1 = \frac{d_1}{\lambda_1},\ R_2 = \frac{d_2}{\lambda_2},\ldots\ldots \begin{cases} d_1, d_2, \ldots\ldots\ldots (m) \\ \lambda_1, \lambda_2, \ldots (W/(m.^{o}C) \end{cases}$$

$$U=\frac{1}{R}$$

$[R] = (m^{2\,0}C/W)$

λ Thermal conductivity

d Thickness of a layer

R_{si}, R_{se} Internal and external surface resistances $(m^{2\,0}C/W)$

U Thermal transmittance = $(W/(m.^{o}C))$

5.8.2 Non-homogeneous structure

If a part of building envelope includes both homogeneous and non-homogenous layers next to each we can calculate the upper limit (RU) and lower limit (RL) for the over-all thermal resistance. The average value of those limits is then given as the over-all thermal resistance (R).

$$R = \frac{R_U + R_L}{2}\ (m^{2\,0}C/W)$$

More precise value for R can be achieved with following formula:

$$R = \frac{R_U + 2R_L}{3}\ (m^{2\,0}C/W)$$

$$Upper\ limit, R_U = \frac{A}{\frac{A_a}{R_a} + \frac{A_b}{R_b} + \cdots + \frac{A_n}{R_n}}\ \ (m^{2\,0}C/W)$$

A	Area of the whole structural part (wall, ceiling..etc.) (m^2)
$A_a, A_{b,\ldots,}A_n$	Areas of different parts a,b,…n, assembly (m^2)
$R_a, R_{b,\ldots,}R_n$	Over-all thermal resistances of parts a, b,…,n $(m^{2\,0}C/W)$

Lower limit,

$$R_L = R_1+R_2+\ldots..+R_i+\ldots\ldots+ R_{si}+R_{se}$$

Where,

$R_1, R_2, \ldots\ R_i$ are thermal resistanced of layers in 1,2,…,i

Lower limit is calculated basically with the logic for homogeneous and non-homogeneous layers. All non-homogeneous layers must be calculated as

$$R_i = \frac{A}{\frac{A_a}{R_{ia}} + \frac{A_b}{R_{ib}} + \cdots + \frac{A_n}{R_{in}}}$$

Where,

$R_{ia} = \frac{d_i}{\lambda_a},\ \ R_{ib} = \frac{d_i}{\lambda_b}$ etc ., concerning layer i

The above described method gives accurate enough values for over-all thermal resistance mainly for timber-framed buildings. Thermal conductivity of wooden material or brick or even concrete is not nearly as dramatically great as metallic materials. Addition to the U-value of a structural part, caused by the thermal bridges, can be calculated with the formula:

$$\Delta U_P = \frac{n}{A}\Delta G_P \quad and$$

$$\Delta U_L = \frac{L}{A}\Delta G_L$$

Where,

n	The amount of similar local thermal bridges in a structural part
A	The area of the structural part (m2)
L	Length (m) of similar linear thermal bridges in a structural part

Table 33 shows thermal transmittance of different materials (Annex C4. NBCF 2003, 3), Table 35 gives the value of moisture resistant of building materials (Korkeamäki, 2017) and Table 36 provides the data for moisture generation in normal air pressure (Korkeamäki, 2017).

Table 34. Thermal transmittance of different building materials

t	Building permit pending in year								
	–1969	1969–	1976–	1978–	1985–	10/2003–	2008–	2010–	2012–
Heated spaces									
External wall	0.81	0.81	0.40	0.35	0.28	0.25	0.24	0.17	0.17
Ground-supported floor	0.47	0.47	0.40	0.40	0.36	0.25	0.24	0.16	0.16
Floor with crawl space	0.47	0.47	0.40	0.40	0.40	0.20	0.20	0.17	0.17
Floor butting against out-door air	0.35	0.35	0.35	0.29	0.22	0.16	0.16	0.09	0.09
Roof	0.47	0.47	0.35	0.29	0.22	0.16	0.15	0.09	0.09
Door	2.2	2.2	1.4	1.4	1.4	1.4	1.4	1.0	1.0
Window	2.8	2.8	2.1	2.1	2.1	1.4	1.4	1.0	1.0

Table 35. Water vapour resistant value for different building materials

Materials with their thickness in mm	**Z_V (s/m) (Typical)**
Polyethylene 0.2	3500×10^3
Air gap 100	4×10^3
Concrete 100	150×10^3
Timber 100	400×10^3
Brick 100	32×10^3
Mineral or Rock wool 100	8×10^3
Polystyrene 100	100×10^3
polyurethane 100	1300×10^3
Gypsum or Plaster board 13	4×10^3
plywood 13	50×10^3
Paints	50×10^3

Table 36. Water vapour content and partial pressure index in normal air pressure 1.01 bar

t^0C	**V_k (g/m^3)**	**P_k (Pa)**	**t^0C**	**V_k(g/m^3)**	**P_k (Pa)**	**t^0C**	**V_k (g/m^3)**	**P_k (Pa)**
-20	0.87	102	14	12.1	1602	48	75.67	11207
-19	0.95	111	15	12.86	1708	49	79.33	11786
-18	1.04	122	16	13.65	1820	50	83.14	12390
-17	1.14	135	17	14.49	1939	51	87.1	13020
-16	1.25	149	18	15.37	2064	52	91.21	13677
-15	1.38	164	19	16.3	2197	53	95.48	14362
-14	1.52	181	20	17.28	2337	54	99.92	15075
-13	1.67	200	21	18.31	2484	55	104.5	15818
-12	1.83	221	22	19.4	2640	56	109.3	16592
-11	2.01	242	23	20.54	2805	57	114.2	17397
-10	2.2	266	24	21.74	2979	58	119.4	18234

-9	2.4	292	25	23	3162	59	124.7	19105
-8	2.61	319	26	24.32	3355	60	130.2	20010
-7	2.84	348	27	25.71	3559	61	135.9	20951
-6	3.08	379	28	27.17	3773	62	141.9	21928
-5	3.33	412	29	28.7	3999	63	143	22943
-4	3.6	447	30	30.31	4237	64	154.3	23997
-3	3.89	485	31	31.99	4487	65	160.9	25090
-2	4.19	524	32	33.75	4750	66	167.7	26224
-1	4.51	566	33	35.6	5027	67	174.7	27401
0	4.85	611	34	37.54	5317	68	181.9	28620
1	5.21	658	35	39.56	5622	69	189.4	29884
2	5.58	708	36	41.68	5940	70	197.1	31194
3	5.98	762	37	43.89	6278	71	205.1	32551
4	6.4	818	38	46.21	6631	72	213.3	33956
5	6.84	878	39	48.63	7000	73	221.8	35410
6	7.31	941	40	51.16	7388	74	230.6	36915
7	7.8	1008	41	53.79	7793	75	239.6	38471
8	8.32	1079	42	56.54	8218	76	248.9	40082
9	8.87	1154	43	59.41	8663	77	258.5	41747
10	9.45	1234	44	62.4	9127	78	268.4	43468
11	10.06	1318	45	65.52	9614	79	278.6	45247
12	10.71	1408	46	68.77	10122	80	289.1	47084
13	11.38	1502	47	72.15	10653			

5.9 **Energy Efficiency of the building**

Vänskä 2016 & Ympäristöministeriönohjeet-D5 2012 state energy consumption for structures, hot water, leakage air and ventilation

$$Q_{tot} = Q_s+Q_{hw}+Q_l+Q_v$$
$$Q_s = Q_{wa}+Q_r+Q_{wi}+Q_f+Q_d+Q_{tb}$$
$$Q_s = \sum UA\times Td /1000 \text{ (kWh)}$$

Td =Degree day = $\sum((T_{in}-T_{out})\times t)$

Where,

T_{in} Indoor temperature of the building

T_{out} Outdoor temperature of the building

Calculating T_d:

if T_{in} = 17 °C and T_{out} = −5°C

T_d = (17°C – (-)5°C×24h×365d= 192720 °Ch

Influence of floor against earth

$T_{earth,a} = T_{meanout,a} + 3..7°C$

Thermal bridge,

$$Q_{HB} = \sum l_{hb} \times \Psi_{hb} \times \sum((T_{in}-T_{out})\times t) /1000$$
$$= \sum l_{hb} \times \Psi_{hb} \times T_d$$

Where,

l_{hb}	Line length of thermal, bridge, m
Ψ_{hb}	Additional conductance, W/(mK)
T_d	Degree day

Heat consumption caused by air leakage,

$$Q_L = \rho \times c \times q_L \times T_d \text{ kWh}$$
$$q_L = q_{50}/(3600 \times x) \times A_{env}$$
$$q_{50} = n_{50} \times V/A_{env}$$

Where,

q_L	Leakage air flow rate (m^3/s)
T_d	Degree day
q_{50}	Leakage air number of building envelope, m^3/h (m^2)
x	Factor depending on floors e.g. 1 floor x=35,
A_{env}	Area of building envelope incl. Floor, m^2
n_{50}	Air leakage number of building, 1/h
V	Air volume of building, m^3

Energy consumption Ventilation,

$$Q_V = \rho \times c \times q_V \times t_d \times t_v \times T_d \times (1-\eta) \text{ kWh}$$

Where,

q_V	Ventilation flow (m^3/s)
ρ	Air density (1.2kg/m^3)
c	Specific heat capacity of air (1.0 kJ/kgK)
t_d	Daily run time ratio h/24h
t_v	Weekly run time ratio d/7d
T_d	Degree day (^oC24h)
η	Efficiency of heat recovery (e.g. 50 % = 0.5)

Energy consumption Domestic hot water,

$Q_{HW} = \rho_w \times c_w \times V_w(T_h - T_c)/3600 \text{ kWh}/ \eta - Q_{HWHR} + Q_{HWS} + Q_{HWcirc}$

Where,

ρ_w	Density of water (1000 kg/m^3)
c_w	Specific heat capacity, water (4.19 kJ/kgK)
V_w	Hot water consumption (m^3)
$T_h - T_c$	Water temperatures, hot, cold (K)
Q_{HWHR}	Heat recovery from waste water
$Q_{HWS} + Q_{HWcirc}$	Hot water storage and circulation losses

Electricity consumption of lighting,

$$W_{light} = \sum P \times A \times \Delta t \times f$$

W_{light}	Electricity consumption of lighting, kWh
P	Total power for lighting in a room / room, area, W/m²
A	Room area, m^2
Δt	Lights on time, h
f	Control mode of lighting (0,7..1,0)

Overall energy consumption of building is calculated from purchased energy by using coefficients of energy mode,

$E = (f_{DH}Q_{DH} + f_{Dcool}Q_{Dcooling} + \sum f_{fuel}Q_{fuels} + f_{electricity}W_{electricity})/A_{net}$

Where,

E	Purchased energy for building, kWh/m^2,a
Q_{DH}	District Heat energy consumption, kWh/a
$Q_{Dcooling}$	District cooling energy consumption, kWh/a
Q_{fuels}	Fuels energy consumption, kWh/a
$W_{electricity}$	Electricity consumption, deducted with self-produced electricity used in building, kWh/a
f_{DH}	Coefficient for district heat 0.7
f_{DC}	Coefficient for district cooling 0.4
f_{fuel}	Coefficient for fuels used 0.5 renewables, 1.0 fossil fuel
$f_{electricity}$	Coefficient for electricity 1.7
A_{net}	Heated net area of building, m^2

5.10 Fire safety requirement

Roof coverings shall be made so that a fire does not spread in the roof covering or its substrate in a hazardous manner. Roof coverings shall in general be of class $B_{ROOF}(t_2)$. Table 37 and Table 38 denote the limit of the building size regarding fire classes and maximum number of occupants of any building. (Annex E1. NBCF 2002, 10). Table 39 shows different class of load bearing building elements regarding fire classes (Annex E1. NBCF 2002, 14).

Table 37. Restrictions on the size of the buildings

Characteristic of the building	Fire class of the building		
	P1	P2	P3
NUMBER OF STOREYS			
– in general	no restriction	maximum 2	maximum 2
– residential building, office premises	no restriction	maximum 4	maximum 2
– production or storage premises, garages	no restriction	maximum 2	maximum 1
HEIGHT			
– in general	no restriction	maximum 9 m	maximum 9 m
– residential building, office premises	no restriction	maximum 14 m	maximum 9 m
– 1-storey production or storage premises	no restriction	no restriction	maximum 14 m
GROSS FLOOR AREA			
In general			
– 1-storey	no restriction	no restriction	max 2400 m^2
– 2-storey	no restriction	no restriction	max 1600 m^2
Gross floor area in production and storage premises and garages			
– 1-storey	no restriction	no restriction	no restriction
– 2-storey	no restriction	no restriction	*not permitted*

Table 38. Maximum number of occupants in a building

Use of the building	Number of storeys	Fire class of the building P1	P2	P3
Residential buildings		no restriction	no restriction	no restriction
Accommodation premises	1	no restriction	150 places	50 places
	2	no restriction	50 places	10 places
Institutions	1	no restriction	100 places	10 places
	2	no restriction	25 places	*not permitted*
Assembly and business premises	1	no restriction	no restriction	500 occupants
	2	no restriction	250 occupants	50 occupants
Office premises	1	no restriction	no restriction	no restriction
	2	no restriction	no restriction	150 employees
Production and storage premises	1	no restriction	no restriction	no restriction
	2	no restriction	50 employees	*not permitted*

Table 39. Class requirements for loadbearing constructions

	Fire class of the building P1 Fire load MJ/m² over 1200	600–1200	under 600	P2	P3
Column	**1**	**2**	**3**	**4**	**5**
Buildings with not more than 2 storeys, in general	R 120*	R 90*	R 60*	R 30	—
– if the insulation materials in the building are not at least of class A2–s1, d0	R 120	R 90	R 60	R 30	—
– institutions, accommodation premises, basements	R 120	R 90	R 60	R 30	—
Buildings with 3-8 storeys, in general	R 180	R 120	R 60	■	■
Residential or office buildings with 3-4 storeys					
– storeys	R 180	R 120	R 60	R 60*	■
– basement storeys	R 180	R 120	R 60	R 120	■
Buildings with more than 8 storeys	R 240	R 180	R 120	■	■
Basement storeys located below the uppermost underground storey	R 240	R 180	R 120	R 120	R 60
Requirements of the uppermost floor constructions, if the insulation materials of the uppermost floor are at least of class A2–s1, d0					
– not more than 2 storeys, no attic; constructions, which are the **primary** part of the load-bearing framework or bracing of the building	R 60	R 60	R 60	R 30	—
– not more than 2 storeys, no attic; constructions, which are a **secondary** part of the load-bearing framework or bracing of the building	R 15	R 15	R 15	R 15	—
– 1 storey, no attic, automatic fire-extinguishing system; constructions, which are a **secondary** part of the load-bearing framework or bracing of the building	—	—	—	—	—
– 1 storey, production or storage buildings; no attic; constructions, which are a **secondary** part of the load-bearing framework or bracing of the building	—	—	—	—	—
The roof constructions of attics or voids, which are not the primary load-bearing constructions of the frame of the building or constructions bracing the framework in case of fire	—	—	—	—	—

Notes to the Table: The fire resistance time requirement of balconies is half of that of the load-bearing constructions of the storey.
Derogations are permitted in production and storage buildings in accordance with the guidelines E2 of the National Building Code of Finland.

Symbols in the Table:
* = if the load-bearing constructions are not at least of class A2–s1, d0, the insulation materials of the building shall be made of materials at least of class A2–s1, d0.
(oval) = the load-bearing constructions shall be made of materials at least of class A2–s1, d0.
— = no class requirement
■ = not possible

Table 40 expresses different class requirements for fire separating building elements (Annex E1. NBCF 2002, 15).

Table 40. Class requirements for fire separating building elements

	Fire class of the building					
	P1			P2		P3
	Fire load MJ/m²			Number of storeys		
	over 1200	600–1200	under 600	3–4	1–2	
Column	1	2	3	4	5	6
Fire-separating building elements in storeys	EI 120	EI 90	EI 60	EI 60	EI 30	EI 30
– partitioning building elements (walls and doors of accommodation rooms)	EI 15	EI 15	EI 15	■	EI 15	EI 15
Fire-separating building elements in attics	EI 30	EI 30	EI 30	EI 30	EI 30	EI 30
– partitioning building elements	EI 15	EI 15	EI 15	EI 15	EI 15	EI 15
Fire-separating building elements in basements	EI 120	EI 90	EI 60	EI 120	EI 60	EI 30

Note to the Table: Class requirements for fire-separating building elements implementing fire-separation by area of production and storage buildings according to guidelines E2 of the National Building Code of Finland, those of garages according to guidelines E4 and the class requirements of fire-separating building elements of boiler rooms and fuel storages according to guidelines E9.

Symbol in the Table: ■ = not possible

Table 41. Class requirements for fire walls (Annex E1. NBCF 2002, 12)

	Fire class of the building				
	P1			P2	P3
	Fire load MJ/m²				
	Over 1200	600–1200	under 600		
Column	1	2	3	4	5
FIRE WALL	EI-M 240	EI-M 180	EI-M 120	EI-M 120	EI-M 60

Note to the Table: = building material of class A1 is required

In addition, all the minimum conditions mentioned in Annex G1 NBCF 2005 have fulfilled in order to design this building.

6 ARCHITECTURAL DESIGN

6.1 Primary data and preliminary overview

The building is going to locate in an urban area not close to the sea, at 109 m AMSL with the terrain category III for wind actions. It is a two storied residential building where the roof is not open to the dwellers. The total are of the plot is slightly over 333 m^2 out of which 224 m^2 that means about 67% of the plot will be built area. In the plan, the dimensions of the building are 16 m × 14 m. The roof top height of the building is assumed less than 10 m. In addition, the working life of the building is considered very typical as 50 years.

6.2 **Basic geometry of the building**

The structure comprises mainly timber together with both reinforced and non-reinforced concrete frame with the fire class P3. The wall and the roof are generally well thermally insulated. In addition to this, frost insulation is used under the ground floor slab and around the foundation. Non-reinforced precast load bearing 120 mm and 150 mm thick concrete walls are assumed to be used for frame and partition wall respectively. The center to center height of each floor is 3.2 m including 300 mm thick floor slab. Pre-fabricated hollow core slabs can be used in the intermediate floor. The roof slope is 150. In fine, wall footing is an ideal condition in this project for foundation design.

6.3 **Detailed architectural design**

6.3.1 **General information**

Total height of the building with 15^0 roof slopes is determined as 8.8 m from the plane. The top edge of the chimney is 700 mm high from the roof top. Metal sheet is used on the roof instead of roof tiles. On the other hand, timber cladding is used for the external wall surface. The building has a balcony, the main entrance and attached parking for a single car. Both the front door and parking gate are south faced. It has also a cellar in which there is a sauna that will be shown later in the section view. However, the overall dimension of the building regarding height, width and length is 8.8 m × 14 m × 16 m. Now, the elevations of the building from all sides are shown in Figures 34, 35, 36 and 37.

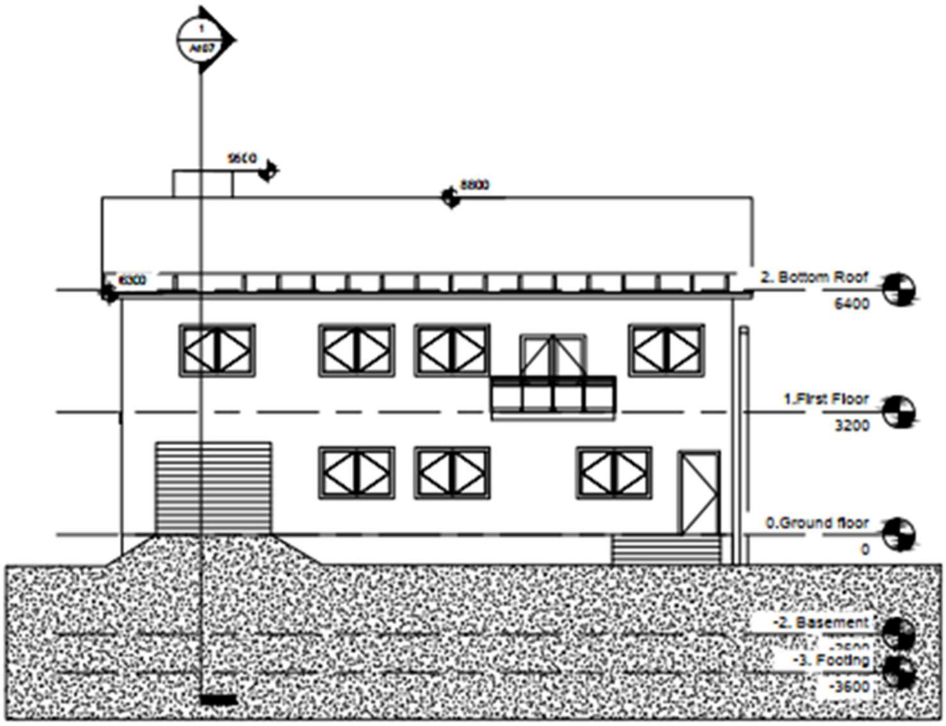


Figure 34. North elevation of the building

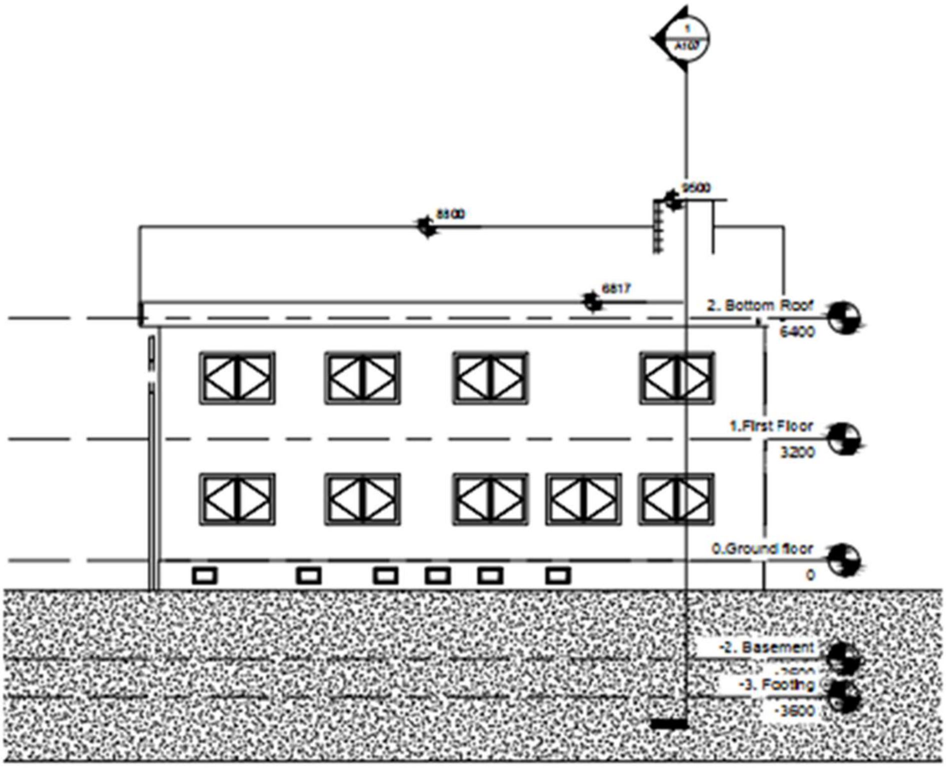


Figure 35. South elevation of the building

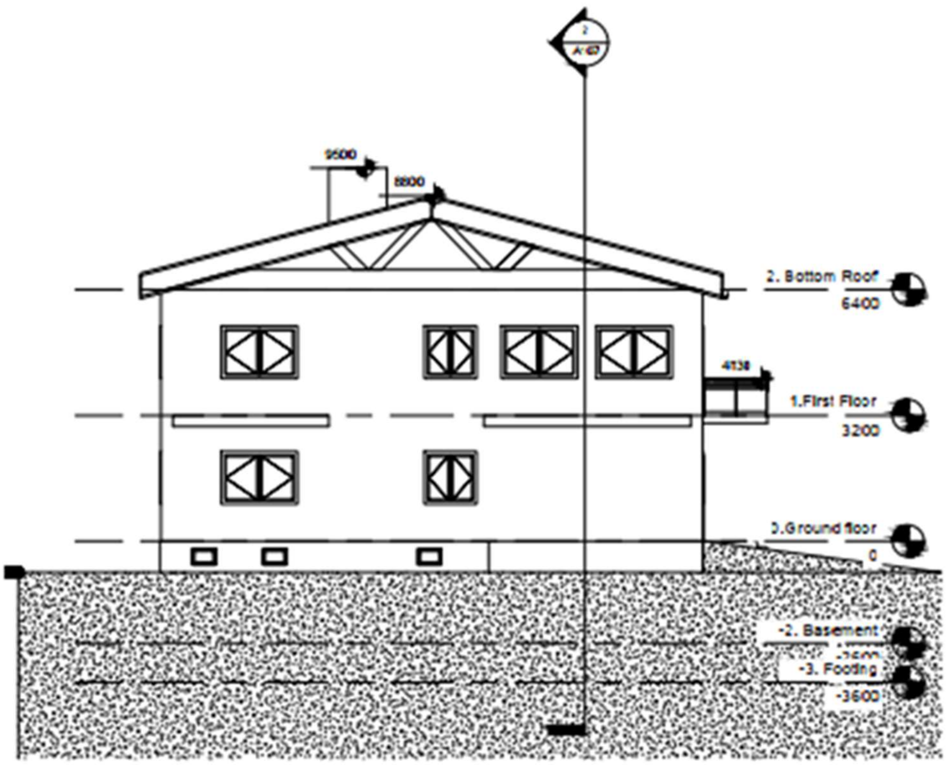


Figure 36. East elevation of the building

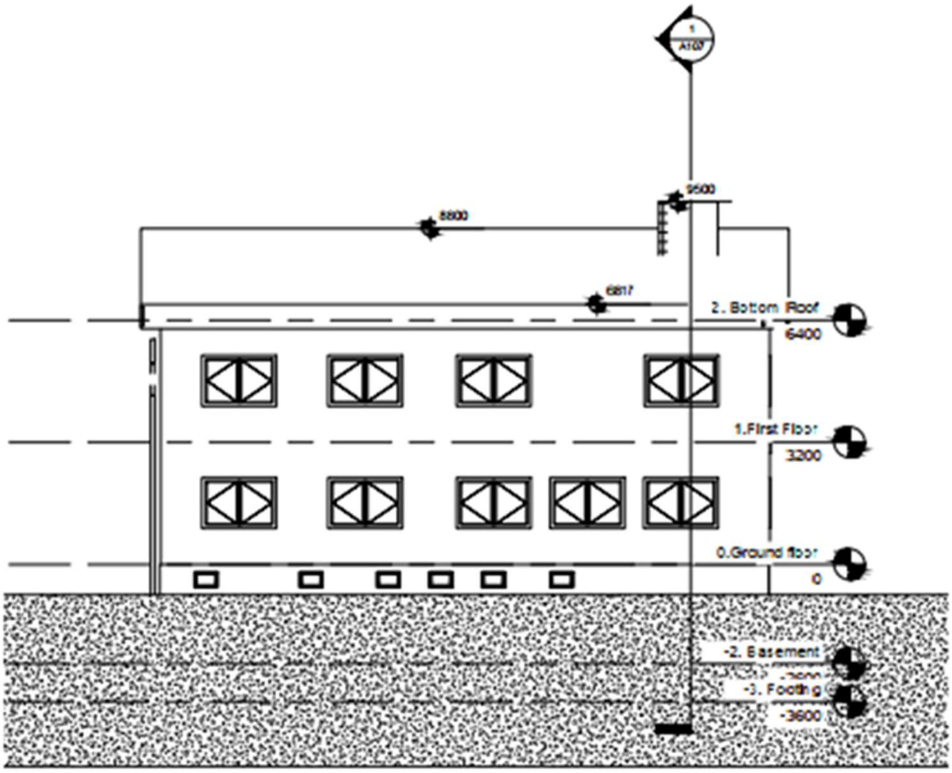


Figure 37. West elevation of the building

6.3.2 **Ground floor**

There is a living room, bedroom, kitchen, exercise room, bathroom and garage on the ground floor. There is no direct provision of entering from garage to the inside of the building other than the main door located at the opposite from the garage. A very small cloak room is also designed next to the stair case area. Dwellers would first step into this room from the outside. The bathroom is designed in the middle just by the garage because it ensures an easy access from all the rooms in this floor. The kitchen is situated in the corner so that no spicy smell or noise could directly come to the drawing place. However, the detailed dimension of the ground floor can be found in Table 42 and Figure 38 which contains a drawing of the ground floor.

Table 42. Architectural details of the ground floor

Name	Amount	Area (m^2)
Living & Dining	1	55
Bed room	1	30
Kitchen	1	34
Washroom	1	12
Exercise room	1	19
Garage	1	22
Staircase	1	6
Closet (Clock room)	1	8.5

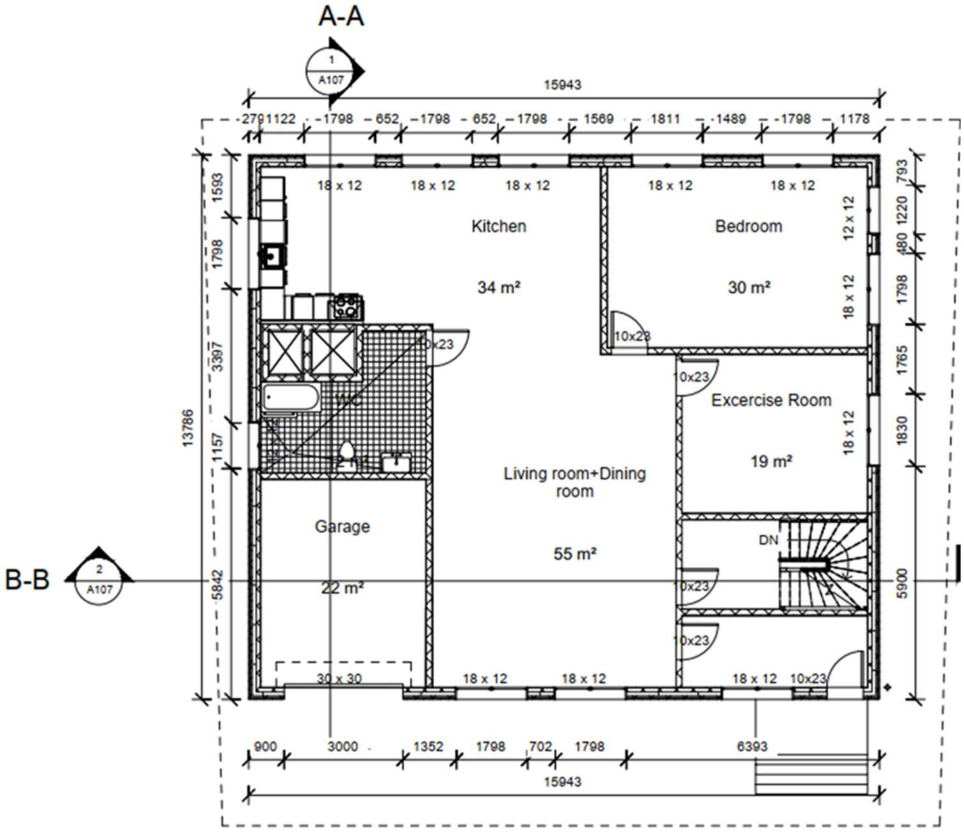


Figure 38. Ground floor of the building

6.3.3 **First floor**

The first floor is designed for a studio, kids' room, toilet, balcony, open space and two bedrooms. After walking on the staircase, a dweller will arrive in the open space at first. This place is designed for adequate natural ventilation and lighting. Moreover, it is also the gateway to the balcony. The two bedrooms have a very easy access at the floor since those are located next to the open space. Besides, it is necessary to have a quite environment in order to work and thus, the studio is designed at the corner next to the wash room. On the other hand, kids' room is placed at the middle and in between the studio and a bedroom because, the place gives the same access of surveillance to the kids from the studio, bedroom or even from the balcony. Moreover, the children can easily run back and forth from their room to the open space when they play. Again, the bathroom is designed in the middle because it ensures the same accessibility from all the rooms in this floor. The detailed dimensions of first floor can be found in Table 43 and Figure 39 which contains a drawing of the 1st floor.

Table 43. Architectural details of the ground floor

Name	Amount	Area (m^2)
Bed room	2	30 & 37
Studio	1	17

Kid's room	1	13
Washroom	1	12
Open space	1	84
Staircase	1	6

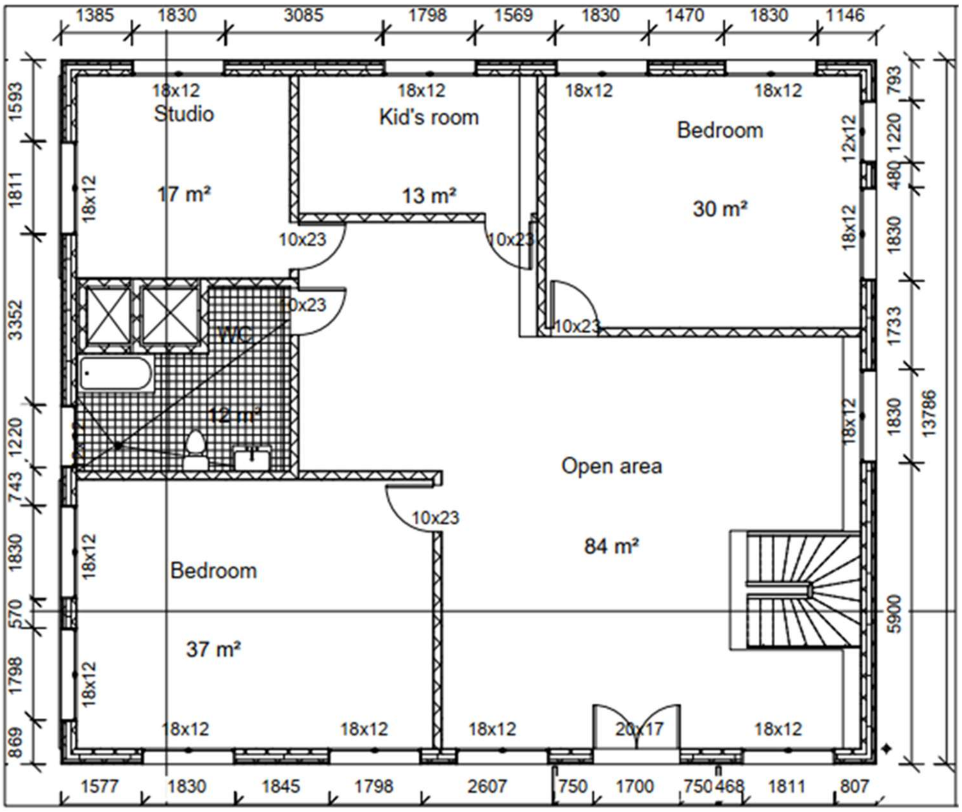


Figure 39. First floor of the building

The building has a balcony on the first floor and a sauna in the basement. The Sauna could be either electric or wood pellet. A bathroom is also located near to the sauna. Despite, there is no attached toiled facility throughout the whole building. All toilets and showers are open to each floor. Moreover, no additional terrace has been designed in this case. The details of the accommodation facilities of the whole building is shown in Table 44 and Figures 40 and 41 which contains the sections of the whole building from two different sides.

Table 44. Architectural details of the whole building

Name	Amount	Area (m^2)
Living & Dining	1	55
Kitchen	1	34
Bed room	3	97
Studio	1	17
Kid's room	1	13
Washroom	3	36
Open space	1	84
Sauna	1	21

Staircase	1	6
Balcony	1	5.5
Front door	1	2.6
Garage gate	1	7.2
Window	28	70.5

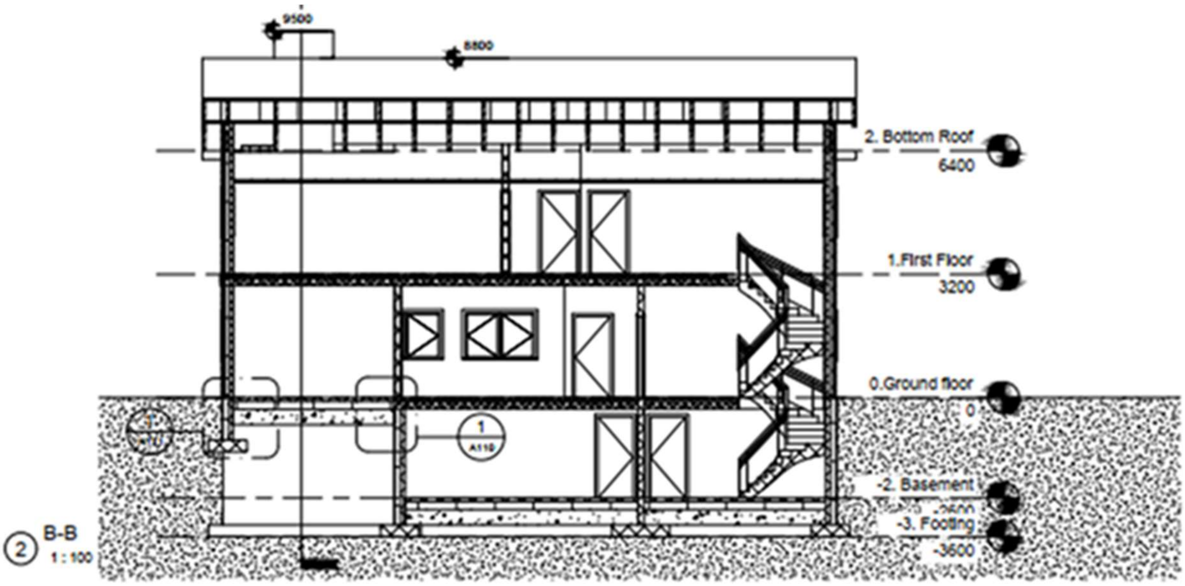


Figure 40. North section of the building

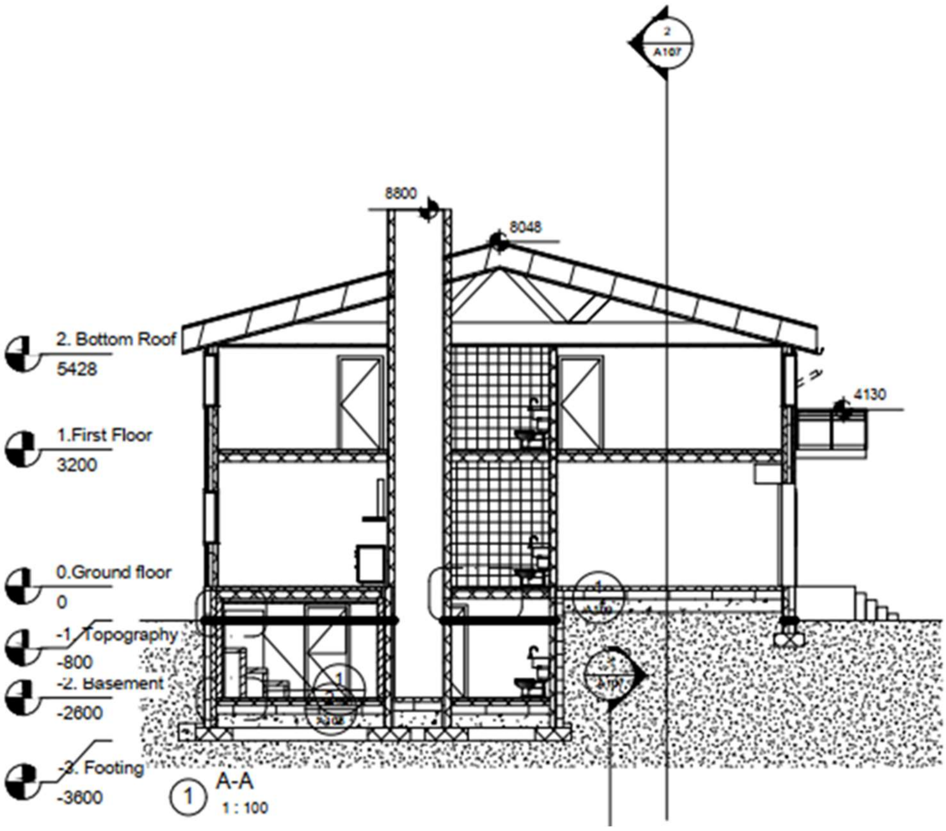


Figure 41. East section of the building

7 STRUCTURAL DESIGN AND ANALYSIS

Based on the architectural drawing and soil test, at first, it is needed to calculate the load acting on the structures and then start to design the structural elements chronologically as follows for building stability: the roof, floor slab and beam, load bearing wall or column and finally the foundation. In addition to this, building engineering, for example, Thermal transmittance, Fire safety, HVAC, Energy efficiency, Electric and Water engineering have to be taken into account later.

7.1 Design basis

The most important characteristics of any structural member are its actual strength. Actual strength must be large enough to resist, with some margin to spare, all foreseeable loads that may act on it during the life of the structure, without failing or other distress. It is logical, therefore, to proportion members to select concrete dimensions and reinforcement, so that member strengths are adequate to resist forces resulting from certain hypothetical overload stages, significantly above loads expected actually to occur in service. This design concept is known as strength design. Figure 42 below shows the chronological design process corresponding to the structure (Darwin, Dolan & Nilson 2016).

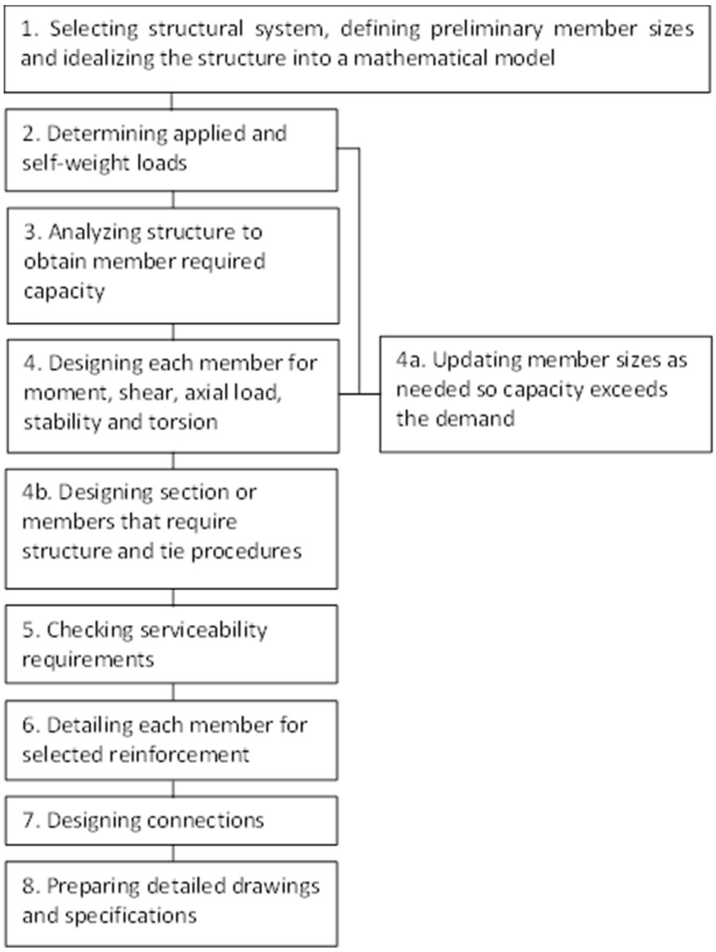


Figure 42. Structural design Sequence

7.2 Load combination

Load combination consists of different permanent and variable loads together in limit states. A variable or imposed load varies in magnitude from live to accidental load. It includes snow, wind, and occupancy and so on. On the other hand, permanent load refers to the self-weight of the structures that could be easily from the volume and density of any structural members. The load combination can be expressed as:

$$\sum \gamma_{G,j} G_{k,j} + \gamma_{Q,j} Q_{k,j}$$

Where,

G_k	Characteristic permanent action
G_d	Design value of a permanent action
Q_k	Characteristic variable action
Q_d	Design value of a variable action
γ_G	Partial factor for permanent actions
γ_Q	Partial factor for variable actions

7.2.1 Snow load

In Finland, snow load is one of the important environmental considerations which is being considered as live load. The expression is found as:

$$S = \mu_i \times C_e \times C_t \times s_k$$

Where,

S	Snow load
μ_i	Shape factor
C_e	Wind shield factor
C_t	Temperature factor
s_k	Snow load on ground

Hence,
We have angle of the pitch of roof, $\alpha = 15^0$
Therefore, $\mu_i = 0.8$ (From Table 5)

So, from the equation,
$S = 0.8 \times 1.0 \times 1.0 \times 2.5 kN/m^2$ (From Tables 4, 5 and Figure 15)
= **$2.0 kN/m^2$**

7.2.2 Wind load

As mentioned in earlier chapter, terrain category III is applicable for this project. In addition, external pressure coefficient is not used here to calculate wind load through the roof zone method since the variation is very negligible. Therefore, simple method is being used for wind load calculation as follows:

Here,

A_{ref}	Reference area
b	Width of the structure (the length of the surface parallel to the wind direction if not otherwise specified)
c_d	Dynamic factor
c_{dir}	Direction factor
c_f	Force coefficient
$c_{f,o}$	Force coefficient of structural elements without free-end flow
$c_o(z)$	Orography factor reference to z
$c_{p,net}$	Net pressure coefficient
c_{pe}	External pressure coefficient
$c_{pe,1}$	Local coefficient of the structure
$c_{pe,10}$	Overall coefficient of the structure
c_{pi}	Internal pressure coefficient
$c_r(z)$	Roughness factor reference to z
c_s	Size factor
c_sc_d	Structural factor
c_{season}	Seasonal factor
d	Depth of the structure (the length of the surface parallel to the wind direction if not otherwise specified)
F_w	Resultant wind force
$I_v(z)$	Turbulance intensity at z height above the ground
K_1	Equivalent factor
K_1	Mode shape factor; shape parameter
k_r	Terrain factor depending on the roughness length z_0
l	Length of the horizontal structure
$q_p(z)$	Peak velocity pressure at z height above the ground
v_b	Basic wind velocity
$v_{b,0}$	Fundamental value of the basic wind velocity
v_m	Mean wind velocity
$v_m(z)$	Mean wind velocity at height z
z	Height above the ground
z_0	Roughness length
$z_{0,II}$	Terrain category II (From Table 7)
z_e, z_i	Reference height for external wind action, internal pressure
z_{max}	Maximum height to be taken as 200 m
z_{min}	Minimum height above the ground
λ	Slenderness ratio
Ψ_r	Reduction factor of force coefficient for square sections with rounded corners
Ψ_λ	Reduction factor of force coefficient for structural elements with end-effects
ρ	Air density

We know that,

$$q_p(z) = [1 + 7 \times I_v(z)] \times 0.5 \times \rho \times V_m^2$$

$$I_v(z) = \frac{k_1}{c_o(z) \times ln(z|z_0)}$$

$= \frac{1}{1 \times ln(8.8/0.3)}$ (From Table 7. and Figure 16)

$= 0.3$

$V_m(z) = c_r(z) \times c_o(z) \times V_b$

$V_b = C_{dir} \times C_{season} \times .V_{b,0}$

$C_{dir} = 1$
$C_{season} = 1$
$V_{b,0} = 21m/s$ (From 5.3.1 in Chapter-5)

So,

$V_b = 1 \times 1 \times 21m/s$
$= 21m/s$

$c_r(z) = k_r \times \ln(z/z_0)$

$K_r = 0.19 \times (z_0/z_{0.II})^{0.007}$
$= 0.19 \times (0.3/0.05)^{0.007}$
$= 0.2$

$c_r(z) = 0.2 \times \ln(8.8/0.3)$
$= 0.7$

$V_m(z) = 0.7 \times 1 \times 21m/s$
$= 14.7m/s$

$q_p(z) = [1+7 \times 0.3] \times 0.5 \times 1.25kg/m^3 \times (14.7m/s)^2$
$= 0.42kN/m^2$

$d/b = 14m/16m = 0.9$

$c_{f0} = 2.2$ (From Figure 18)
$\lambda = 70$ (From Table 8)
$\Psi r = 1$ (From Figure 19)
$\Psi_\lambda = 0.6$ (From Figure 20)

$c_f = c_{f0} \times \Psi_r \times \Psi_\lambda$
$= 2.2x1x0.6$
$= 1.32$

$F_w = c_s c_d \times c_f \times q_p(z)$
$= 1 \times 1.32 \times 0.42kN/m^2$ (*From 5.3.2* in Chaper-5)
$= 0.52kN/m^2$
= **$0.5kN/m^2$**

7.3 Structural element design

Structural elements include roof, slab, beam, column, footing and so on. The calculation processes of these elements are shown below:

7.3.1 Roof truss: Acting force

Here,

γ_M	Partial factor for material properties
a	Nail spacing
b	Width of the member; For connection design represents height of the member
d	Fastener diameter
$E_{0.05}$	Fifth percentile value of modulus of elasticity
F	Force acting on the truss
$f_{c,0,d}$	Design compressive strength along the grain
$f_{t,0,d}$	Design tensile strength along the grain
$f_{c,0,k}$	Characteristic compressive strength along the grain
$f_{c,90,d}$	Design compressive strength perpendicular to the grain
$f_{h,i,k}$	Characteristic embedment strength of timber member i
f_{mk}	Characteristic bending strength
$f_{m,y,d}$	Corresponding design bending strength along y-axes
$f_{m,z,d}$	Corresponding design bending strength along z-axes
F_{Rd}	Design value of axial withdrawal capacity of the fastener
$F_{ax,Rk}$	Characteristic axial withdrawal capacity of the fastener
$F_{v,Rk}$	Characteristic load-carrying capacity per shear plane per fastener
$f_{t,0,k}$	Characteristic tensile strength along the grain
$f_{t,90,k}$	Characteristic tensile strength perpendicular to the grain
$f_{h,i,,k}$	Characteristic embedment strength of timber i
$f_{u,k}$	Characteristic tensile strength of bolts
$f_{v,k}$	Characteristic shear strength
$f_{v,d}$	design shear strength for the actual condition
h	Height of the member
H	Height of the truss
h_e	Embedment depth
i_y	Radius of Gyration at y-direction
i_z	Radius of Gyration at z-direction
$k_{c,90}$	Factor taking in to account the load configuration, possibility of splitting and degree of compressive deformation (the value is usually 1)
$k_{c,y}$	Instability factor
$K_{c,z}$	Instability factor
k_{def}	Deformation factor
k_m	Factor (0.7 for rectangular and 1 for other cross sections)
k_{mod}	Modification factor for duration of load and moisture content
k_y	Instability factor corresponding to bending about y-axis

k_z	Instability factor corresponding to bending about z-axis
$L_{cr,y}$	Buckling length of the member where force acted on y-direction
$L_{cr,z}$	Buckling length of the member where force acted on z-direction
$M_{y,Rk}$	Characteristic yield moment of fastener
N_{Ed}	Axial force
R	Resultant force
S_n	Axial force
t_i	Timber or board thickness or penetration depth
α	Angle between the force and x-direction
β	Angle between the force and x-direction in specific point; Ratio between the embedment strength of the members
$\lambda_{rel,y}$	Relative slenderness ratio corresponding to bending about y-axis
$\lambda_{rel,z}$	Relative slenderness ratio corresponding to bending about z-axis
λ_y	Slenderness ratio corresponding to bending about y-axis (Deflection in the z-direction)
λ_z	Slenderness ratio corresponding to bending about z-axis (Deflection in the y-direction)
ρ_k	Characteristic density
ρ_{mean}	Mean density
$\sigma_{c,0,d}$	Design compressive stress along the grain
$\sigma_{c,90,d}$	Design compressive stress in the contact area perpendicular to the grain
$\sigma_{t,0,d}$	Design tensile stress along the grain
$\sigma_{m,y,d}$	Corresponding design bending stress along y-axes
$\sigma_{m,z,d}$	Corresponding design bending stress along z-axes
τ_d	Design shear stress

Although snow and wind loads are counted for a short-term structure, whereas only self-load for permanent only (Table 14). Here, self-load for the roof truss is 1kN/m2 (0.5kN/m2 for roof sheathing material and the same value for timber truss itself) which is very little. Therefore, short term condition is applied without considering any partial safety factor in this case in order to calculate the force or load only that acts on the truss. The rest of the calculations follow the conditions for permanent class of load duration. Figure 43 shows the pattern of the timber roof truss designed for the building.

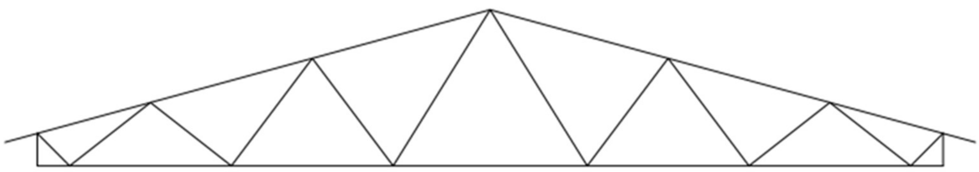


Figure 43. Roof truss designed for this building (Details of the drawing are mentioned in Appendix 1)

Conditions:
Truss spacing = 900mm
$\alpha = 15^0$
h = 2.4m

At ultimate limit state,
F1 = 0.9 x [0.5+0.5 x (0.5m+2.5m)] x (2.0kN/m^2 + 0.5kN/m^2)
= 3.1kN

F2 = 0.9x[0.875m+0.5×(1.25 m+0.5m×2.5m)]×(2.0kN/m^2+0.5kN/m^2)
= 4.8kN

F3 = 0.9 x [1.25 m x 0.5× (1.25m+0.5× 3m)] x (2.0kN/m^2 + 0.5kN/m^2)
= 5.9kN

F4 = 0.9 x [0.5 x (1.25m+1.5m) ×2] x (2.0kN/m^2 + 0.5kN/m^2)
= 6.2kN

$R_A = R_B$
= F1 + F2 + F3 + 0.5F4
= 3.1kN + 4.8kN + 5.9kN + 0.5×6.2kN
= 16.9kN

∑F = 2×16.9kN
= 33.8kN

Joint method analysis is used to calculate the acting forces on the truss members. Therefore, 7 joints were found (For details, Appendix 1) which were the prerequisites to continue the calculation process. Figurs 44-50 represent the joints.

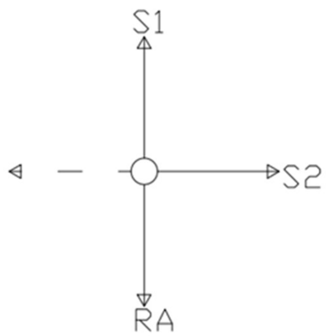


Figure 44. Joint-I

$S_2 = 0$ (Horizontal force)
$R_A + S_1 = 0$ (Vertical force)
$S_1 = -R_A$
$= -16.9$ kN (Compression)

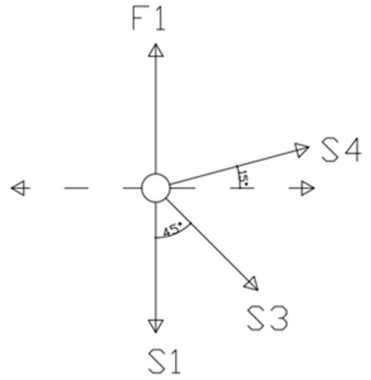


Figure 45. Joint-II

$\beta = 45^0$
$S_4 .\cos\beta + S_3.\cos\alpha = 0$ (Horizontal force)
$S_3 = -S_4.(\cos\alpha / \cos\beta)$
$= -S_4.(\cos15^0 / \cos315^0)$
$= -1.37 \times S_4$

$S_4.\sin\alpha - F_1 - S_1 + S_3.\sin\beta = 0$ (Vertical force)
$S_4. \sin\alpha = F_1 + S_1 - S_3.\sin\beta$
$S_4 = \frac{S_1+F_1}{-1.23}$
$= \frac{3.1-16.9}{1.23}$
$= -11.2$ kN (Compression)

$S_3 = -1.37 \times S_4$
$= -1.37 \times (-11.2)$
$= 15.3$ kN (Tension)

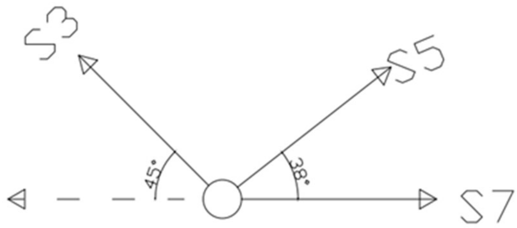


Figure 46. Joint-III

$S_5.\sin\beta + S_3.\sin\beta = 0$ (Vertical force)
$S_5.\sin38^0 + S_3.\sin135^0 = 0$
$S_5 = -\frac{S3.sin135^o}{\sin 38^0}$
$= -\frac{15.3 \times sin135^o}{\sin 38^0}$
$= -17.6$ kN (Compression)

$S_7 + S_5.\cos\beta + S_3.\cos\beta = 0$ (Horizontal force)
$S_7 + S_5.\cos75^0 + S_3.\cos135^0 = 0$
$S_7 = -[-17.6 \cos 38^0 + 15.3 cos135^0]$[Since, $S_5 = S_3$]
$= 24.7$ kN (Tension)

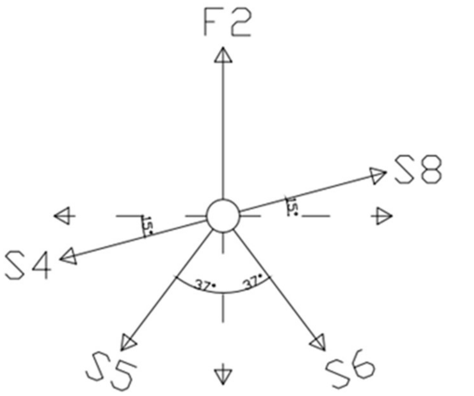


Figure 47. Joint-IV

$S_8.\sin15^0 - F_2 + S_4.\sin195^0 + S_5.\sin233^0 + S_6.\sin307^0 = 0$ (Vertical force)

$S_6 = \frac{F_2 - S_8 \sin 15^0 - S_4 \sin 195^0 - S_5 \sin 233^0}{\sin 307^0}$

$= [(4.8kN - S_8.\sin15^0 - (-)\ 11.2kN.\ \sin195^0 - (-)\ 11.2kN.\sin233^0]/\sin307^0$

$= 15.2kN + 0.3\times S_8$

$S_8.\cos15^0 + S_4\cos195^0 + S_5\cos233^0 + S_6\cos307^0 = 0$ (Horizontal force)

$\rightarrow S_8.\cos15^0 + (-11.2.\cos195^0) + (-17.6.\cos233^0) + (15.2+0.3.S_8).\cos307^0$

$\rightarrow S_8 + 10.82 + 10.6 + 9.2 + 0.2.S_8 = 0$

$\rightarrow 1.2S_8 = -30.6$

$\rightarrow S_8 = -25.5$ kN (Compression)

$S_6 = 15.2kN + 0.3(-25.5)$

$= 7.6$ kN (Tension)

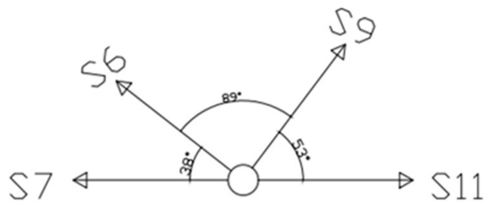


Figure 48. Joint-V

$S_9.\sin53^0 + S_6\sin142^0 = 0$ (Vertical force)

$\rightarrow 0.8S_9 + 7.6\sin105^0 = 0$

$\rightarrow S_9 = -4.598/0.8$

$= -5.9$ kN (Compression)

$S_{11} + S_9\cos75^0 + S_6.\cos105^0 - S_7 = 0$ (Horizontal force)

$\rightarrow S_{11} + (-5.9)\cos53^0 + 7.6.\cos142^0 - 24.7 = 0$

$\rightarrow S_{11} = 34.2$ kN (Tension)

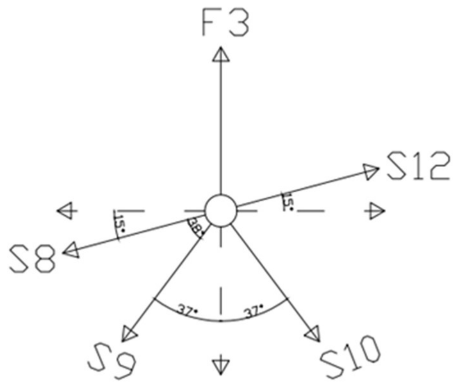


Figure 49. Joint-VI

$S_{12}.\sin15^0 - F_3 + S_8 .\sin195^0 + S_9.\sin233^0 + S_{10}.\text{Sin}307^0 = 0$ (Vertical force)
$\rightarrow 0.3S_{12} - 5.9 + (-)25.5\sin195^0 + (-)5.9\sin233^0 - 0.8S_{10} = 0$
$\rightarrow S_{10} = (0.3S_{12}+5.4)/0.8$
$\rightarrow S_{10} = 0.4S_{12}+6.8$

$S_{12}\cos15^0 + S_8\cos195^0 + S_9.\cos233^0 + S_{10}.\cos307^0 = 0$ (Horizontal force)
$\rightarrow S_{12} + (-)\ 25.5\cos195^0 + (-)5.9\cos233^0 + (0.4S_{12}+6.8)\cos307^0 = 0$
$\rightarrow S_{12} = -32.2/1.24$
$\rightarrow S_{12} = -26.0$ kN (Compression)

$S_{10} = 0.4\times(-)26.0 + 6.8$
$\rightarrow S_{10} = -3.6$ kN (Compression)

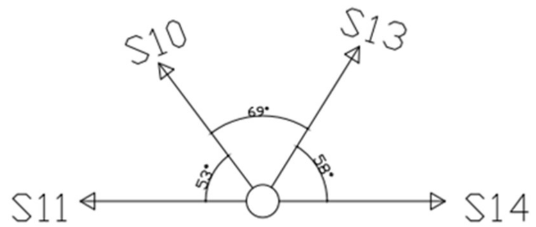


Figure 50. Joint-VII

$S_{13}.\sin58^0 + S_{10}\sin127^0 = 0$ (Vertical Force)
$\rightarrow S_{13} = -3.6\sin127^0 / \sin58^0$
$\rightarrow S_{13} = -3.4$ kN (Compression)

$S_{14} + S_{13}\cos58^0 + S_{10}.\cos127^0 - S_{11} = 0$ (Horizontal Force)
$\rightarrow S_{14} = -[(-)3.5\cos58^0 + (-)\cos127^0 - 34.2]$
$\rightarrow S_{14} = 33.8$ kN (Tension)

Findings:
For horizontal chord,
Highest $N_{Ed} = 16.1$kN at S_{11}
L= 2.5 m
$L_{cr} = 1\times2.5 = 2.5$m

For diagonal brace,
Highest N_{Ed} =17.6kN at S_5
L= 1.6m
L_{cr} = 1×1.6 = 1.6m

For vertical brace,
Highest N_{ED} = 16.9kN at S_1
L = 0.5m
L_{cr} = 1×0.5= 0.5m

7.3.2 **Roof truss: Profile selection**

Horizontal Chord:

b = 100mm , h = 150mm, L_1 = 2500mm
k_{mod} = 0.6, (From Table 19, Permanent duration, Service class 2)
γ_M = 1.4 (From Table 16)
Considering timber class C24 so,
f_{mk} = 24MPa, $f_{t.0.k}$ = 14MPa, $f_{t.90.k}$ = 0.5MPa
$f_{c.0.k}$ = 21MPa, $f_{c.90.k}$ = 2.5MPa, $f_{v.k}$ = 2.5MPa,
$E_{0.05}$ = 7.4GPa
N_{Ed} = 34.2kN

$\sigma_{c.0.d} = \frac{N_{Ed}}{b \text{ x } h} = 2.28 \times \frac{N}{mm^2}$

$L_{cr.y}$ = L_1 x 1.0
= 2.5 x 10^3 mm

$i_y = \sqrt{\frac{\frac{b.h^3}{12}}{b.h}}$ = 43.301 mm

$\lambda_y = \frac{(L_{cr.y})}{i_y}$ = 57.735

$\lambda_{rel.y} = \frac{\lambda_y}{\pi} . \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ = 0.979

k_y = 0.5 x [1 + 0.2 × ($\lambda_{rel.y}$ − 0.3) + $\lambda_{rel.y}{}^2$] = 1.047

$k_{c.y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel.y}^2}}$ =0.705

$f_{c.0.d} = \frac{f_{c,0,k} k_{mod}}{\gamma_M}$ = 9N/mm^2

$\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}}$ = 0.359 The beam can take the force in y-y axis

$L_{cr.z}$ = $L_{cr.y}$

$i_z = \sqrt{\frac{\frac{b.h^3}{12}}{b.h}} = 28.868$ mm

$\lambda_z = \frac{(L_{cr.z})}{i_z} = 86.603$

$\lambda_{rel.z} = \frac{\lambda_z}{\pi} . \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} = 1.469$

$k_z = 0.5 \times [1 + 0.2 \times (\lambda_{rel.z} - 0.3) + \lambda_{rel.z}^2] = 1.695$

$k_{c.z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel.z}^2}} = 0.393$

$f_{c.0.d} = \frac{f_{c,0,k} k_{mod}}{\gamma_M} = 9 N/mm^2$

$\frac{\sigma_{c.0.d}}{k_{c.z} \cdot f_{c.0.d}} = 0.303$ The beam can take the force in z-z axis

Diagonal brace:
b = 100mm, h = 100mm, L_1 = 1600mm
Timber class C24
N_{Ed} = 17.6kN

$\sigma_{c.0.d} = \frac{N_{Ed}}{b \times h} = 1.76 \times \frac{N}{mm^2}$

$L_{cr.y} = L_1 \times 1.0 = 1.6 \times 10^3$ mm

$i_y = \sqrt{\frac{\frac{b.h^3}{12}}{b.h}} = 28.868$ mm

$\lambda_y = \frac{(L_{cr.y})}{i_y} = 55.426$

$\lambda_{rel.y} = \frac{\lambda_y}{\pi} . \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} = 0.94$

$k_y = 0.5 \times [1 + 0.2 \times (\lambda_{rel.y} - 0.3) + \lambda_{rel.y}^2] = 1.006$

$k_{c.y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel.y}^2}} = 0.733$

$f_{c.0.d} = \frac{f_{c,0,k} k_{mod}}{\gamma_M} = 9 N/mm^2$

$\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} = 0.267$ The beam can take the force in y-y axis

$L_{cr.z} = L_{cr.y}$

$i_z = \sqrt{\frac{\frac{b.h^3}{12}}{b.h}}$ = 28.868 mm

$\lambda_z = \frac{(L_{cr.z})}{i_z}$ = 55.426

$\lambda_{rel.z} = \frac{\lambda_z}{\pi} . \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ = 0.94

$k_z = 0.5 \times [1 + 0.2 \times (\lambda_{rel.z} - 0.3) + \lambda_{rel.z}{}^2]$ = 1.006

$k_{c.z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel.z}^2}}$ =0.733

$f_{c.0.d} = \frac{f_{c,0,k} k_{mod}}{\gamma_M}$ = 9N/mm^2

$\frac{\sigma_{c.0.d}}{k_{c.z} \cdot f_{c.0.d}}$ = 0.267 The beam can take the force in z-z axis

Vertical brace:
b = 50mm, h = 100mm, L_1 = 500mm
Timber class C24
N_{Ed} = 16.9kN

$\sigma_{c.0.d} = \frac{N_{Ed}}{b \times h} = 3.38 \times \frac{N}{mm^2}$
$L_{cr.y} = L_1 \times 1.0$ = 500 mm

$i_y = \sqrt{\frac{\frac{b.h^3}{12}}{b.h}}$ = 28.868 mm

$\lambda_y = \frac{(L_{cr.y})}{i_y}$ = 17.321
$\lambda_{rel.y} = \frac{\lambda_y}{\pi} . \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ = 0.294

$k_y = 0.5 \times [1 + 0.2 \times (\lambda_{rel.y} - 0.3) + \lambda_{rel.y}{}^2]$ = 0.543

$k_{c.y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel.y}^2}}$ =1.001

$f_{c.0.d} = \frac{f_{c,0,k} k_{mod}}{\gamma_M}$ = 9N/mm^2

$\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}}$ = 0.375 The beam can take the force in y-y axis

$L_{cr.z} = L_{cr.y}$

$i_z = \sqrt{\frac{\frac{b.h^3}{12}}{b.h}}$ = 14.434 mm

$\lambda_z = \frac{(L_{cr.z})}{i_z}$ = 34.641

$\lambda_{rel.z} = \frac{\lambda_z}{\pi} . \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ = 0.587

$k_z = 0.5 \times [1 + 0.2 \times (\lambda_{rel.z} - 0.3) + \lambda_{rel.z}{}^2]$ = 0.701

$k_{c.z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel.z}{}^2}}$ = 0.922

$f_{c.0.d} = \frac{f_{c,0,k} k_{mod}}{\gamma_M}$ = 9N/mm²

$\frac{\sigma_{c.0.d}}{k_{c.z} \cdot f_{c.0.d}}$ = 0.407 The beam can take the force in z-z axis

7.3.3 Roof truss: Fastener connection

Main Joint:
Nails have been chosen to connect the roof truss members.

d = 2.5mm a = 50mm	k_{mod} = 0.6 (From Table 19)
h = 60mm b =150mm	γ_M = 1.4 (From Table 16)
f_u = 600N/m² ρ_{k_1} = 350kg/m³	k_{def} = 0.8 (From Table 20)
ρ_{mean} = 420kg/m³ ρ_{k_2} = 460kg/m³	
t_1 = 12mm, t_2 = a = 50 mm	

$M_{y.Rk} = 0.3 \times f_u \times d^{2.6}$
$= (0.3 \times 600 \times 2.5^{2.6})$ N.mm
$= 1.949 \times 10^3$ N.mm

Characteristic embedment strength without predrilled holes,

$f_{h,i,k1} = 0.082 \times 350 \times (2.5^{-0.3})$ N/mm² = 21.802 N/mm²

$f_{h,i,k2} = 0.082 \times 460 \times (2.5^{-0.3})$ N/mm² = 28.654 N/mm²

$\beta = \frac{f_{h,i,k2}}{f_{h,i,k1}}$ = 1.314

$f_{v.k.a} = f_{h,i,k1} \times t_1 \times d$ = 654.065N
$f_{v.k.b} = 0.5 f_{h,i,k2} \times t_2 \times d = 1.791 \times 10^3$ N

$f_{v.k.c} = 1.05 \times \frac{f_{h,i,k1} \text{ x } t_1 \text{ x d}}{2+\beta} \times \sqrt{2.\beta.(1+\beta) + \frac{4.\beta.(2+\beta).M_{y.Rk}}{f_{h,i,k1}.t_{1.d}^{2}}} - \beta$= 396.259N

$f_{v.k.d} = 1.15 \times \sqrt{\frac{2\beta}{1+\beta}} \times \sqrt{2.M_{y.Rk} \times f_{h,i,k1} \times \mathrm{d}}$ = 0.565kN

$F_{v.k}$ = min($f_{v.k.a}$, $f_{v.k.b}$, $f_{v.k.c}$,$f_{v.k.d}$) = 0.396 kN

$F_{v.d} = \frac{k_{mod} \text{ x } F_{v.k}}{\gamma_M}$ = 0.17 kN
cos33 = 0.84

$F_{v.d.1} = F_{v.k} \times 16 \times 2$ =12.68kN

F_{maxJ1} = 12.37kN × cos33 = 10.391kN

$\frac{F_{maxJ1}}{F_{v,d,1}}$ = 0.819 Ok

Spitting perpendicular to the grain: h_e = 60 mm, h =150mm

$F_{90.Rk} = 14.48 \times \left[\frac{60}{\left(1-\frac{60}{148}\right)}\right]^{0.5} = 6.75 \times 10^3$
$F_{90.Rk1}$ = 6750kN

$F_{90.Rd} = k_{mod} \times \frac{F_{90.Rk1}}{\gamma_M} = 2.893 \text{ x } 10^6$ N

$F_{90.Rd} > F_{maxJ1}$ OK

So, the Joint will not fail before the truss buckling

Minimum required spacings of nails:

a_1 = d × (5+ 5cos(57)) = 23.748 mm

a_2 = 5 × d = 12.5 mm

a_3.t = d × (10+ 5 cos(57)) = 36.248 mm

a_3.c = 10 × d = 25 mm

a_4.t = d × (5 + 2 sin(57)) = 14.681 mm

a_4.c = 5 × d = 12.5 mm

Designed spacings of nails:
a_1= 50mm, a_2= 20mm, a_{3t}= 40mm and a_{4t}= 30mm

Truss Corner Joint:

$F_{v.d.1} = F_{v.k} \times 16 \times 2 = 12.68$ kN

$F_{maxJ2} = 22.64$kN

$\frac{F_{maxJ2}}{F_{v,d,1}} = 1.785$ Not Ok

The Joint is Critical in here and might fail at a smaller maximum anticipated load. So, design needs to be reconsidered.

Splitting perpendicular to the grain: $h_e = 40$ mm, $h = 150$mm

$F_{90.Rk} = 14.48 \times \left[\frac{40}{\left(1-\frac{40}{148}\right)}\right]^{0.5} = 4.975 \times 10^3$

$F_{90.Rk1} = 4975$kN

$F_{90.Rd} = k_{mod} \times \frac{F_{90.Rk1}}{\gamma_M} = 2.132 \times 10^6$ N

$F_{90.Rd} > F_{maxJ2}$ OK

So, the Joint will not fail before the truss buckling.

Minimum required spacings of nails:

$a_1 = d \times (5+ 5\cos 57)) = 23.748$ mm

$a_2 = 5 \times d = 12.5$ mm

$a_3.t = d \times (10+ 5 \cos(57)) = 36.248$ mm

$a_{3.c} = 10 \times d = 25$ mm

$a_{4.t} = d \times (5 + 2 \sin(57)) = 14.681$ mm

$a_{4.c} = 5 \times d = 12.5$ mm

Designed spacings of nails:
a_1= 50mm, a_2= 20mm, a_{3t}= 40mm, a_{3c}= 30mm, a_{4t}= 30mm and a_{4c}= 40mm

Vertical Joint:
$F_{v.d.1} = F_{v.k} \times 8 \times 2 = 6.34$ kN

$F_{maxJ1} = 0.7$kN

$\frac{F_{maxJ1}}{F_{v,d,1}} = 0.11$ Ok

Splitting perpendicular to the grain: $h_e = 40$ mm

$$F_{90.Rk} = 14.48 \times \left[\frac{40}{\left(1-\frac{40}{148}\right)}\right]^{0.5} = 4.975 \times 10^3$$

$F_{90.Rk1} = 4975$kN

$$F_{90.Rd} = k_{mod} \times \frac{F_{90.Rk1}}{\gamma_M} = 2.132 \times 10^6 \text{ N}$$

$F_{90.Rd} > F_{maxJ1}$ OK

So, the Joint will not fail before the truss buckling.

Minimum required spacings of nails:

$a_1 = d \times (5 + 5\cos(0)) = 25$ mm

$a_2 = 5 \times d = 12.5$ mm

$a_{3.t} = d \times (10 + 5\cos 0)) = 37.5$ mm

$a_{3.c} = 10 \times d = 25$ mm

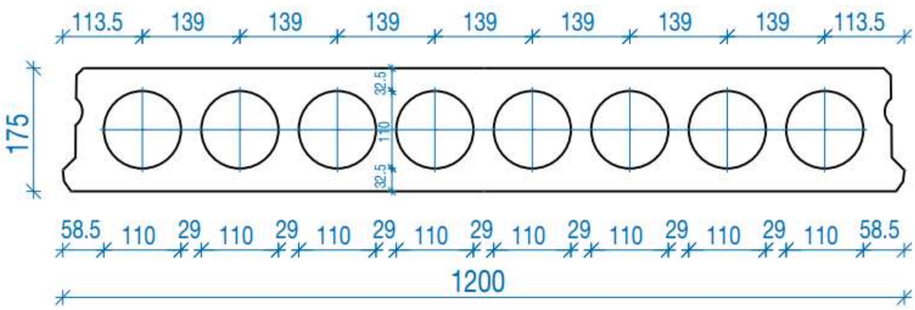
$a_{4.t} = d \times (5 + 2\sin(0)) = 12.5$ mm

$a_{4.c} = 5 \times d = 12.5$ mm

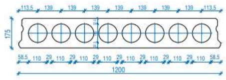
Designed spacings of nails:
a_1= 40mm, a_2= 20mm, a_{3t}= 30mm, a_{3c}= 30mm, a_{4t}= 20mm and a_{4c}= 20mm

7.4 Intermediate floor slab

A hollow core slab P18 from Parma was chosen whereas a cross-sectional property of the slab from Kingspan was shown as an example. The structural properties of the slab according to the given data on their website are shown below:

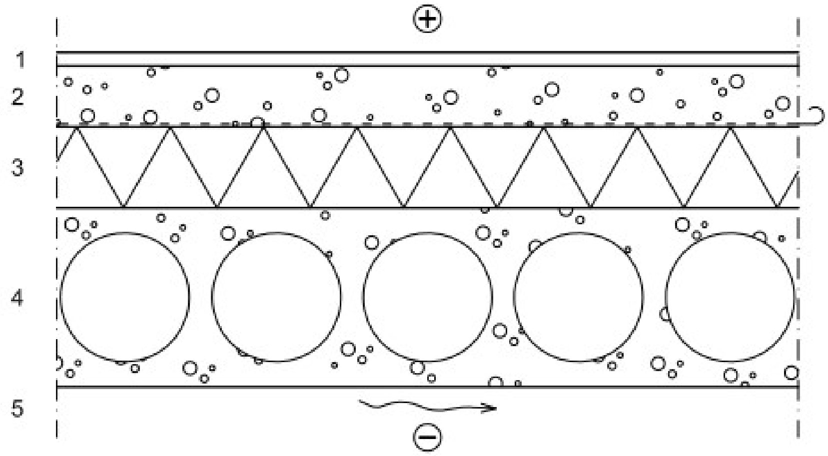


ONTELOLAATAT – OMINAISUUSTAULUKKO

Ontelo-laatta	Poikkileikkaus	Suunnittelu-tukipinta mm	Laatan omapaino kg/m²	Laatan paino saumattuna kg/m²	Palon-kestävyys kantavana ja osastoivana rakenteena
P18		60	265	280	REI30 REI60

Paloluokka	Laattatyyppi	Paloeriste	Esimerkki*
REI120	P18, P18M, P20, P27, P32, P37, P40, P50	50 mm 20 mm	Paroc-Cos4, Paroc-AKU Paroc–FPS 14 tai Promatec-palonsuojalevy
REI180	P18, P18M, P20, P27, P32, P37, P40, P50	60 mm	Paroc–FPS 14
REI240	P18, P18M, P20, P27, P32, P37, P40, P50	80 mm	Paroc–FPS 14
	2P27, 2P32, 2P37, 2P40, 2P50	20 mm	Promatec-palonsuojalevy

Figure 51 shows the detailed properties and composition of different structural members attached in the chosen hollow core floor slab.

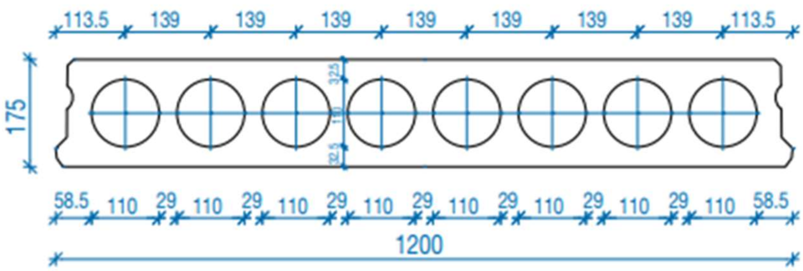


Rakenne	1	Pintamateriaalit/käsittely
	2	Betonivalu rakennesuunnitelmien mukaan
	3	Kingspan Therma™ TF70 120 mm, saumat vaahdotetaan
	4	Kantava ontelolaatta rakennesuunnitelmien mukaan
	5	Tuulettuva tila
U-arvo		0,17 W/m²K (TF70 λ_U 0,022 W/mK)

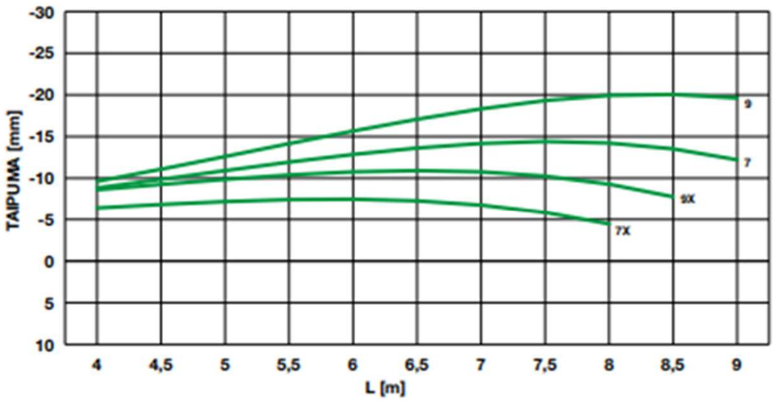
Figure 51. Hollow core slab (Kingspan eristeet, Tunnus: AP 3.2.0)

P18-ontelolaatta

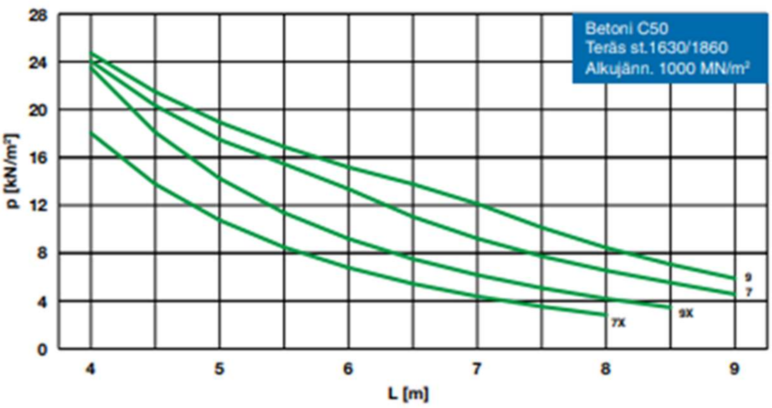
POIKKILEIKKAUS



TAIPUMA P18



KANTOKYKY P18 – asunnot, toimistot, lumikuorma



Ontelolaatan viivakuormakestävyys kN/m				Viivakuorma laataston keskellä		
Saumavalu C20/25				Ei pintavalua		
Laatan pituus	P18, P18M, P20	P27	P32	P37	P40, P40R	P50, P50R
Pituus 4000 mm	13	24	21	33	24	27
Pituus 6000 mm	10	22	21	33	24	27
Pituus 8000 mm	8	18	20	33	24	27
Pituus 10000 mm		15	17	29	24	27
Pituus 12000 mm			15	25	21	27
Pituus 14000 mm				22	19	27
Pituus 16000 mm					17	24

Ontelolaatan viivakuormakestävyys kN/m				Viivakuorma laataston reunassa		
Saumavalu C20/25				Ei pintavalua		
Laatan pituus	P18, P18M, P20	P27	P32	P37	P40, P40R	P50, P50R
Pituus 4000 mm	6	12	10	16	12	13
Pituus 6000 mm	4	10	10	16	12	13
Pituus 8000 mm	3,5	8	9	16	12	13
Pituus 10000 mm		7	8	13	12	13
Pituus 12000 mm			7	11	10	13
Pituus 14000 mm				10	9	13
Pituus 16000 mm					8	11

7.5 Design of load bearing partition wall

The partition wall usually placed in the middle is playing as the main load carrying element for this building.

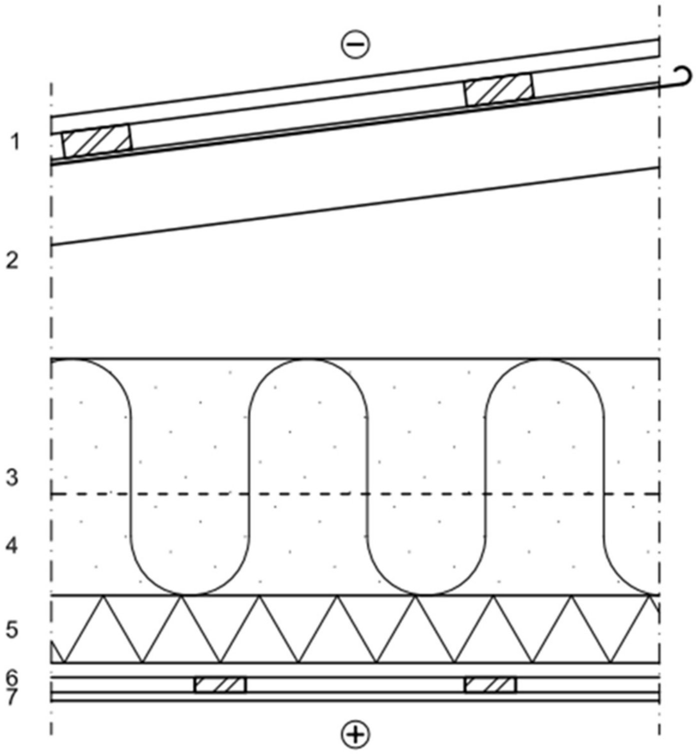
Here,

γ_C	Partial factor for concrete
γ_G	Partial factor for permanent actions, G
γ_M	Partial factor for a material property; γ_m
γ_Q	Partial factor for variable factor, Q
γ_S	Partial factor for reinforcing
ε_c	Compressive strain in the concrete
ε_{c1}	Compressive strain in the concrete at the peak stress f_c
ε_{cu}	Ultimate compressive strain in the concrete
ε_u	Strain of reinforcement at maximum load
ε_{uk}	Characteristic strain of reinforcement at maximum load
ψ_0	Combination values
ψ_1	Frequent values
ψ_2	Quasi-permanent values
μ	Relative moment
A	Accidental area
A	Cross sectional area
a	Distance
A_c	Cross sectional area of concrete
A_s	Cross sectional area of steel
$A_{s,min}$	Minimum cross sectional area of reinforcement
A_{sw}	Cross sectional area of shear reinforcement

b	Overall width of a cross-section
b_w	Width of the web on T, I or L beams
d	Depth
d	Effective depth of a cross-section
e	Eccentricity
E	Effect of action
e_0	Basic eccentricity
$E_{c,eff}$	Effective modulus of elasticity of concrete
E_{cd}	Design value of modulus of elasticity of concrete
E_{cm}	Secant modulus of elasticity of concrete
e_i	Additional eccentricity
EI	Bending stiffness
E_s	Design value of modulus of elasticity of reinforcing steel
e_{Tot}	Total eccentricity
F	Action
f_c	Compressive strength of concrete
f_{cd}	Design value of concrete compressive strength
$f_{cd,pl}$	Design value of plane concrete (non-reinforced) compressive strength
f_{ck}	Characteristic compressive cylinder strength of concrete at 28 days
f_{cm}	Mean value of concrete cylinder compressive strength
f_{ctk}	Characteristic axial tensile strength of concrete
f_{ctm}	Mean value of axial tensile strength of concrete
F_d	Design value of an action
F_k	Characteristic value of an action
f_y	Yield strength of reinforcement
f_{yd}	Design yield strength of reinforcement
f_{yk}	Characteristic yield strength of reinforcement
f_{ywd}	Design yield of shear reinforcement
g	Dead load; Permanent load
G_k	Characteristic permanent action
h	Height
h_w	Thickness of a cross-section
i	Radius of gyration
I	Secant moment of area of concrete
k	Coefficient; Factor
L	Length
l	Length of the footing
L	Length; Span
l_w	Height of the wall
M	Bending moment
m_d	Design value for bending moment
M_{Ed}	Design value of the applied internal bending moment
N	Axial force
n	Relative normal force or load level
N_c	Neutral axis for compressive strength
n_d	Design load

N_{Ed}	Design value of the applied axial force (tension or compression)
N_s	Neutral axis for tensile strength
P_d	Design value for soil pressure
P_k	Characteristic soil pressure
q	live load
Q_k	Characteristic variable action
R	Resistance
R_d	Design value for the resistance
S	Spacing for reinforcement
SLS	Serviceability limit state
t	Thickness
t	Time being considered
t_0	The age of concrete at the time of loading
u,v,w	Component of the displacement of a point
ULS	Ultimate limit state
V	Shear force
V_{Ed}	Design value of the applied shear force
V_{Ed}	Design value of the applied shear force
x	Neutral axis depth
z	Lever arm of internal force
α	Angle; Ratio
α_{cc}	Coefficient taking into account for long term effects on compressive strength and unfavourable effects from the way the load is applied
β	Angle; Ratio; Coefficient depending on the support system; Relative compression height
θ	Angle
λ	Slenderness ratio
λ_{lim}	Limited slenderness ratio
ρ	Oven-dry density of concrete in kg/m^3
ρ_1	Reinforcement ratio for longitudinal reinforcement
ρ_w	Reinforcement ratio for shear reinforcement
σ_c	Compressive stress in the concrete
σ_{cp}	Compressive stress in the concrete from axial load
σ_{cu}	Compressive stress in the concrete at the ultimate compressive strain ε_{cu}
Φ	Coefficient corresponding to eccentricity and creep
ω	Mechanical reinforcement ratio
γ	Partial factor
δ	Redistribution ratio
$\varphi(\infty, t_0)$	Final value of creep coefficient
$\varphi(t, t_0)$	Creep coefficient, defining creep between times t and t_0, related to elastic deformation at 28 days
ψ	Factors defining representative values of variable actions

Figure 52 shows the cross sectional view of the roof. The figure also gives the information about the building materials used in the roof.



Rakenne	1	Huopa-, pelti- tai tiilikate alusrakenteineen
	2	Tuulettuva tila ≥100 mm, tuuletus harjalta ja päädystä
	3	Puhallusvilla 350 mm
	4	Alapaarre k900 rakennesuunnitelmien mukaan
	5	Kingspan Therma^{TM} TP10 100 mm, saumat vaahdotetaan
	6	Asennustila, ristikoolaus 22x75 mm, toisiokannattajat k400
	7	Sisäverhouslevy

U-arvo 0,07 W/m²K (TP10 λ_U 0,022 W/mK, puhallusvilla λ_U 0,041 W/mK)

Figure 52. Roof element (Kingspan eristeet Tunnus: YP 1.5.2.h)

Figure 53 gives a clear picture to understand how the load is arranged in this building. Moreover, the figure helps to understand and calculate in further that how much load the partition wall can carry.

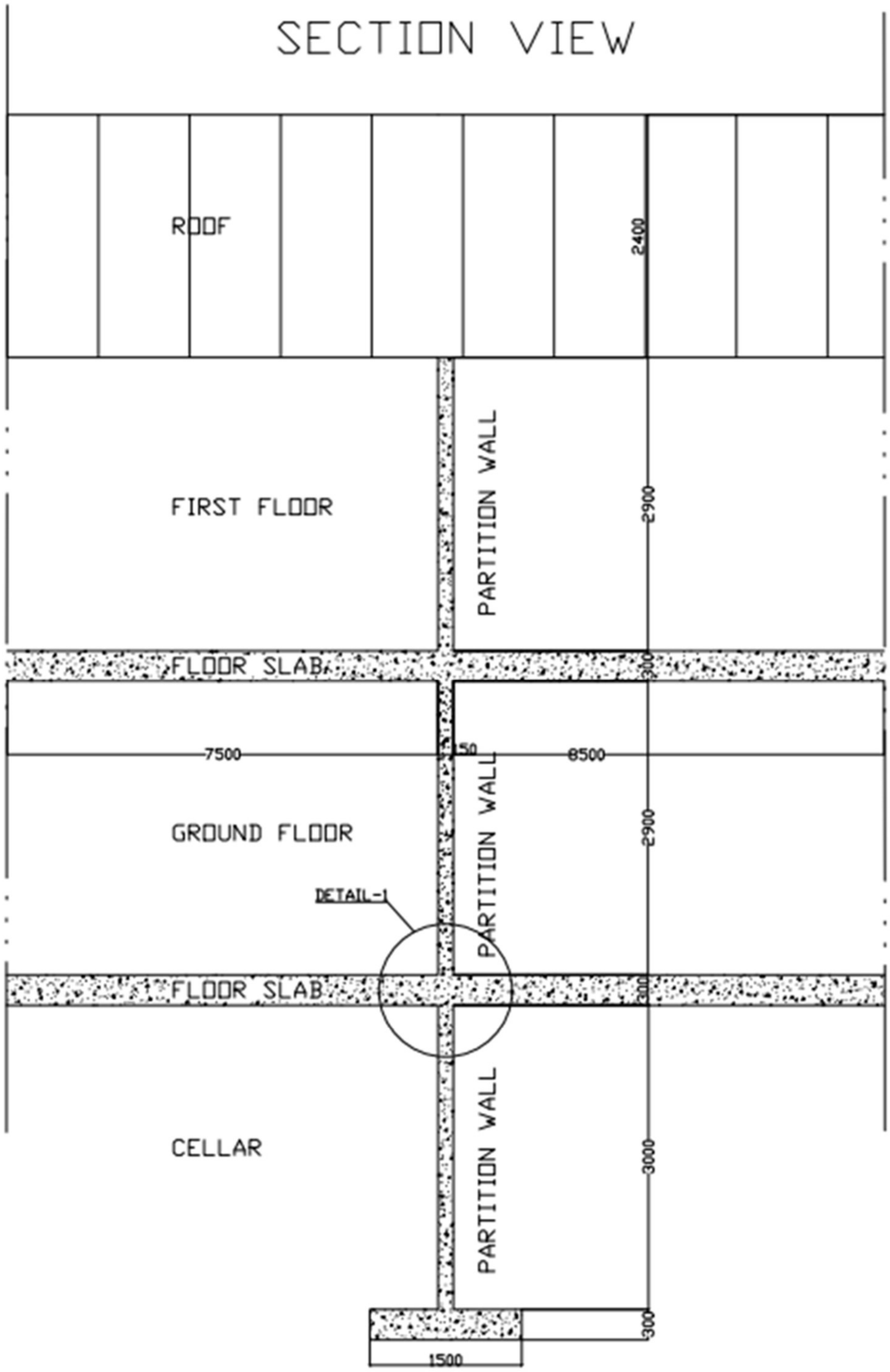


Figure 53. Section of the building with detailed dimensions applied to the load bearing partition wall

Figure 54 shows the bigger and detailed drawing version of the critical load point for the structural design of the partition wall.

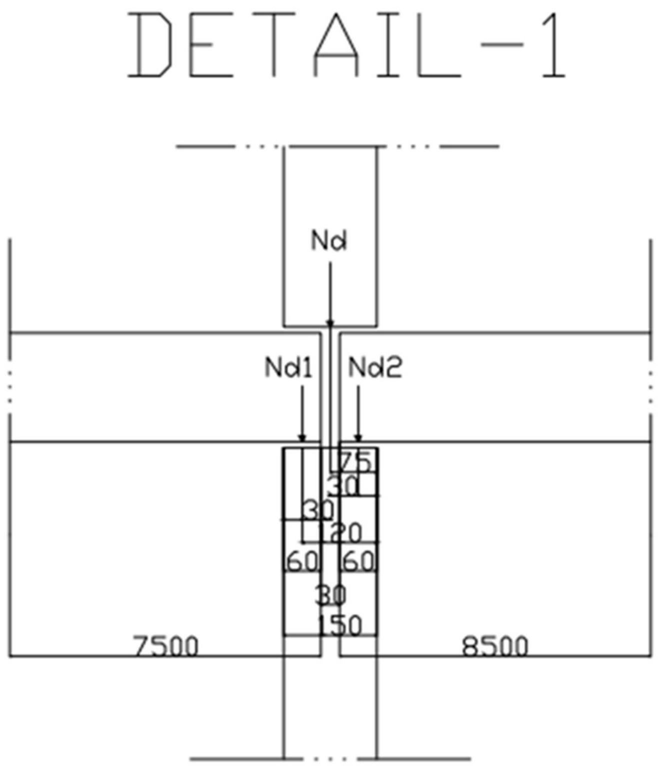


Figure 54. Detail section view with dimensions of the load bearing partition wall

Here,
e_1= 30 mm; e_2 = 120 mm; e_3= 75 mm
L_1=8.5 mm; L_2= 7.5 m
For insulation (450 mm), ρ= 40 kg/m^3 (Kingspan product)

$\therefore$ q = $\frac{40}{100}$ kn/m^3×0.45m = 0.18~0.2 kN/m^2

G_k(truss) = 0.25 kN/m^2
Let's assume, G_k = 1 kN/m^2

$n_{d,roof}$ = 0.5(8.5m+7.5m) [1.35(1.0 kN/m^2)+1.5(2.0 kN/m^2)] = 34.8 kN/m^2
g_{HCS} = 280 kg/m^2 = 2.8 kN/m^2
g_{con} (70 mm) = 0.07×25 kN/m^3 = 1.75 kN/m^2
g_{ins} = 0.2 kN/m^2
g_{tim} (10 mm) = 0.07 kN/m^2

So, g_{beam} = 5.0 kN/m^2

n_{d1} = 0.5× 8.5 m (1.35×5.0 kN/m^2+1.5×2.0 kn/m^2) = 41.4 kN/m
n_{d2} = 0.5× 7.5 m (1.35×5.0 kN/m^2+1.5×2.0 kn/m^2) = 36.6 kN/m
$n_{d1,wall}$ = (2.9 m×0.15×25 kn/m^3) × 1.35 = 14.7 kN/m

εn_d= N_{Ed} = $n_{d,roof}$ +1. (n_{d1}+n_{d2})+2.$n_{d,wall}$
= 34.8 kN/m + (41.4+36.6) kN/m + 2×14.7 kN/m
=142.2 kN/m

It denotes the resultant of the load on the roof of first floor.
So, the position of the resultant:

e = $\frac{M_{Ed}}{N_d}$
= $\frac{n_{d1}\times e_1+n_{d2}\times e_2+\varepsilon_{n_d}\times e_3}{n_{d1}+n_{d2}+\varepsilon_{n_d}}$
= $\frac{41.4\ kN\times 30\ mm+36.6\ kN\times 120\ mm+1\quad .2\ kN\times 75\ mm}{41.4\ kN+36.6\ kN+142.2\ kN}$
= $74\ mm$

e_0 = $|e-e_3|$
= $|74-75|$
= $1\ mm$

e_o ≥ 20 mm or
e_o ≥ $\frac{h_w}{30}$ = $\frac{150mm}{30}$ = 5 mm

again, e_i = $\frac{l_0}{400}$
e_{tot} = e_o + e_i
= (1 + 5.3) mm
= 6.3 mm

But , e_{tot} ≥ 20 mm
e_{tot} ≥ $\frac{h_w}{30}$ but always ≥ 20 mm

$\frac{e_{tot}}{h_w}$ = $\frac{20\text{ mm}}{150\text{ mm}}$ = 0.13

$\frac{l_0}{h_w}$ = $\frac{2125\text{ mm}}{150\text{ mm}}$ = 14.16

Now,
$\lambda = \frac{l_0}{i}$
i = $\sqrt{I/A} = \sqrt{\frac{bh^3/12}{b.h}}$ = 0.289 h = 0.289 × 150 mm
= 43.4 mm

l_0 = β. l_ω = $\frac{1}{1+\left(\frac{l_w}{b}\right)^2}\times l_w$
= $\frac{2900mm}{1+\left(\frac{2900mm}{4750mm}\right)^2}$
= 2112.6 mm
≈ 2125 mm

$e_i = \frac{l_0}{400} = \frac{2125\text{mm}}{400}$
= 5.3 mm

$\lambda = \frac{2125\text{mm}}{43.4\text{ mm}}$ = 48.96 ~ 49

If breadth
b$\geq l_w$ then

$$\beta = \frac{1}{1+\left(\frac{l_w}{b}\right)^2}$$

And if b$< l_w$

$$\beta = \frac{b}{2l_w}$$

b= 4750 mm
l_w = 2900 mm

$\lambda_{lim} = 20 \times A \times B \times C \times \frac{1}{\sqrt{n}}$ $= 20 \times 0.7 \times 1.1 \times 0.7 \times 1$ $= 10.8$	A = 0.7 B = 1.1 C = 0.7, n= 1 if $f_{ck} \leq 50MP_a$

n = $\frac{N_{Ed}}{A_c \times f_{cd} \times p_l}$

N_d, b, wall = 1.35 × 0.15m × 2.3 m × 25 kN/mm^3 = 11.6 kN/m

$N_{Ed} = n_{d,roof} + 2 \times (n_{d_1} + n_{d_2}) + 2n_{d,wall} + n_{d,b,wall}$
= 34.8 kN/m + 2 × (41.4+ 36.6) kN/m + 2 × 14.7 kN/m
+ 15.3kN/m
= 235.5kN/m ~ 235.5 N/mm

$f_{cd.p1} = \frac{\alpha_{cc,pl}}{\gamma_c} \times f_{ck}$
= 0.8 × $\frac{0.85}{1.5}$ × 25 N/mm^2
= 11.3 N/ mm^2
= 113000 kn/mm^2

A_c = b × h = 2900mm × 150 mm = 435000 mm^2

∴ n = $\frac{235.5\text{ N/mm}}{435000\text{ mm}^2 \times 11.3\text{ kN/mm}^2}$ = 4.8 × 10^{-5} N/mm

∴ We found, $\lambda \geq \lambda_{lim}$; $49 \geq 10.8$;

∴ So, $N_{Rd} = b \times h_w \times f_{cd.p1} \times \emptyset$

If $\lambda \leq \lambda_{\text{lim}}$; then $N_{rd} = n \times f_{cd} \times p1 \times b \times h_w (1 - 2 \times \frac{e}{h_w})$

$$\phi = 1.14 \text{ x } (1 - 2 \text{ x } \frac{etot}{h_w}) - 0.02 \text{ x } \frac{lo}{h_w} \leq (1 - 2 \text{ x } \frac{etot}{h_w})$$
$$= 1.14 \text{ x } (1 - 2 \text{ x } \frac{20mm}{150mm}) - 0.02 \text{ x } \frac{2125mm}{150mm} \leq (1 - 2 \text{ x } \frac{20mm}{150mm})$$
$$= 0.55 \text{ mm} \leq 0.73$$
$$= 0.6$$

$R_d = 1000 \text{ mm} \times 150 \text{ mm} \times 11.3 \text{ N/ mm}^2 \times 0.6$
$= 1017 \text{ kN}$

$R_d > N_{Ed}$ Ok

7.6 Design of load bearing external frame wall

The following type of wall was chosen from many offered by the company. This wall has a timber external cladding. Figure 55 shows the cross sectional view of the external wall. The figure also gives the information about the building materials used in the wall.

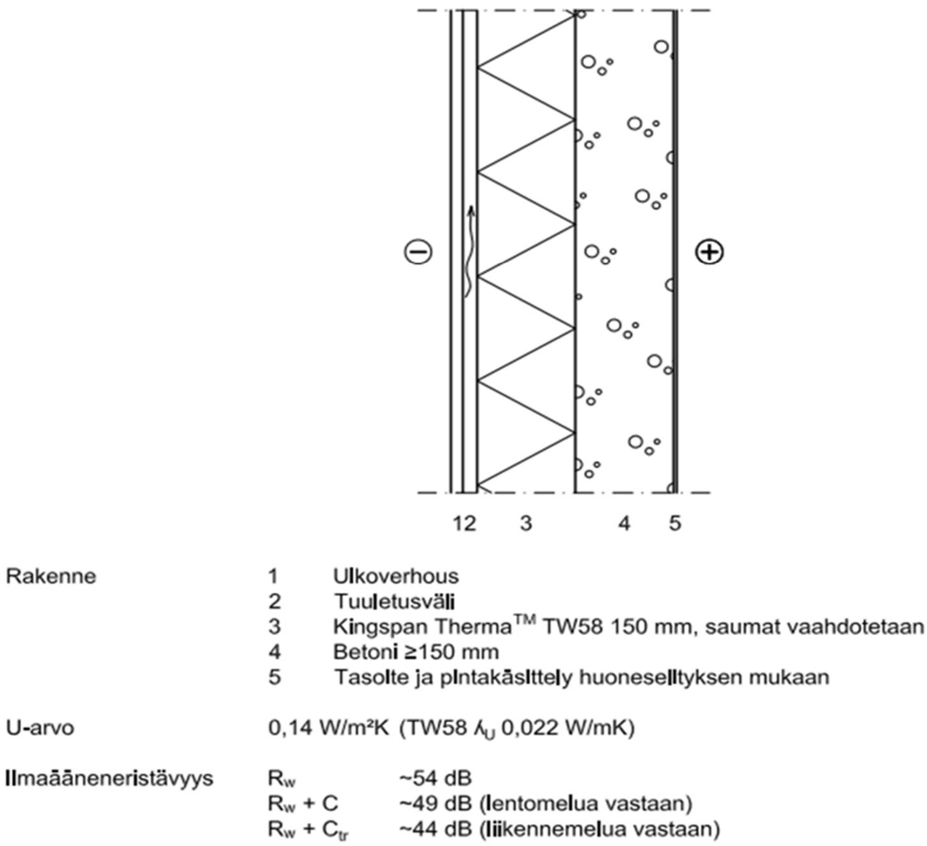


Figure 55. External wall (Kingspan eristeet Tunnus: US 5.1.1)

Figure 56 gives a clear picture to understand how the load is arranged in this building. Moreover, the figure helps to understand and calculate in further that how much load the frame wall can carry.

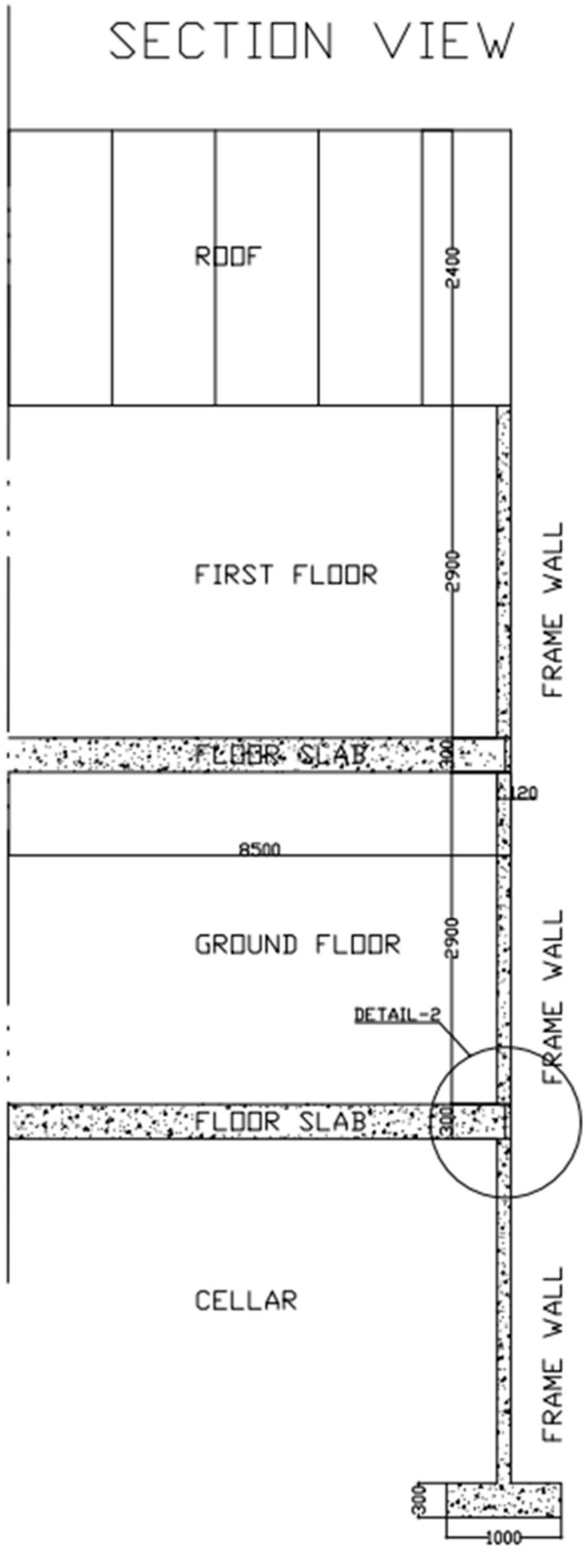


Figure 56. Section of the building with detailed dimensions applied to the load bearing external wall

Figure 57 shows the bigger and detailed drawing version of the critical load point for the structural design of the frame wall.

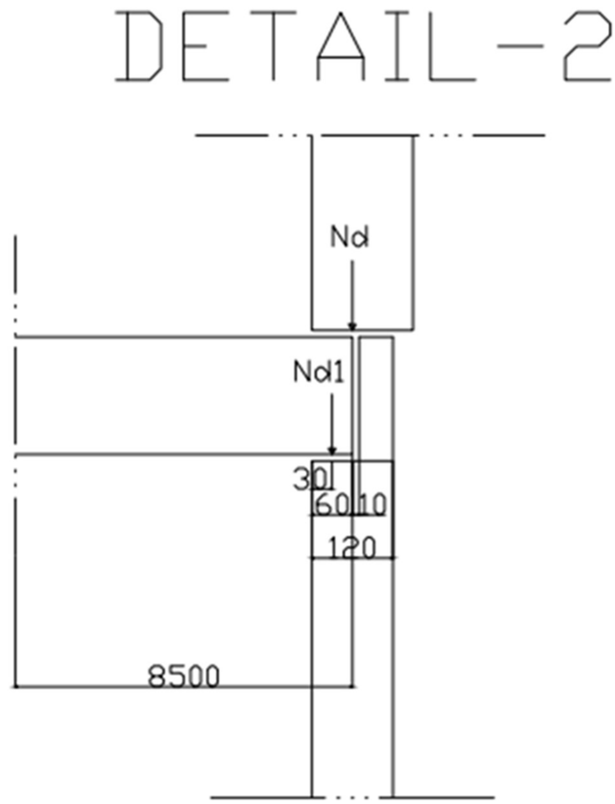


Figure 57. Detail section view with dimensions of the external load bearing wall

$n_{d,roof}$ = 0.5 x 8.5 m $[1.35 \times 1.0\ kN/m^2\ + 1.5\ \times\ 2.0\ kN/m^2]$
= 18.5 kN/m

n_{d1} = 0.5 × 8.5 $[1.35 \times 5.0\ kN/m^2\ + 1.5\ \times\ 2.0\ kN/m^2]$
= 41.4 kN/m

$n_{d,wall}$ = 2.9m × 0.12m × 25 kN/m^3 × 1.35
= 11.8 kN/m

$n_{d,b,wall}$ = 3.0m × 0.15m ×25 kN/m^3 × 1.35
= 15.3 kN/m

$n_{d1,b,wall}$ = 2.3m × 0.12m ×25 kN/m^3 × 1.35
= 9.3 kN/m

$\sum n_d$= (18.5+41.4+2 ×11.8) kN/m = 83.5 kN/m

e = $\frac{41.4k\quad \times 30\ mm + 83.5\ \times 60}{41.4\ kN + 83.5\ kN}$ = 50 mm

e_o = $|e - e_1|$ = |50 - 60| = 10 mm

e_{tot} = (10 + 6.3) mm = 16.3 mm

$\frac{e_{tot}}{h_w} = \frac{20mm}{120mm} = 0.17$

$\frac{l_o}{h_w} = \frac{2125\ mm}{120\ mm} = 17.7$

$\phi = 0.4$

h= 0.289 × 120 mm = 34.7 mm

$\lambda = \frac{2125\ mm}{34.7mm} = 61.2$ mm

λ_{lim} = 10.8mm

$\phi_d = 1.14\ (1 - 2 \times \frac{20\ mm}{120\ mm}) - 0.02 \times \frac{2125mm}{120mm} \leq 1 - 2 \times \frac{20mm}{120mm}$
= 0.41 ≤ 0.67

N_{Rd} = 1000 mm × 120 mm × 11.3 N/mm^2 × 0.4 = 542.4 kN

N_{Ed} = 18.5 kN/m + 2 × 41.4 kN/m + 2 × 1.8 k N/m + 15.3 kN/m
= 140.2 kN/m

∴ **$N_{Rd} > N_{Ed}$** Ok

7.7 Design of footing for the partition or middle wall

Characteristic load:

Roof: $n_{k,roof}$ = (0.5 × 8.5m) × (1.0kN/m^2 + 2.0kN/m^2) = 12.75kN/m
Floor: $n_{k,floor}$= 0.5×(8.5m + 7.5m)x(5.0kN/m^2+2.0kN/m^2) = 129.75kN/m
Wall: $n_{k,wall}$ = 2.9m × 0.12m × 25KN/m^3) = 18.7kN/m
Footing: $n_{footing}$ = 0.5m × 1.5m × 25KN/m^3) = 18.7kN/m

$\sum n_k = nk_{roof} + 2 \times n_{k,floor} + 3 \times n_{k,wall} + n_{k,footing}$
= 24kN/m + 2 × 56kN/m + 2 × 10.9kN/m + 18.8kN/m
= 193.1kN/m

P_k = 150 kN/m^2
$P_k = \frac{\sum n_k}{l \times b}$
$\rightarrow l = \frac{\sum n_k}{l \times \sum P_k}$
$= \frac{193.1\ \text{kN}}{1.0\ m\ x\ 150kN/m^2}$
= 1.29m

Width, b = 1000 mm (Always)
Length, l = 1500mm (Initial assumption)
Thickness, h = 500mm

So, l = 1.5m (Assumption is correct).

Concrete C 25/30
Exposure class of foundation is XC2

$$C_{min} = \begin{cases} C_{min} \times b = 12\ mm \\ C_{min,Dur} = 15mm \\ 10\ mm \end{cases}$$

$C_{nom} = C_{min} + \Delta C_{DEV}$
$= 15mm + 30\ mm$
$= 45 \sim 50mm$

$f_{cd} = \frac{0.85 \times 25\ \text{MPa}}{1.5}$
$= 14.2 N/mm^2$

$f_{ctd} = \frac{f_{ctk,0.05}}{1.5}$
$= \frac{1.8\ \text{MPa}}{1.5}$
$= 1.2 N/mm^2$

Designing the height or thickness of the foundation at ULS:

Roof: $n_{d,roof}$ = 0.5×(8.5m+7.5m)×(1.35×1.0kN/m^2+1.5×2.0kN/m^2)
= 34.8kN/m
Floor: $n_{d,floor}$ = 0.5×(8.5m+7.5m)×(1.35×5.0kN/m^2+1.5×2.0KN/m^2)
= 78.0KN/m
Wall: $n_{d,wall}$ = 1.35 m × 2.9 m × 0.15m x 25kN/m^3) = 14.8KN/m
Cellar Wall: $n_{c,wall}$ = 1.35 × 3.3m × 0.2m × 25kN/m^3) = 22.3KN/m
Footing: $n_{d,footing}$ = 25.4KN/m

$\sum n_d = n_{d,roof} + 2 \times n_{d,floor} + 2 \times n_{d,wall} + n_{c,wall} + n_{d,footing}$
= 34.8kN/m + 2 × 78.0 kN/m + 2 × 14.8kN/m
+ 25.4kN/m + 22.3KN/m
= 268kN/m

$P_d = \frac{\sum n_d}{b \times l}$
$= \frac{261\text{kN}}{1.0\ m \times 1.5m}$
= 178.7 KN/m^2
= 0.178 N/mm^2

Effective Height,
$d \geq \frac{c}{1+0.3 \times \frac{f_{ctd}}{P_d}}$
$= \frac{675\text{mm}}{1+\frac{0.3 \times 1.2 N/mm^2}{0.178 N/mm^2}}$
= 220mm

∴ Required, h = 215 mm + 50 mm + $\frac{12mm}{2}$
= 271 mm
∴ We can choose, h = 300 mm

$\therefore$ d = (300 – 50 – 6) mm
= 244 mm

The cross-section view of the wall footing is shown below in Figure 58.

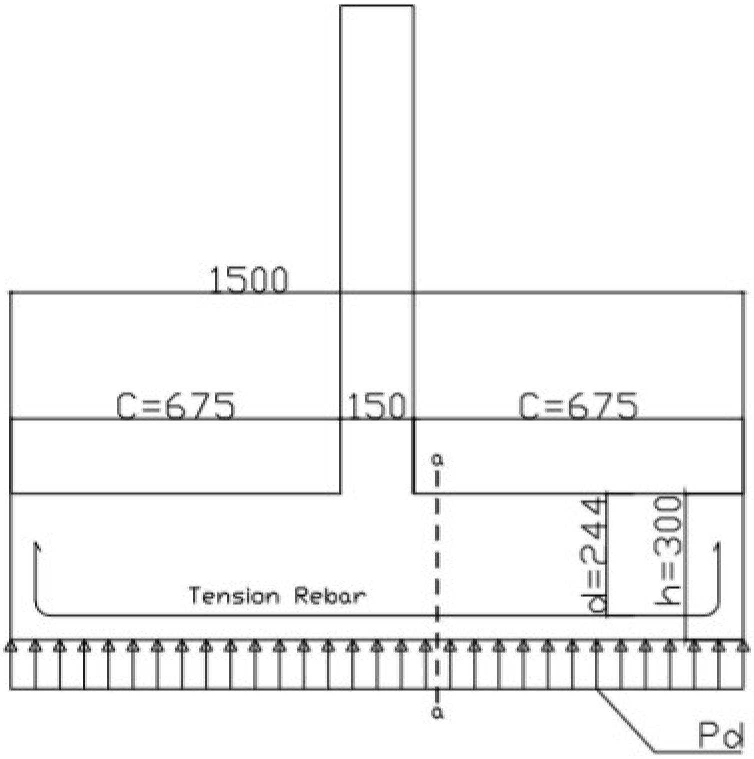


Figure 58. Cross-section of the foundation under the load bearing partition wall in the middle of the building

$m_{d(a\text{-}a)} = 0.5 \times P_d \times C^2 = 0.5 \times 173.7$ kN/m $\times (0.675\text{m})^2$
= 40.7 kN/m
= 40.7×10^6 Nmm

Relative moment,

$$\mu = \frac{m_{\mathrm{d(a-a)}}}{b \times d^2 \times f_{cd}}$$

$$= \frac{40.7 \times 10^6 \text{ nm}}{1000\text{mm} \times (244\text{mm})^2 \times 19.8\text{kN/m}^2}$$

= 0.345 < 0.358 OK

$$\beta = 1- \sqrt{1-2u}$$

$$= 1- \sqrt{1 - 2 \times 0.345}$$

= 0.044 < 0.467 OK

$$Z = d\left(1 - \frac{\beta}{2}\right)$$

$$= 244(1- \frac{0.044}{2})$$

= 190.32mm

$$A_s = \frac{m_{\mathrm{d(a-a)}}}{z \times f_{yd}}$$

$$= \frac{40.7 \times 10^6 \text{ Nmm}}{191\text{mm} \times 435\text{N/mm}^2}$$

= 490 mm^2 / METER

$$f_{yd} = \frac{f_{yk}}{\gamma_s}$$
$$= \frac{500\ N/mm^2}{1.15}$$
$$= 435\ \text{N/mm}^2$$

$$A_{S.min} = \max\begin{cases}0.26 \times bd \times f_{c.t.m}/f_{yk} \\ 0.0013 \times bd\end{cases}$$

$$= \max\begin{cases}0.26 \ \times 3.2/500 \times 1000 \times 244 \\ 0.0013 \times 1000 \times 244\end{cases}$$

$$= \max\begin{cases}406\ mm^2/\text{METER} \\ 317\ mm^2/\text{METER}\end{cases}$$

So, A_s = 490 mm^2 / METER is ok.

Hence, we choose T12 Rebars

$$S = \frac{1000\ mm \times 113\text{mm}^2}{490\ \text{mm}^2}$$
$$= 230\ \text{mm} \sim 250\text{mm}$$

So, $A_{s3} = 0.2 \times 490\ \text{mm}^2 = 98\ \text{mm}^2$ [20% of main bars]

$$P_{CS\ T12} = \frac{A_S}{A_{T1}}$$
$$= \frac{490\text{mm}^2}{113\text{mm}^2}$$
$$= 4.33 \sim 5$$

The longitudinal arrangement of reinforcement is shown in Figure 59.

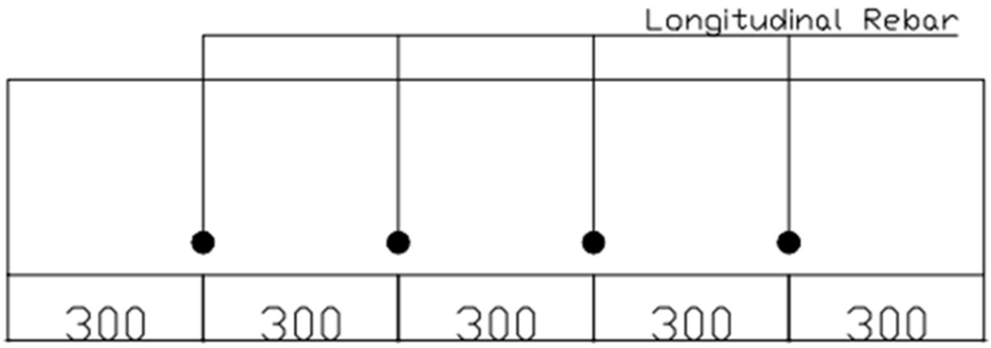


Figure 59. Bar arrangement in the foundation

So, we will use 5 pcs of bar.

Similarly, for the corner or frame wall, foundation dimension 300mm × 1000mm can be taken from a gentle man guess since this edge foundation carries fewer loads than that of the middle.

7.8 Foundation engineering

In this section, soil pressure acting on the basement wall is basically tried to find out and verify with the designed thickness of wall whether or not that can withstand without deflection against that pressure. In order to do this, maximum allowable load bearing capacity of the foundation is also investigated.

Here,

γ_1	Bulk weight volume of soil above the footing
γ_2	Bulk weight volume of soil under the footing
D	Depth of footing from the zero plane.
B	Width of footing.
N_B/N_D	Factor of safety for friction
ϕ_k	Characteristic friction angle
C	Cohesion

Design conditions for the calculation:

D=D_1

$\gamma_1 = 20\ kN/m^3$

$\gamma_2 = (20 - 10)\ kN/m^3 = 10\ kN/m^3$

F.O.S = 1.25

$\phi_k = 38^o$ (From Table 31.)

C= 0

Figure 60 shows the formation of the underground level of the building.

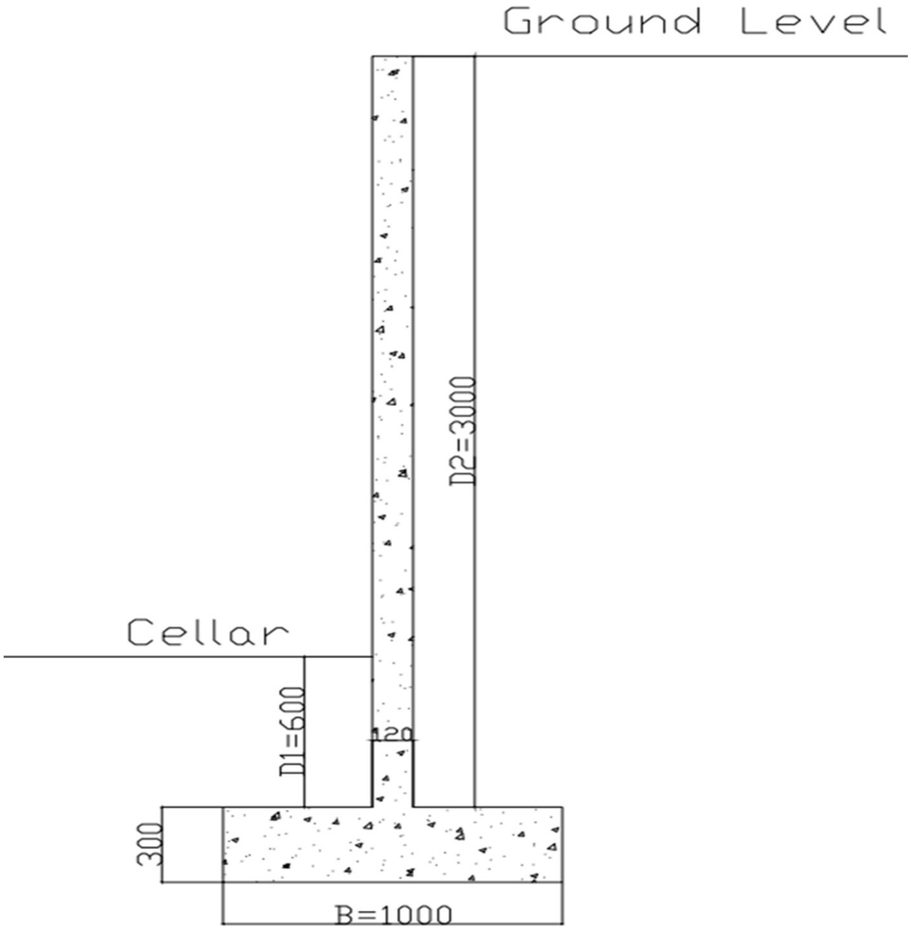


Figure 60. Foundation of the building corresponding to the cellar

$$\phi_D = \tan^{-1}\left(\frac{\tan 38^o}{1.25}\right) = 32^o$$
$$N_D = \left[\tan\left(45^o + \frac{32^o}{2}\right)\right]^2 \times e^{\pi \tan \quad ^o} = 23.2$$
$$N_B = 2(23.2^o - 1) \times \tan 32^o = 27.75$$

$$\delta_{allowed} = 20 \; kN/m^3 \times 23.2 \times 0.6m + 0.5 \times 10 \; kN/m^3 \times 27.75 \times 1.5m = 417.15 \; kN/m^3$$

Maximum load bearing capacity = $417.15 \; kN/m^3 \times 1.0 \; m$
= $417.15 \; kN/wall$ –METER Ok

Passive soil pressure,

$$P_p = 12 \; kN/m^2 \left[\tan\left(45^o + \frac{32^o}{2}\right)\right]^2$$
= $39.1 \; kN/m^2$

Where, $\overline{\delta_p} = \gamma_1 \times D_1 = 20 \; kN/m^3 \times 0.6 \; m = 12 \; kN/m^2$

Active soil pressure,

$$P_a = 60 \; kN/m^2 \left[\tan\left(45^o - \frac{32^o}{2}\right)\right]^2$$
= $18.4 \; kN/m^2$

Where,

$\overline{\delta_a} = \gamma_1 \times D_2 = 20 \; kN/m^3 \times 3 \; m = 60 \; kN/m^2$

$N_{P,Ed} = 39.1 \; kN/m^2 \times 0.6 \; m \times 0.5 = 11.73 \; kN/wall - METER$

$N_{a,Ed} = 18.4 \; kN/m^2 \times 3 \; m \times 0.5 = 27.6 \; kN/wall - METER$

$M_d = 26.7 \; kN \times 2m - 11.73 \; kN \times 2.8m + 0.5 \times 9kN/m^2 \times (3m)^2$
= $62.9 \; kNm$

Relationship between the active and passive soil pressure on the basement wall is shown in Figure 61.

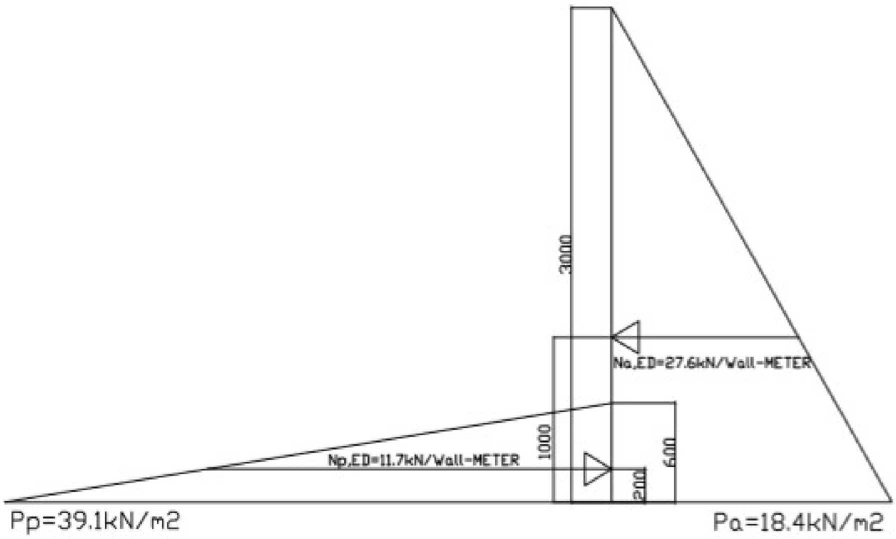


Figure 61. Relationship between the active and passive load resultant on the foundation wall

Now relative moment, [Concrete C 25/30]

$$\mu = \frac{M_d}{b.d^2.f_{cd}}$$

$$= \frac{62.9 \times 10^6 Nmm}{1000\ mm \times (120\ mm)^2 \times 14.2 N/mm}$$

$$= 0.3\ < 3\quad (satisfied)$$

If wall thickness, $d = 150\ mm$ then $\mu = 0.20$

∴ $150\ mm$ thick wall is recommended.

7.8.1 Deflection checking for the basement wall

To verify the resistance capacity (Deflection) of the designed basement wall against the horizontal soil pressure, the following process has been done. Figure 62 shows the forces caused by the soil acting on the wall.

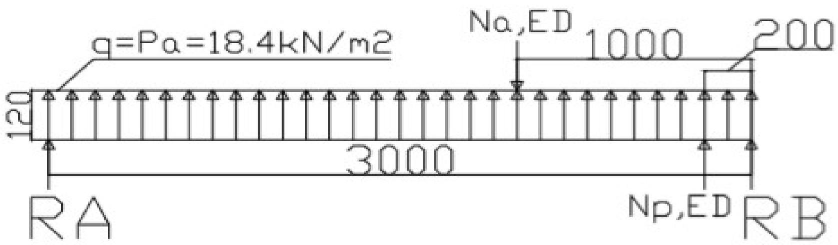


Figure 62. Load distribution along the basement wall

$$I = \frac{bh^3}{12}$$

$$E_{c25/30} = 31\ GPa$$

Deflection,

$$\delta = \frac{Fa^2b^2}{3EIL} \quad \text{or}$$

$$\delta = \frac{5PL^4}{384EI}$$

$P = 1m \times 18.4 kN/m^2$ = 18.4 N/mm

$$I = \frac{3m \times 0.15m^3}{12} = 8.4 \times 10^8 mm^4$$

$$\delta_a = \frac{27.6\ N \times 2000mm^4 \times 1000mm^2}{3 \times 31 \times 10^3\ N/mm^2 \times 8.4 \times 10^8 mm^4 \times 3000mm}$$

$$= 4.71 \times 10^{-4} mm$$

$$\delta_a = \frac{5 \times 18.4 \times 10^{-3}\ N/mm^2 \times 3000mm^4}{384 \times 31 \times 10^3\ N/mm^2 \times 8.4 \times 10^8 mm^4} = 7.45 \times 10^{-4} mm$$

Basement wall selection is ok.

7.9 Moisture behaviour of the structure during winter

The objective of the study in this part is to determine the moisture behavior i.e. whether the water generated moisture from the building structure can go away or where it goes during the winter regarding the amount of vapour coming in and going out through the ventilation gap.

Condition:
The water from the vapour barrier transforms into vapour that rises in the up through the insulation to the ventilation gap and transfers outside of the structure through the ventilation gap in the roof. The rooftop snow acts as insulation and warms up the temperature of the air blowing through the ventilation gap.

7.9.1 Determination of moisture behavior

The moisture behavior for any buildings in Finlnd is very important. If the building is damp and does not get enough facility to dry, there is a high health risk for the dwellers. The process of determining the moisture behavior of a building is shown below.

Z_V = Water vapour resistant
Now from the Table 35,

$Z_{Gypsum\ Board} = 4\times10^3 s/m$
$Z_{Min.Wool} = [\{(100mm/100mm)\times8 + (300mm/100mm)\times8\}\times10^3 s/m]$
$= 32\times10^3 s/m$
$Z_{WindBoard} = (30mm/13mm)\times4\times10^3 s/m$
$= 9.23\times10^3 s/m$
$Z_{Air} = 4\times10^3 s/m$
$Z_{Snow} = 0 s/m$

$Z_{Total} = (Z_{Gypsum\ Board} + Z_{Min.Wool} + Z_{WindBoard} + Z_{Air} + Z_{Snow})$
$= 4\times10^3 s/m + 32\times10^3 s/m + 9.23\times10^3 s/m + 4\times10^3 s/m + 0 s/m$
$= 49.23\times10^3 s/m$

Considering,
Internal temperature, $t_i = +22^0C$
Relative humidity for internal temperature, $RH_i = 45\%$

External temperature, $t_e = -10^0C$
Relative humidity for external temperature, $RH_e = 90\%$

We know that,

$$V_t = \Phi \times V_{t,\ sat}$$
$$\Phi = RH/100$$

Where,

V_t	Volume of actual water vapour content in temperature t^0C
$V_{t,\,sat}$	Volume of saturated water vapour content in temperature t^0C
Φ	Relative humidity

Hence,
Internal water vapour content at $+22^0$ C,

$$V_i = 0.45 \times 19.4 g/m^3$$
$$= 8.73 g/m^3$$

External water vapour content at -10^0 C,

$$V_e = 0.9 \times 2.2 g/m^3$$
$$= 1.98 g/m^3$$

Moisture or vapour flow, $G_{Flow} = [\{T \times (V_i - V_e)/Z_{Total}\}]$
where, T represents the period or time

$G_{Flow} = [(8.73g/m^3 - 1.98g/m^3)/\ 49.23 \times 10^3 s/m) \times 60 \times 60 \times 24s]$
$= 11.8g/m^2$ (per day)………………(1)

Here, in the ventilation or air gap at the roof, the following conditions are assumed:

Temperature in the air gap, $t_{Air} = -\ 6^0C$
Relative humidity in the air gap, $RH_{Air} = 90\%$
Rate of air flow in the air gap, $U_{Air} = 0.2m/s$
Water vapour content in the air gap,

$$V_{Air} = 0.9 \times 3.08 g/m^3$$
$$= 2.77\ g/m^3$$

Again,
Drying capacity of water vapour, G_{Drying} = , $u_{Air} \times A_{Air} (V_{Air} - V_O) \times T$
Since, the opening or ventilation gap is 100mm and the truss spacing is 900mm,
So, the area of opening,

$$A_{Air} = 0.1m \times 0.9m$$
$$= 0.09m^2$$

Now,
$G_{Drying} = 0.2m/s \times 0.09m^2 \times (2.77g/m^3 - 1.98g/m^3) \times 60 \times 60 \times 24s$
$= 1228.6g/m^2$ (per day)

Since, the length of the roof is 16m, so

G_{Drying} per 1m span $= 1228.6g/m^2/16m$
$= 76.8g/m^2$ (per day)………………………(2)

Hence, equation (1) and (2) gives,

$$G_{Drying} > G_{Flow}$$

As we can see, the drying capacity of the ventilation gap is greater than the rate of water vapour flow generated by the structure, so all the vapour can normally pass through the ventilation gap and therefore, there will be no condensation. However, we always have some water on the vapour barrier during the construction, so the question is where this water will go. Therefore, it will condensate on the top of the ventilation gap or just under the roof sheathing in winter time, and it will remain as ice there until summer comes. Moreover, it would further take some years to dry away all the water. Noted that, the drying capacity of the water vapour will be less if there is no rooftop snow because; the snow acts as insulation that leads to warm up the air in the ventilation gap. Consequently, Drying Capacity increases.

7.10 Heat Loss and E-value Calculation

In order to calculate energy efficiency of the building, heat loss is the main key to determine first. Finally, E-value can be found dividing the total heat loss by the net area of the building. The process is shown below.

Here,

T_{in}	Indoor temperature (^oC)
T_{out}	Dimensioning outdoor temperature(^oC)
l_{hb}	Lenght of linear thermal bridge, m
Ψ_{hb}	Additional conductance, W/(mK)
ρ	Air density (1.2kg/m^3)
c	Specific heat capacity of air (1.0 kJ/kgK)
q_{Lair}	Leakage air flow rate (m^3/s)
q_{50}	Leakage air number of building envelope, m^3/(h m^2)
x	Factor depending on floors e.g. 1 floor x=35, e.g. 2 floor x= 25; 3 floor house, x=20 ; 4 floors, x= 15
A_{env}	Area of building envelope incl. Floor, m^2
n_{50}	Leakage air number of building, 1/h
V	Air volume of building, m^3
q_{50}	4 if not known
q_V	Ventilation flow (m^3/s)
η	Efficiency of heat recovery (e.g. 50 % = 0.5)
q_w	Design flow of (m^3/s)
T_c,T_h	Water temperatures , cold, hot (K)
$\varnothing_{Hcirc}$	Heat loss for hot water circulation net
$\varnothing_{Hcirc}$	Generally 5^oC or 0.002 kW/m^2×A
T_{dd}	Temperature in degree days = ($t^0 \times 3600s \times$24h×365d)

Figure 63 shows different dimensions needed for calculating heat losses of the building.

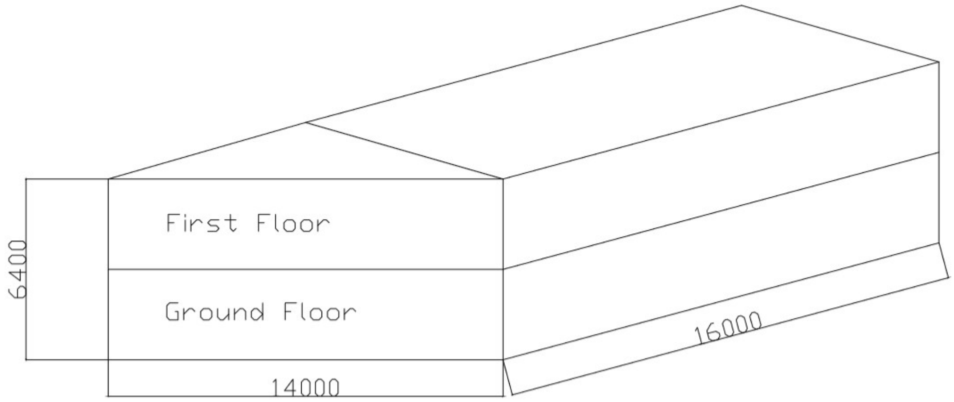


Figure 63. Different dimensions of the building for energy calculation

$$A = 16m\times 14m$$
$$= 224\ m^2$$

$$V = 16m\times 14m\times 6.4m$$
$$= 1433.6 \approx 1433.6\ m^3$$

A_{env}=2(16m×6.4m)+2(14m×6.4m)+2[(16m−2×0.35m)×(143−2×0.35)]
= 791m^2

A_{window} = 1.95m× 1.35m × [4+9+7+4]pcs + 1.35 m×1.35m×[2+2]pcs
= 63.18 m^2+7.29m^2
=70.47 ≈70.5 m^2

A_{Door} = (2.38×1.07)m^2+(2.0×1.7)m^2+(2.4×3.0)m^2
= 13.15 ≈13.5 m^2

A_{wall} = [{2× (16×6.4)+2(14×6.4)}−(70.5+13.2)]m^2
= 300.3 m^2

A_{floor} = [(16m−2×0.35m) ×(14m−2×0.35m)]
= 203.5 m^2

A_{roof} = [(16m−2×0.35m) ×(14m−2×0.35m)]
= 203.5 m^2

Considered U-values are:

Wall= 0.14W/m^2 ^{0}C
Floor= 0.10W/m^2 ^{0}C
Roof= 0.07W/m^2 ^{0}C
Window= 0.7W/m^2 ^{0}C
Door = 0.4W/m^2 ^{0}C

Q_{wall} = 0.14W/m^2 ^{0}C×300 m^2×(22+10)^{0}C
= 84W

Q_{window} = 0.7W/m^2 ^{0}C×70.5 m^2×(22+10)^{0}C
= 1579W

Q_{Door} = 0.4W/m^2 ^{0}C×13.5 m^2×(22+10)^{0}C
= 173W

Q_{Floor} = 0.1W/m^2 ^{0}C×203.5 m^2×(22+10)^{0}C
= 651W

Q_{roof} = 0.07W/m^2 ^{0}C×203.5 m^2×(22+10)^{0}C
= 456W

Q_{str} = 2943W

Q_{therm} = 10% of Q_{str}

= 0.1 × 2943W

= 294.3W

$Q_{Lair} = \rho \times C \times q_L \times (T_{in} - T_{out})$

$q_L = [q_{50}/3600s \times x] \times A_{envelope}$

$$q_{50} = \frac{n_{50} \times V}{A_{envelope}}$$

$$= \frac{4 \times 1434\ m^3}{791 m^2}$$

= 7.3m

$$Q_L = \left[\frac{7.3\ m}{3600s \times 25}\right] \times 791\ m^2$$

= 0.06m^3/s

Q_{Lair} = 1.2 kg/m^3×1.0 kJ/KgC ×0.06 m^3/s × (22+10)

= 2.3×1000

= 2300W

$Q_V = \rho \times C \times q_L \times (T_{in} - T_{out}) \times (1 - \eta)$

$$q_V = \frac{n \times V}{t}$$

$$= \frac{0.5 \times 1434\ m^3}{3600\ s}$$

= 0.2m^3/s

Q_V = 1.2 kg/m^3×1.0 kJ/KgC ×0.2 m^3/s×32^{0}C×0.5

= 3840 W

Q_{tot} = (2943+294.3+2300+3840)W (Hot water consumption is omitted)

= 9377.3W

= 9.4kW

= 9.4×24×365 kWh/a

= 82344kWh/a

∴ E = Q_{Tot}/A

$$= \frac{82344\ kWh/a}{224\ m^2}$$

= 367.6 ≈ 368kWh/m^2/a

Energy class: F ≤ 390 kWh/m²/a

According to section 6, 4/13 of the decree of Ministry of Environment, Energy requirement for single-family houses and terraced and other attached houses should be ≤ 180 kWh/m^2 and E-value should be ≤ 0.8 × 180 kWh/m^2. So, redesign is necessary to comply with the design prerequisite. Noted that the energy consumption for hot water was not considered in this calculation.

8 CONCLUSION

The strength of the materials from which the structure is made of describes the strength of a structure. Minimum material strengths are specified in standardized ways in this purpose. Structural strength depends on the care with which a structure is built, which is a reflection of the quality of the supervision and inspection.

The outcomes of this project are designed in limit state and followed Eurocode mostly. The architectural design port-folio attached in the appendix is ready to apply for the building permit in the city council.

A comparative analysis with American Standard for Building could be made on the basis of this design work in further. A cost effective analysis or redesign towards nearly zero energy building might also be good options to start from this work.

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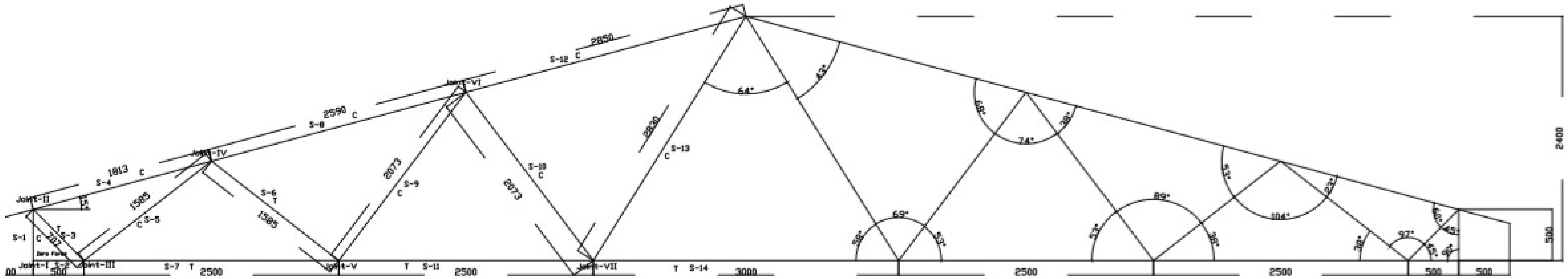
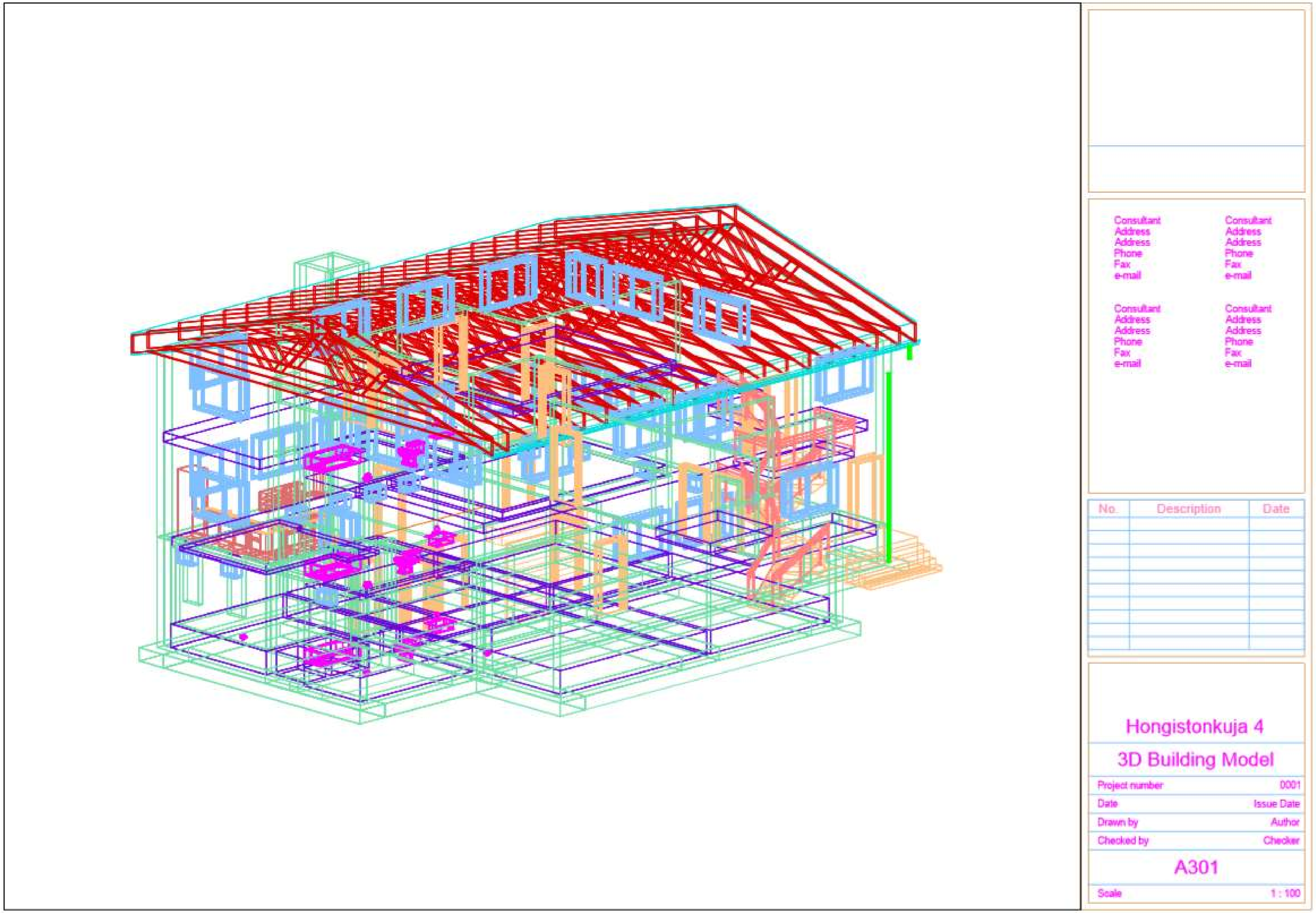
Appendix 1




Consultant
Address
Address
Phone
Fax
e-mail
Consultant
Address
Address
Phone
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e-mail
Consultant
Address
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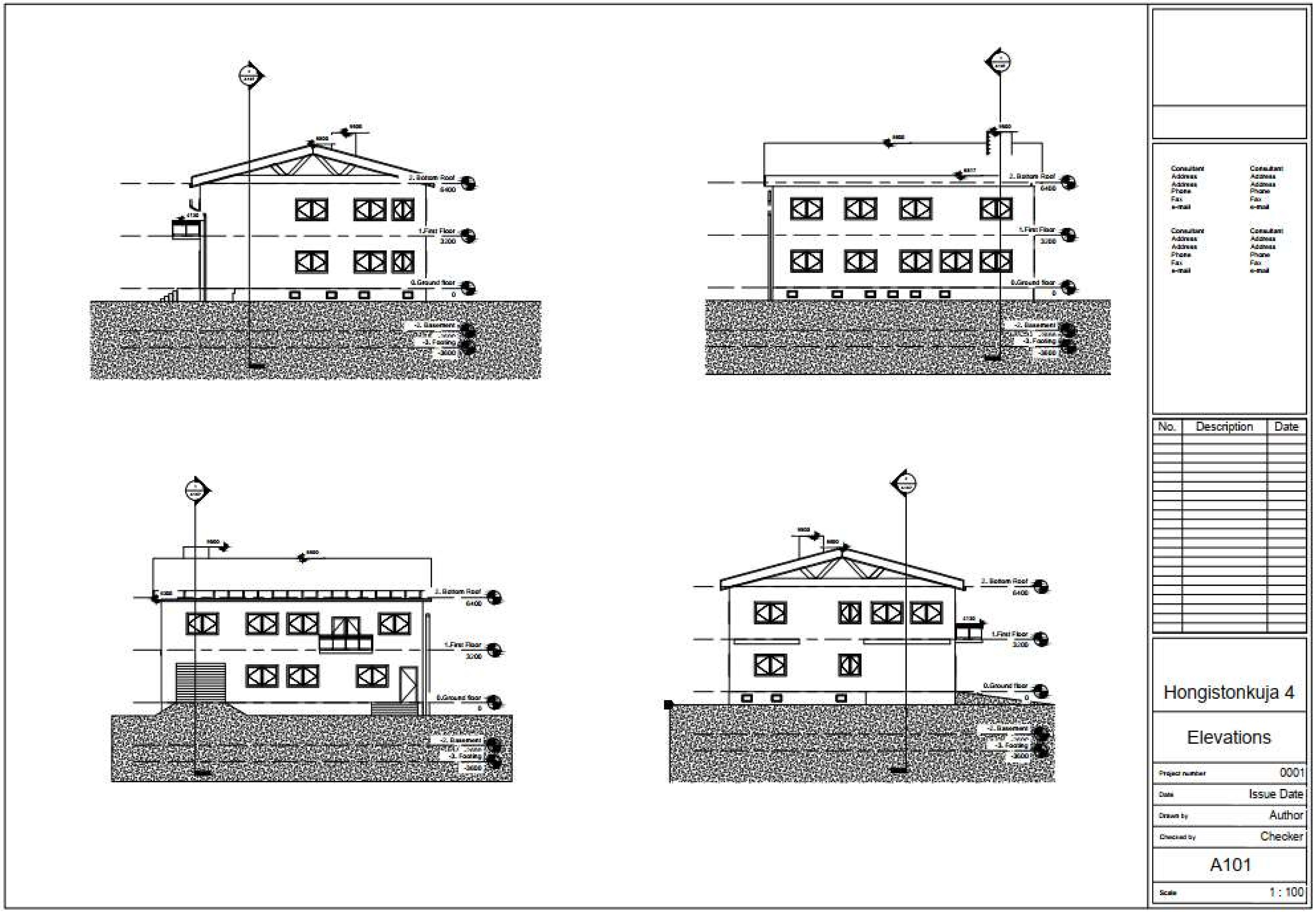
Phone
Fax
e-mail
Consultant
Address
Address
Phone
Fax
e-mail
No.
Description
Date
Hongistonkuja 4
3D Building Model
Project number
0001
Date
Issue Date
Drawn by
Author
Checked by
Checker
A301
Scale
1 : 100


2. Bottom Roof
6400
1.First Floor
3200
0.Ground floor
0
-2. Basement
-3. Footing
-3600
Consultant
Address
Address
Phone
Fax
e-mail
No.
Description
Date
Hongistonkuja 4
Elevations
Project number
0001
Date
Issue Date
Drawn by
Author
Checked by
Checker
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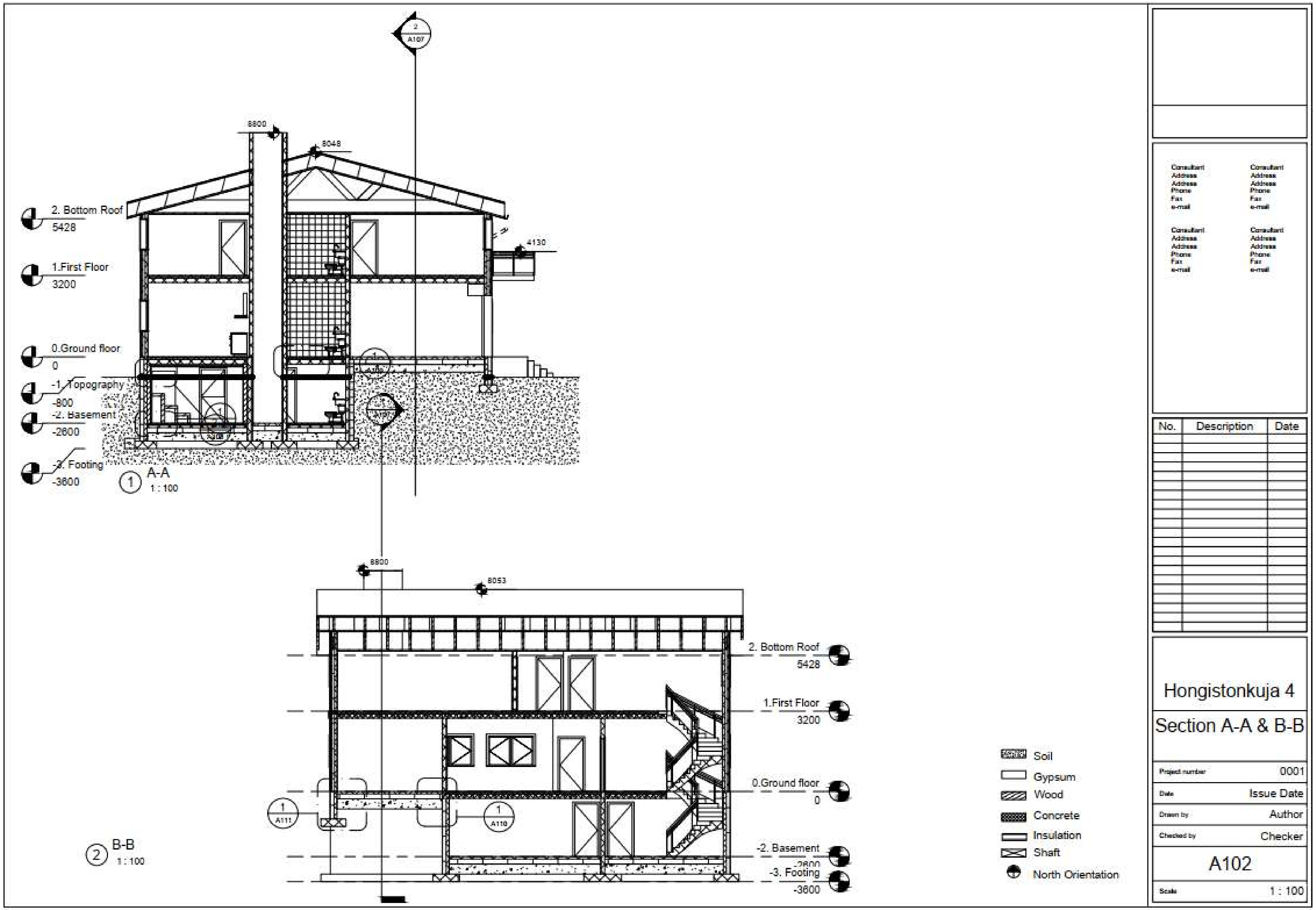
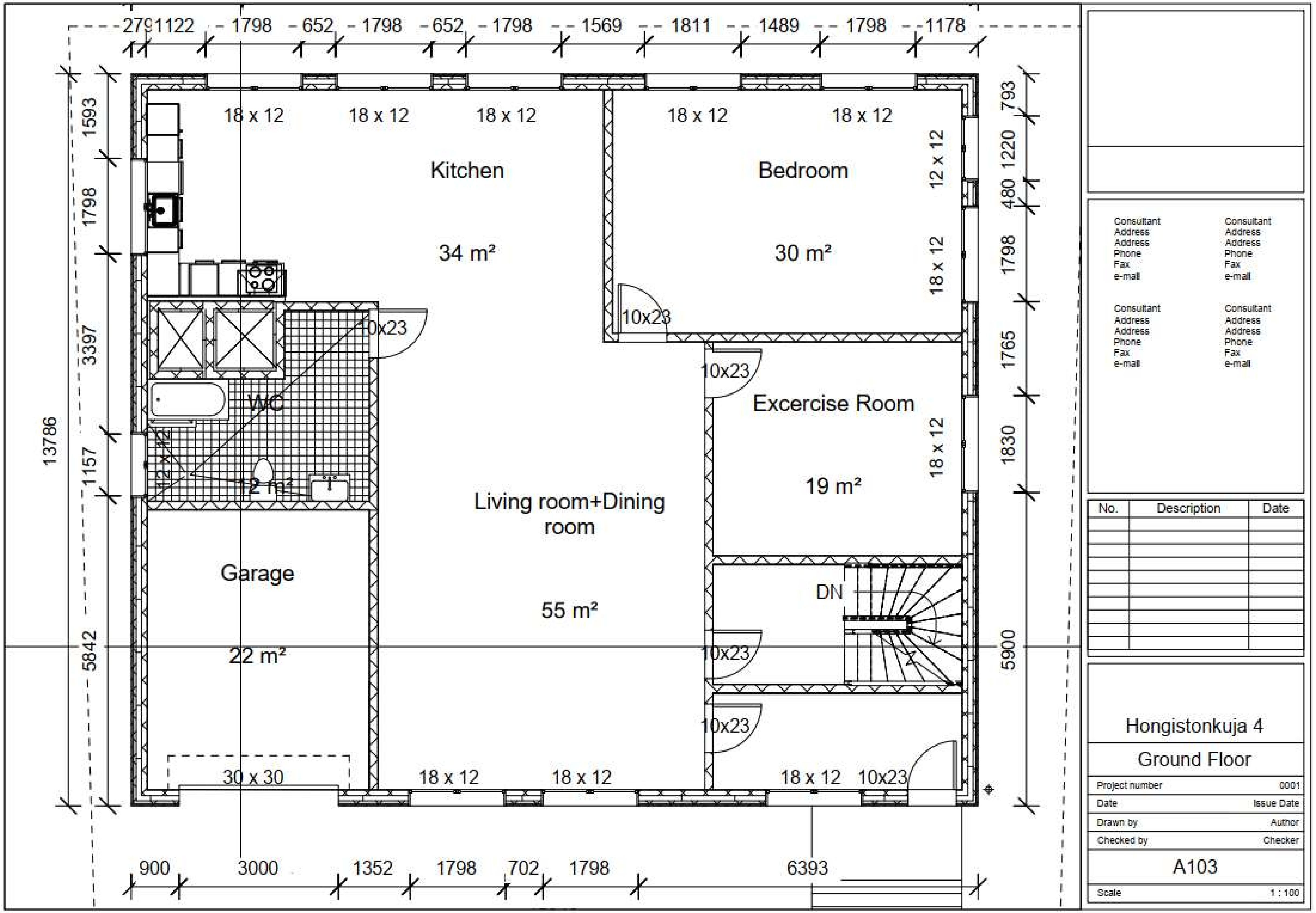
Scale
1 : 100

Appendix 2/3 (page 7)

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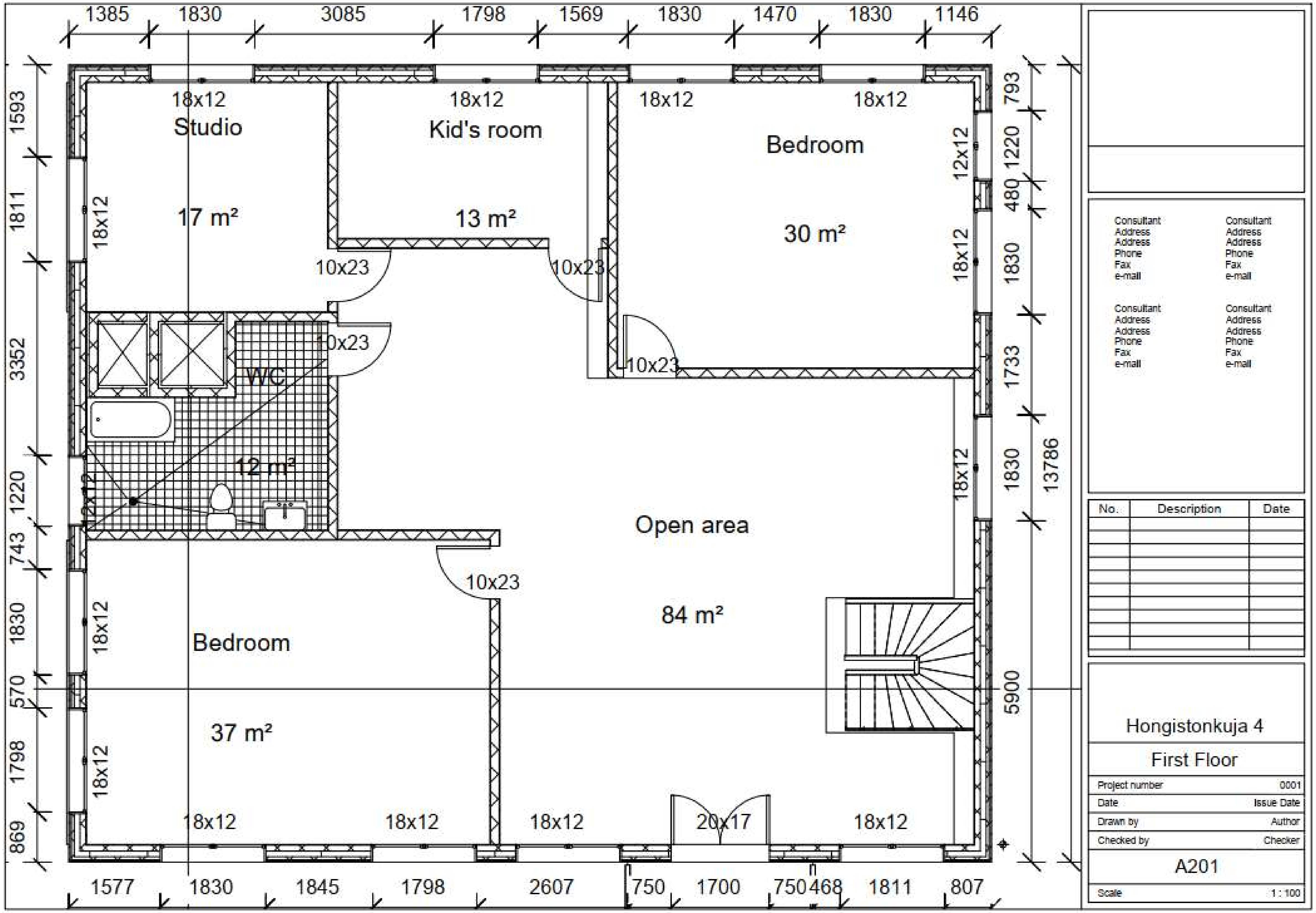
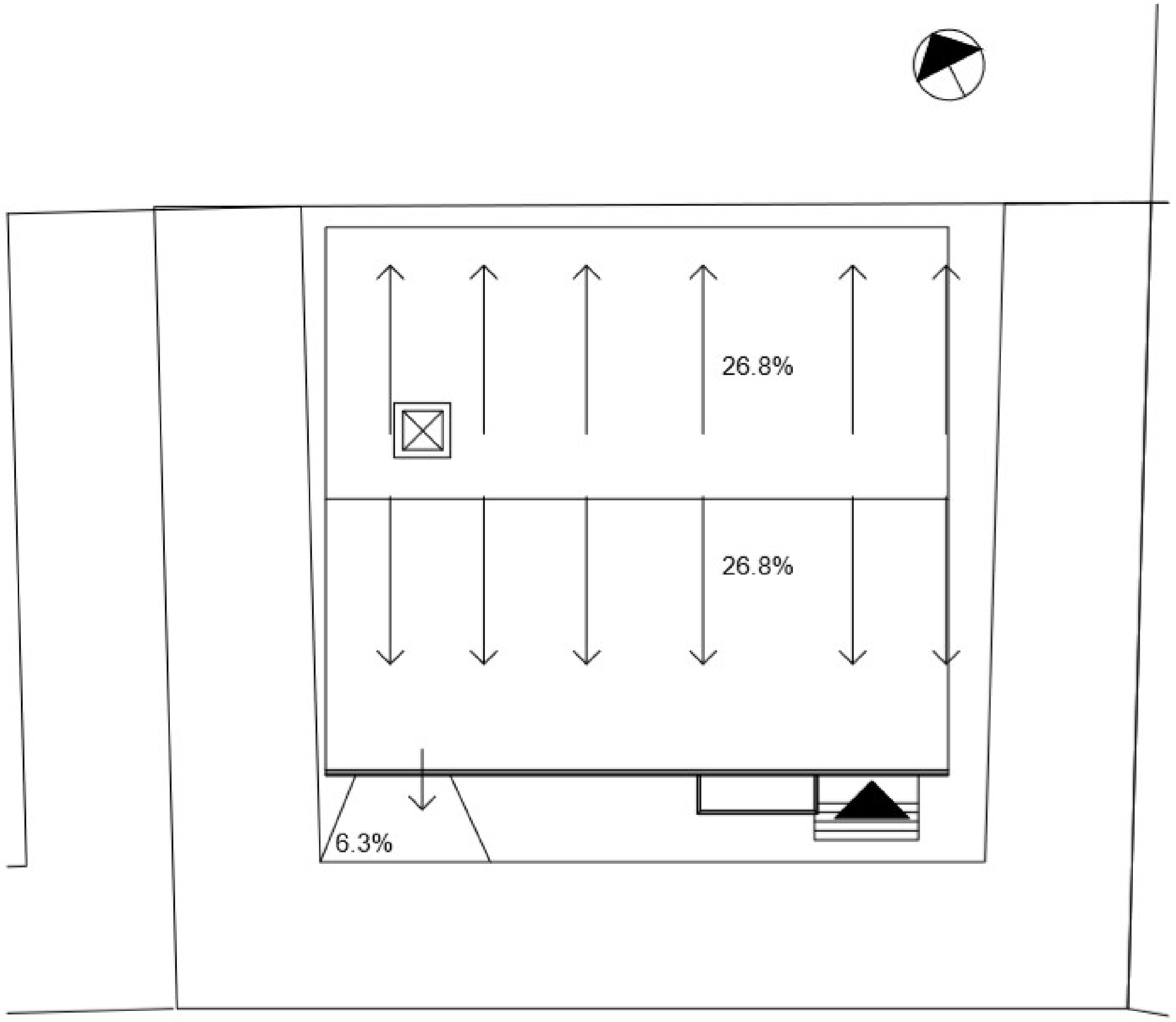
34 m²
Bedroom
30 m²
WC
12 m²
Excercise Room
19 m²
Living room+Dining room
55 m²
Garage
22 m²
DN
Hongistonkuja 4
Ground Floor
Project number 0001
Date Issue Date
Drawn by Author
Checked by Checker
A103
Scale 1 : 100
No. Description Date
Consultant
Address
Address
Phone
Fax
e-mail


Studio
17 m²
Kid's room
13 m²
Bedroom
30 m²
WC
12 m²
Open area
84 m²
Bedroom
37 m²
Hongistonkuja 4
First Floor
Project number 0001
Date Issue Date
Drawn by Author
Checked by Checker
A201
Scale 1 : 100
No. Description Date



General Information

Adress: Hongistonkuja 4,
13500 Hämeenlinna,
Finland
Use: Residential
Plot Area: 333.68 m2
Built Area 224 m2

- Entrance
- Shaft
- North Orientation

Consultant Address Address Phone Fax e-mail	Consultant Address Address Phone Fax e-mail
Consultant Address Address Phone Fax e-mail	Consultant Address Address Phone Fax e-mail

No.	Description	Date

Hongistonkuja 4

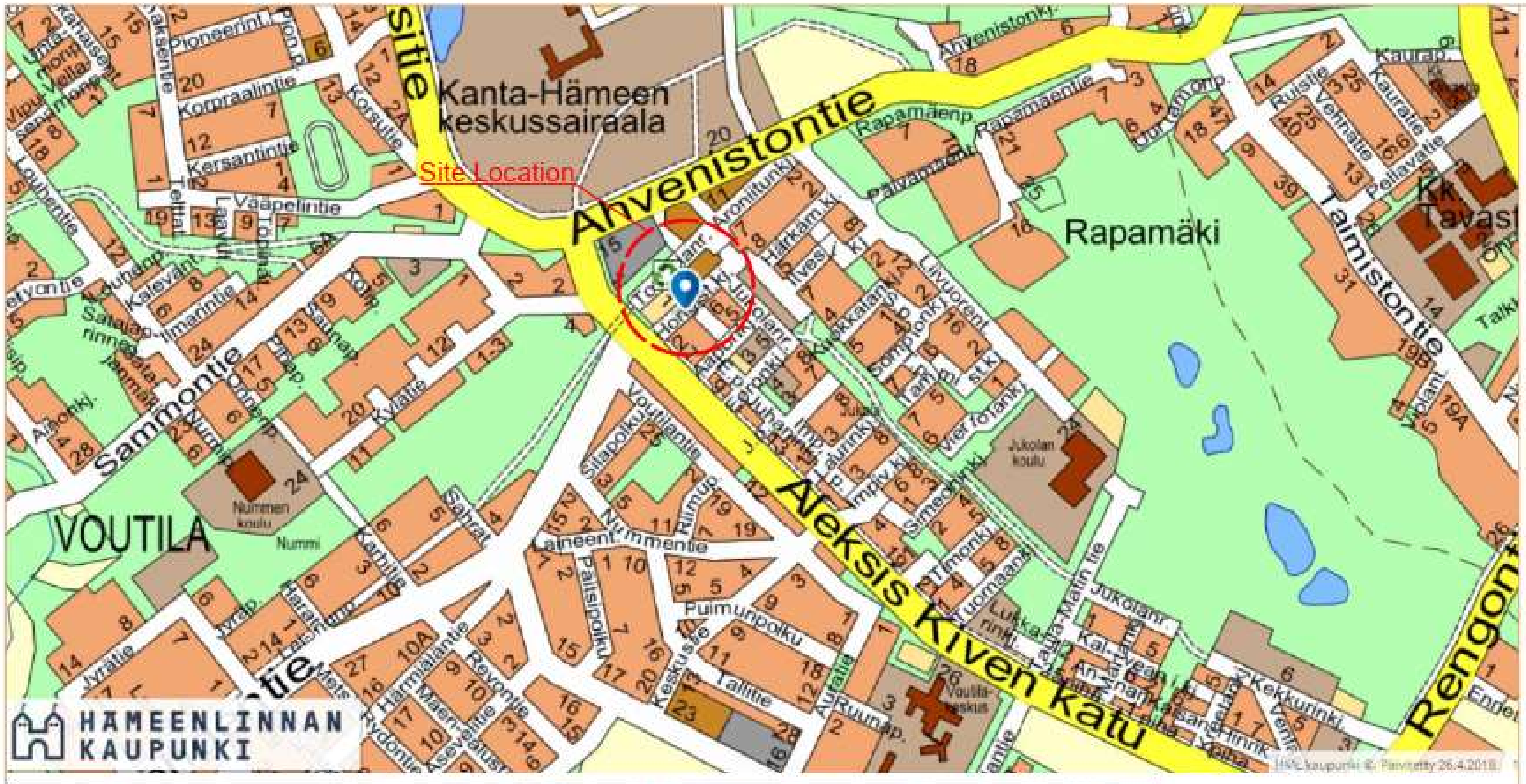
Site Plan

Project number	0001
Date	Issue Date
Drawn by	Author
Checked by	Checker

A001

Scale	1 : 100

Appendix 2/7 (page 7)



General Information

Adress: Hongistonkuja 4
13500 Hämeenlinna
Finland

Use: Residential
Plot Area: 333.68 m2

Consultant Address Address Phone Fax e-mail	Consultant Address Address Phone Fax e-mail
Consultant Address Address Phone Fax e-mail	Consultant Address Address Phone Fax e-mail

No.	Description	Date

Hongistonkuja 4

Location Map

Project number	0001
Date	Issue Date
Drawn by	Author
Checked by	Checker

A002

Scale	1 : 100