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Technical analysis of mountains wooden cabins against avalanche load according to the Norwegian standards and wood's role in circular economy

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Abstract

Today, more and more wooden cabins are being built in Norway. Most of these cabins are built in mountainous areas. One of the main challenges of mountain cabins is the avalanche hazard that causes major or minor accidents such as loss of life and values. Therefore, the stability of the mountain cabins is important in relation to such natural phenomena. The characteristics of the cabin's components are crucial to how they behave, and a greater understanding of how different parameters affect the cabin's stability is important for the development of the cabins.

This thesis is divided into two parts; a technical part that studied the analysis of Norwegian wooden cabins against avalanches and an economic part that has looked at the role of wood in circular economy.

In the technical part, this thesis focuses on horizontal stability of wooden cabins according to the Norwegian standard. The cabin was designed in the TimberTech software where the wind load was used as a horizontal load instead of avalanche load. Two different cases were investigated to reveal how change of the building's components and their properties affect the cabin's stiffening.

Analyses showed that stiffness and mass are important parameters for horizontal stiffening. Furthermore, hold down anchors are important components for sliding protection, tension and to prevent uplifting of walls. Location of shear walls is also important for increased horizontal stiffening so that the structure can take shear forces, prevent rotations and distribute the horizontal forces in all directions.

The economic part of the thesis has looked at the role of wood in circular economy. Properties of wood materials were discussed and compared to other buildings materials. Wood is a natural and renewable material that can contribute to emissions reduction. The construction industry is included as a large consumer of resources and greenhouse gas emissions and wood as building materials can be important materials for transition to circular economy.

Acknowledgment

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The thesis has taught me a lot about what possibilities and limitations wood has as a construction material and how natural phenomena such as avalanches affect the design of wooden cabins.

I am very grateful to all contributors to this thesis. I would like to thank my supervisors Prof. Roberto Tomasi (NMBU), Tor Kristian Stevik (NMBU) and Dieter Issler (NGI) who have given valuable help and good guidance. I would also to thank Peter Gauer (NGI) for his support and information during the process.

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1 Introduction

Scandinavia has a long and strong cabin-/leisure home tradition. In Norway such homes were initially built in connection with farms, for use by the farm's family members who moved to more urban areas to work. After world war II, a building boom for leisure homes occurred in Scandinavia, especially in coastal and mountainous areas and the popularity of holiday homes has increased steadily in recent decades (Farstad, Rye, & Almås, 2008).

Norway has a thousand-year tradition in building houses from timber and most of the cabins in Norway are made of wooden materials. The forest in Norway covers approximately 38% of the country's area and this gives a good supply of wood as raw material. The use of wood as a building material has both technical and economic effects. It is a renewable resource, it can be reused, has good durability, gives greater growth in the forest and has great strength (Grønvold, 10.Jan.2019).

In Norway, there are now approximately 450,000 cabins, and a large proportion are built in the mountain areas (Øye, 26.Mars.2015). A good number of these buildings are exposed to avalanche risk in various places in the country. Every winter, major or minor accidents occur as result of avalanches. The main consequences are loss of human life and physical and mental injuries to people. On average every 13 years, there is a large avalanche in the country with 10-20 deaths and 100.200 mill. NOK in material damage. Those who are usually exposed to avalanches are people who either live or work in avalanche area or those that take part in outdoor activities in mountain terrain (Kristensen, April.2003). The table 1-1 shows the number of deaths in avalanche accidents and the number of people involved in avalanche.

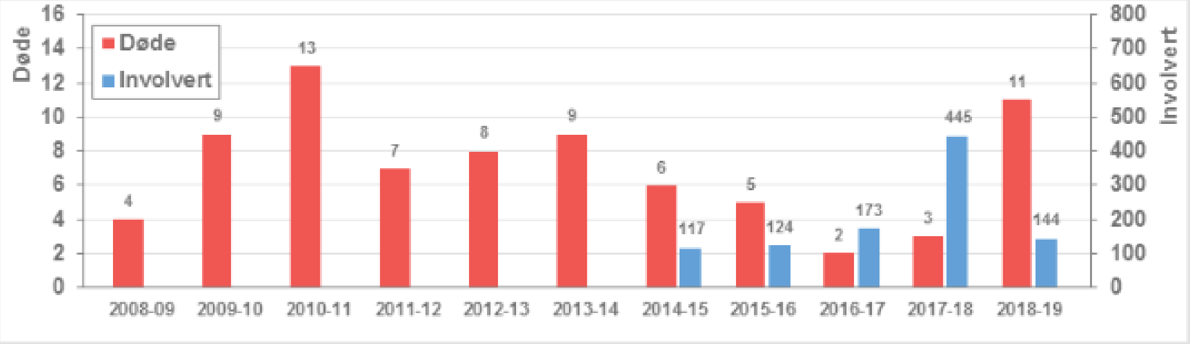


Table 1-1: The number of the deaths in avalanche accidents and the number of people involved in avalanche from autumn 2008 to today (Varsom.no, Feb.2019)

Geo hazard are natural processes that happen on earth and society must adapt to them. Norway is a country with a lot of topography and avalanches are linked to this special landscape and geological history (Norges Geologiske Undersøkelse, 14.Mars.2016). Buildings in avalanche-exposed terrain must be secured with physical security measures. These should be assessed with a risk analysis where both avalanche risk and consequence are included. Only based on such analyzes can the existing buildings be secured (Ruud, 27.Des. 2016).

1.1 Avalanche load and current research

After several tragic avalanches in the late 1960s- and early 1970s, the Norwegian Geotechnical Institute (NGI) was announced by a Norwegian parliamentary report in 1972 as a national center of expertise for avalanches that was to conduct research in this field in Norway. In 1973, NGI established a research station for avalanches on Strynefjellet in Fonnbu. Here, the connection between weather and snow cover development has been a central research field.

Full-scale trials with avalanches in Ryggfonn, near Fonnbu, is another pilot project with an avalanche path over 1000 meters height difference. Here, avalanche movement, velocity and pressure effects are measured and analyzed for development of safer methods for design against avalanches. Avalanches can be triggered by detonation of explosives. Ryggfonn is one of two such full-scale avalanches in the world. Ryggfonn has been modernized considerably, amongst other things with detonation equipment, instrumented power and velocity gauges in the avalanche path and with a 15-meter-high catch ramp at the bottom of the avalanche path (Norges Geotekniske Institutt, April.2019).

Buildings and roads exposed to avalanches have the most attention. The Planning and Building Act (PBL) has clear requirements for safety against avalanche. The statistics show that there have been few fatalities as a result of avalanches against building and road in the last few decades. Presently steep outdoor life and scooter driving constitute the largest group of fatalities in avalanches now.

The aim of the avalanche research is to increase knowledge of avalanches, thereby reducing society’s loss of values in this natural disaster. NGI cooperates closely with the Norwegian Meteorological institute, the Norwegian Water Resource and Energy Directorate (NVE) and avalanche research institutions in Switzerland and Austria.

Avalanche research at NGI is in the following areas:

- Full scale tests and measurements of avalanche in Ryggfonn
- Analysis, modeling and calculation of outlet distance and avalanche loads
- Hazard zone mapping
- Avalanche hazard warning - new methods for avalanche warning and emergency procedures
- Safety measures against avalanches, such as avalanche ramparts and screens (Norges Geotekniske Institutt, April.2019)

1.2 Research objective and problem statement

The thesis will consist of two parts:

- Technical analysis of Norwegian wooden cabins against avalanche load according to the Norwegian standard. This will be the main part of the thesis
- The second part of the thesis is the circular economy

The technical part will look at how the cabin construction system behaves when it is exposed to the horizontal load and the goal is to investigate solutions for horizontal stiffening of Norwegian wooden cabins against the avalanche load.

In order to find the answer to the problem statement, three sub-questions are derived that must be resolved in order to conclude.

1- Which components are most exposed to the horizontal load?
2- Which parameters are crucial for the building's horizontal stability?
3- What are the solutions for horizontal stiffening?

This will be done by using the TimberTech Buildings software where two cases will be studied and the effect of change in components properties and rotational rigidity on foundation will be investigated.

The economical part will look at the negative and positive aspects of wood materials as building materials in circular economy.

1.2.1 Delimitations

- It is not looked at solutions for mechanical connectors such as between columns and beams. They are assumed to be solved for the designed cabins
- The thesis only looks at the building's structural behavior and other requirements such as sound and fire requirements will not be considered
- Designed cabin is loaded by wind load corresponding to the avalanche load
- It is not looked on the impact of openings such as windows and doors

2 Literature study

2.1 Wood cabins in Norway

Cabins building and cabin life have long traditions and great scope in Norway. Cabin building is important for traveling life and tourism and can be a significant factor in maintaining settlement and strengthening the local economy. More than one third of cabins are in Hedmark, Oppland, Buskerud and Telemark and the most of them are as mountain or inland cabins (Taugbøl et al., 2001).

The materials to build a cabin is the main factor for both design and quality. The cabin stays nice for several years by use of materials that are durable, especially for cabins near the sea and in the mountains where the weather is hard and varying.

Wood is a building material that has a long tradition in the Norwegian construction industry. This material due to availability, great strength in relation to weight and ease of processing and production has been the dominant building material in Norway. It is also renewable, natural and environmentally friendly that makes it a very popular material and developed a lot in the last ten years (Kebony, 10.April.2018) (Skåren, 2012).

2.1.1 Woods Constructions

Most of Norwegians small houses, especially the cabins are built with timber construction in the form of half-timber. The development of various Engineered Wood Products (EWP) has led to properties of the wood being optimized and adopted to different uses. Glulam, Solid wood, I beam and LVL beams are the examples of EWP products. The glulam technique gives the possibilities to produce construction elements of timber in all conceivable shapes and dimensions. These construction elements can be used for floor, walls and roof in houses. The principle for structural systems of solid wood is that beams joined to elements by gluing, nailing or dowels. After changing to the building regulation in 1997 can the timber contractions be used to multi-story houses ((NTNU), 20. Feb. 2018; Skåren, 2012).

2.1.1.1 Benefit of wood materials

Currently, concrete and steel are dominant as construction for large buildings. Wood material are used mostly for smaller buildings, but the latter has interest in the wood as a building material increased in the larger buildings. There are several advantages to use wood as a building material in prefabricated buildings (Skåren, 2012).

Environment:

Wood as a renewable raw material is a natural material and has little negative impact on the environment and is provided by certified and sustainably managed forest. This applies to all forests in Norway. The wood as raw material is constantly renewed and obtained from the earth's natural cycle and after use, it can be returned to environment without any negative affecting to the environment. Therefore, the wood is one of the most environmentally friendly buildings material today.

Some important wood's environmental properties are:

- Renewable resource and a large proportion of renewable energy
- Reduce of CO_2 emission
- Provides a good indoor environment
- Easy recycling and reuse

Today it is estimated that 40% of the environmental impact in the construction industry comes from the production phase. There is very little waste production by wood factory production because the production lines are well organized, and the processes are good planned. And this results to a reduced environmental impact in term of consumption of raw material, production of waste and transport as needed (Skåren, 2012).

Energy:

Wood has a good heat-insulating property, and this reduces the influence of thermal bridge and the risk of condensation (Skåren, 2012).

***Strength and flexibility*:**

Wood as a building material, unlike most other materials is an organic and living material. It makes wood as a non-homogeneous material and a material with great variance in strength. Wood is made up of fiber that are mainly in the same direction. This direction is the longitudinal direction of the wood. There are some medullary rays in the radial direction that hold the fibers together. The size of the fibers varies with the season and growth condition. The different wood species also have large difference in density, strength and moisture content. Wood is also an orthotropic material which means is has different properties in different directions (Bell et al., 2015; Bæren, Leikåsen, & Western, 2013).

Strength and flexibility ensure the choice opportunities for the developer. Some advantages of timber structure are:

- Small manufacturing and high form stability at normal temperature and humidity intervals
- Large range of tension because of the high strength relation to own weight
- Flexible production with lower costs than the other materials
- Wood as a dry construction material can carry full load immediately after installation (Bell et al., 2015)

Maintenance:

Laminated wood does not need any surface treatment at low requirements for appearance. But the construction must be protected in some other way during construction to avoid dehumidification and soiling. Some examples for laminated wood surface treatment are stained, pained or painted.

Wood as an organic material that can be attacked by fungi and pests in some certain conditions. This has a negative impact in the life of the building but can be as one of the major advantages in ecological context. Lasting protection of the wood against rot attack can be done by pressure impregnation and constructive wood protection that means keeping the wood dry.

Fire safety:

Wood is a combustible material that through the history has led to many major fire disasters in Norwegian cities. But when it comes to coarse wood structures, such as glulam or solid wood, they hold their carrying capacity well during fire. During fire, the temperature of coarse wood structure remains below 100 ^{o} C in unburned portions of the cross section.

This is because when a glulam beams burns a coal layer is formed which insulates the heat out and prevents the air entering to new combustible material. This means that the firing will be time-dependent and will occur at a slower rate. On this basis, wood has the great ability during fire, compared with steel; wood has a better fire resistance. However steel does not have the same ability and at 500-600º C will steel lose a lot of its strength which is dependent on the quality of the steel (Bæren et al., 2013).

Glulam is one of wood constructions that has high fire resistance. One advantage of glulam is that it is possible to predict the charring of the wood and determine the wood's resistance during fire. It is also possible to treat surface of glulam to counteract heat dissipation and smoke development during fire.

Economy:

Some economic advantages of wood as building material.

- Low transport- and assembly costs because of Low specific weight.
- Prefabrication ensure shorter construction time, and this ensures:
    - Faster completion
    - Reduction of price increasing during construction
    - Early rental income

Deconstruction:

Deconstructions of buildings will be more important in the future, especially regarding to costs and environmental impact of materials that are difficult to deposit. But wood structures are advantageous since wood is biodegradable (Skåren, 2012).

2.1.1.2 Glulam

Glulam or glued laminated timber came to the Nordic countries a little later in the 1900s. The first glulam constructions were imported to Norway in 1916, this was done by Dr. Guttorm Brekke. Brekke acquired the rights for production and sale of glulam structure to Norway, Sweden and United States in 1916 and learned techniques and production methods in Hetzer before starting his own factory in Østfold. There are about ten glulam factories in the Nordic countries. The standard EN 14080 is published for the first time in 2005 and revised in 2013. The standard regulates the general requirements for glulam so that the manufacturers to be able CE-mark their products. Most of the glulam that used in the Nordic Countries goes to industrial buildings, schools and residential buildings, including multi-story buildings. Totally, this covers 60 % of consumption.

Modern bonding technique in combination with good strength properties of wood material make the glulam to a highly qualified construction material with unique properties (Bell et al., 2015).

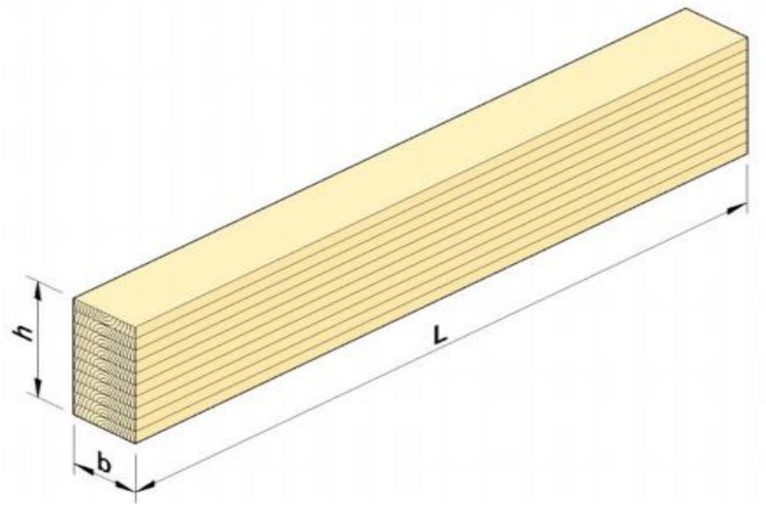


Figure 2-1: Glulam (Rønquist, 2018)

Glulam is a processed wood product. Vertical columns or horizontal beams can be produced in varying shape and size by laminating several smaller pieces of lumber of strength-sorted wood. An example of glulam is shown in figure 2-1.

Strength and stiffness are excellent properties of glulam that in relation to weight are better than steel. Glulam structures is technically possible to design with a span up to 150 meters. Glulam with good formability, variable cross section is often a first choice in projects with high architectural value.

Today, glulam is used for virtually all types of load-bearing structures and because of good protection against climate stresses there is a few limitations against using of glulam.

Glulam has essentially the same strength properties as ordinary constructions timber:

- The strength varies with the angle between force and fiber direction
- The strength decreases with increasing moisture content
- The strength decreased with increasing load duration

The glulam component has higher strength and less dispersion in the strength properties in compared with a corresponding component of timber construction as shown in the figure 2-2 (Bell et al., 2015).

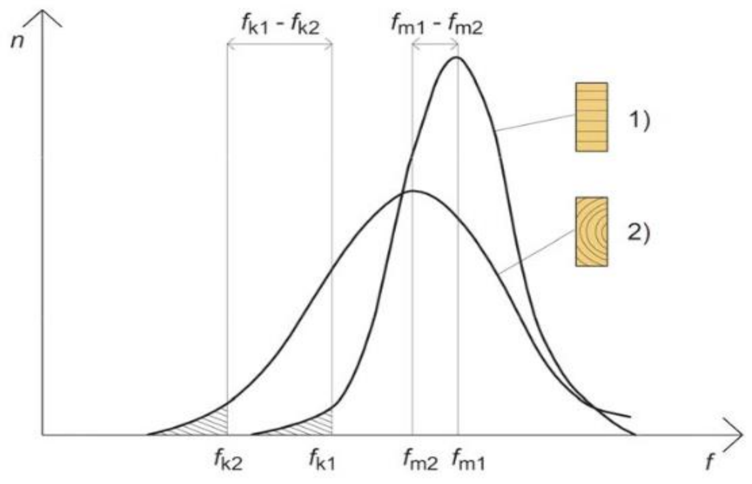


Figure 2-2: Strength properties of glulam (1), compared with timber construction component (2) (Bell et al., 2015)

In order to get the desired properties in glulam, it is important that the production takes place correctly, such that the finished beam or column gets the strength it should have. When the wood comes from sawmills and is strength-sorted, it must be dried before further treatment to obtain good strength in glue lines. In addition, it is important that the moisture in the glulam should be fairly the same as in the finished construction in order to avoid large cracks. The slats should have a humidity of 8-15 % when they glued together (Bell et al., 2015).

The development of glulam has been exceptional for the past 30 years and has led to the possibility of getting bigger spans and better nodes. This is a result of several national and Nordic research projects. It has made also possible to construct large buildings such as the Viking ship and support-system of glulam in Gardermoen. Glulam can also be used as follows:

- Support system of straight and curved beams with large spans
- Straight beam that can be used as rafters or columns
- Roof beams with straight or curved underside
- Floor joist
- Frame structures
- Reinforcement in other load-bearing elements

Due to the properties of the wood and modern bonding techniques, glulam has become a construction material with unique properties. It is characterized by fast and easy assembly and can be loaded immediately after installation (Bæren et al., 2013).

2.1.1.3 Solid wood

Solid wood has created new opportunities for wood as a building material. The need for new rational and environmentally efficient constructions solutions has led to development of building with solid wood elements. The development began early in 1990s in central Europe and spread to the Nordic countries.

Today construction with solid wood elements is a recognized building method that used in residential building, multi-story houses, commercial buildings and schools (Jarle Aarstad, Geir Glasø & Aasmund Bunkholt, 01.Aug.2011).

Solid wood elements are slats assembled by use of nails, screws, dowels, glue or steel rods. Both the thickness of the element and the number of slats is dependent on the function and application of the element. The solid wood elements are divided into the following main categories which are shown in figures 2-3 and 2-4.

Edged elements: is a common term for elements that are assembled by standing slats and these elements are connected by screws, nails, dowels and steel rod.

Multilayer elements: is a common term for elements that are composed of slats in different layers. The layers are 90 or 45 degrees in relation to each other and the connective in these elements is glue or dowels (Jarle Aarstad og Geir Glasø & Aasmund Bunkholt, 01.Aug.2011).

Figure 2-4: Edged solid wood (Hegle, 2018)

Figure 2-3: Multilayer solid wood (Hegle, 2018)

Wood is much stronger parallel to the grain than perpendicular to the grain, therefore the layers in the element are placed perpendicular to each other. This cross-type is often called Cross-laminated Timber (CLT) and gives the element equal strength in both directions.

CLT often has three, seven or nine layers of wood and it is usual that the outer layers have higher strength than the inmost layers. Other types of solid material are Laminated Veneer Lumber (LVL) and Laminated Strand Lumber (LSL). LVL elements are made of thin layers wood veneer with a structural glue and it becomes an intermediate between glulam and plywood. LSL elements are composite materials which made of wood shavings mixed with glue and pressed together parallel to the longitudinal direction of the elements (Hegle, 2018; Skullestad, 2016).

Solid wood elements can be used as load-bearing elements or non-loadbearing in floors, walls and roofs. The entire bearing system can be built up of solid wood elements or in combination with other materials. Solid wood elements as building materials have several advantages:

- Great flexibility in design, plan and construction
- Short build time and good total economy
- Easy to combine with other material
- Low weight and easy technical installation
- Good work environment and tidy workplace
- Positive environmental properties
- Good raw material and wood properties utilization (Jarle Aarstad and Geir Glasø & Aasmund Bunkholt, 01.Aug.2011)

The surface of the solid wood elements may be visible and need no cladding. But in some cases, due to fire/sound insulation requirements, it may be necessary to dress this. Support system of solid wood can satisfy fire requirement up to RE90.

The mounting of solid wood elements is stated to take less time than concrete elements. The low weight of the wood makes transport and mounting more efficient. The elements are joined together with screws. If the elements must be stored on the construction site, they must be protected against dirt and moisture. Solid wood is not suitable for direct contact with water or soil. The wood will move during moisture stress and it will be necessary to take into account shrinkage and expansion (Finstad, 2014).

2.1.2 Geometric forms of Norwegian cabins

Different forms of leisure cabins make different demands depending on the location thus provides different consequences for the environment and raises various challenges for local communities and municipalities. Thus, the municipalities should themselves treat the various forms of leisure cabin in relation to the various challenges they create. It applies both to the planning of new development and to the management and further development of existing areas. Cabin types in Norway divide as following:

Hunting and fishing cabins are simple cabins that are characterized by:

- Small cabins, often one bedroom
- Completely simple standard
- Can lie together or spread
- Location and use traditionally linked to hunting and fishing

Hunting and fishing cabins that are located in the high mountains or by the water are often not perceived as conflict -filled, because they are experienced as traditional buildings with place-adapted and traditional design with limit size and standard.

Traditional leisure cabins can be characterized by:

- Limited size, typical size 40-80 m^2
- Limited technical standard often not include water, simple toilet facility.
- No electricity
- Do not drive access all the way
- Used during holiday and weekend

High standard cabins can be characterized by:

- Typical size from 80 m^2 and upward
- Include water, drains and electricity
- Drive access all the way
- Can stay for longer period and be counted as house number 2
- Can use as rental cabins

Holiday homes or leisure cabins can be characterized by:

- Size as small house and upward
- Standard, installation and access as house
- Use during holidays and weekends
- Several can be inhabited for longer periods and be counted as house number 2
- Some can be used as permanent home, and some can be used as rental unit (Bjørnøy, Nov.2005)

2.1.2.1 Terrain

The choice of landscape type for development areas provides different challenges for the terrain adaption in next step. In steep and rough terrain, knowledge and experience will be required to place cabins well in the terrain, beyond that can be solved in the regulation plan. The slope of the terrain is crucial if the area is suitable for development. Higher level of utilization of the planning provisions require a smoother terrain. The slope is measured primarily on land and land areas but also as average measurements over slightly large areas.

Close to nature and cultural landscape is a central reason for building the cabin. But at the same time, the settlement becomes an intervention in these values. Here are some general principles that should take into account when building of cabins:

- Pay attention to landscape, vegetation and settlement in the surrounding
- Avoid cabin building in areas with great nature values, valuable cultural environment and in open landscapes
- Follow and reinforce the local tradition of placement in the terrain, where it exists
- Build tight and low
- Emphasis on quality and overall design (Bjørnøy, Nov.2005)

2.1.2.2 Design of the cabins

Cabins areas will be best if common frameworks be created for volumes, use of materials and color. Today there is an increasing number of cabins and prefabricated solutions that do not fit the land or the environment where they are placed.

The possibilities for good terrain adaptation are closely linked to the size of the cabins. Several small cabins can be easier to adapt to the terrain than one large cabin. Cabin groups should have uniform design. Grouping of cabins provides extra qualities between the cabins (shielding, local climate and good outdoor spaces).

The dimensioning of materials should be adapted to the size and use of the buildings. Traditionally, leisure cabins have few and small windows and doors. In cabin areas in forests and mountains, the use of materials should be adapted to the nature where it is located. The natural materials can be wood, stone, peat etc. (Bjørnøy, Nov.2005).

In the case of color used on cabins, these must be avoided:

- Use of sharp color and glossy surface
- Great contrast, e.g. between windshields and wall

Preferably, it is better to use medium to dark soil colors for cabins in forests and mountains (Bjørnøy, Nov.2005).

Cabins in forests and mountains have traditional pitched roof, and this should be used if there is nothing special to take into account. Roof angle and eaves have regional variations and local building tradition can be exemplary. Material's choice of roof is also important because of the cabin's appearance (Bjørnøy, Nov.2005).

Here are some rules and laws that need to be taken into account in building of cabins:

- Buildings should be placed as shown in the plan map
- Situation plan for planned buildings must follow the buildings application
- The settlement of the cabins must have a uniform and harmonious design
- The color of the facades and the roof must be approved by the permanent committee for planning matter
- The cabins must have pitched roof
- Buildings in the cabins area must be designed in line with local building customs (Bjørnøy, Nov.2005)

2.2 Design methods of timber structure

It is necessary to analyses the structure and build up a fit design model before starting of formal calculation. Design rules for load-bearing building structures should primarily ensure the risk of breakdown of the construction and secondly, ensure that the building works satisfactory during normal use.

Timber constructions must be designed with respect to the requirement of the material's strength, behavior and durability. Design is based on verification and the verification can be demonstrated by calculation or testing or by a combination of these methods. The purpose of the verification is to show that the relevant requirement is satisfied for selected system, dimensions and materials.

Design life for the buildings is often recommended 50 years. Design life is the time that the construction will work in relation to intended function and normal maintenance without the need for significant repairs (Bell et al., 2015).

2.2.1 Eurocodes

The Eurocodes have been prepared on behalf of the European commission. The purpose of the Eurocodes is to provide harmonized technical provisions for the design of structures to replace the national rules of the European member countries. The construction-related Eurocodes consist of 10 standards that each normally consists of several parts. The design and detailing of timber structures is performed according to Euro-codes 0, 1 and 5. Eurocode 0 (EN 1990) is basis for design of structures, Euro code 1 (EN 1991) is loading of structure and Eurocode 5 (EN 1995) is for designing of timber construction.

In order to verify a construction or a part of it must a designer be able to distinguish between design situations as well as impact, loads, loads combinations and their effects. The table 2-1 shows design situations and their verification needs.

Design situation		***Verification***
Persistent	Normal use	ULS, SLS
Transient	Assembly, Random situation that the constructions is exposed to maintenance and repairs.	ULS, SLS
Accident	Normal use	ULS
	Assembly	ULS
Seismic	Normal use	ULS, SLS
	Assembly	ULS, SLS

Table 2-1: Design situations and their verification needs (Bell et al., 2015)

Eurocodes are based on the limit state design. A limit state design defines a condition that the construction cannot exceeded if it will satisfy the relevant performance requirement. These conditions are classified as ultimate limit state (ULS) and serviceability limit state (SLS). Ultimate limit state is related to security against breakdown and other form of construction failure. And the serviceability limit state is related to conditions that the construction is standing but it's no longer satisfied defined usage requirements, like deflections and vibrations (Bell et al., 2015; TEMTIS, May.2008).

2.2.1.1 Partial factors

Concept of characteristic values is an important basis for the partial factor method. The values should be based on clear statistical definition as characteristic loads as defined in a set of Eurocode 1. Design loads are found by multiplying the characteristic load with a partial safety factor. The value of a partial safety factor is depending on the definition of the characteristic value that it will be used with. The table 2-2 shows values of partial safety factors for permanent and variable actions.

For time dependents load as snow and wind will the characteristic value Q_K typically be defined by the fact that the probability that the value is exceeded is 2 % per year. It's means that the load level Q_K exceeded on average, only once during a 50 years period (Bell et al., 2015; TEMTIS, May.2008).

Action	**Permanent**	**Variable**
Favorable	$\gamma_G = 1$	$\gamma_Q = 0$
Unfavorable	$\gamma_G = 1{,}35$	$\gamma_Q = 1{,}5$

Table 2-2: Partial safety factors for actions (Porteous & Kermani, 2013)

2.2.1.2 Load effect and load combinations

Load effect includes the internal forces, moments and displacements that caused by loads. As a basic rule, a construction is not dimensioned for a single load but is dimensioned for a combination of loads, for example self-weight and snow load. To determine a design combination of loads must each last be considered as the dominant variable last and combined with the other loads with its combination values. These combination values describe a reduction of the characteristic value Q_K via the factors Ψ_0, Ψ_1 and Ψ_2. Appendix A shows the values of these factors for different categories.

The combination value ($\Psi_0 Q_K$) is used to verify the ultimate limit state and serviceability limit state when it is appropriate to check for the characteristic load combination. The frequently occurring value ($\Psi_1 Q_K$) is used for the verification of ultimate limit state related to accident loads and for verification of serviceability limit state. The quasi-permanent value ($\Psi_2 Q_K$) is used to judge long-term effects in the serviceability limit state, e.g. offsets or cracks. These rules define how dead and live loads should be combined to determine the load effect. The following equation is valid for design in sustained or transient situations in ultimate limit state (Bell et al., 2015; TEMTIS, May.2008).

$$E_d = \sum_{j \geq 1} \Upsilon_{G,j} * G_{k,j} + \Upsilon_{Q,1} * Q_{k,1} \sum_{i>1} \Upsilon_{Q,i} * \Psi_{0,i} * Q_{k,1} \tag{2.1}$$

Index (j) indicates permanent load number and index (i) indicates variable load number. $Q_{k,1}$ is characteristic value of the dominant variable load. $\Upsilon_{Q,1}$ is the partial safety factor associated with $Q_{k,1}$ (Bell et al., 2015).

2.2.1.3 Strength classes

The mechanical properties of wood vary with moisture content. Increasing of the wood moisture gives lower values for both strength and E-module. This is because when the cell wall swells there will be less cell wall material per unit area. The most important thing is that when water enters the cell wall, the bonds that holding the microfibrils together become worse. Humidity above the fiber saturation point has no importance on the mechanical properties because there are only variations in the amount of free water in the cell cavity. The different strengths do not change the same when the wood moisture changes.

For example, increased humidity will have a greater impact on the compressive strength than on the tensile strength parallel to the grain, because it is the buckling of each wood cell that leads to pressure failure (Forening).

The wood is placed in different strength classes with clearly define values for the mechanical properties of the result of visual or mechanical sorting. All wood regardless of origin, can therefore be used for load-bearing structure if matching standards of sorting and design are followed.

In Norway, NS 3470 is the current standard for design and NS INSTA 142 the current standard for visual sorting. In the European common standards, the Eurocode 5 (EC5) are the current rules for the design of timber structure. Strength classes of glulam and solid wood are shown in appendix B.

2.2.1.4 Service classes

The moisture content and its variation have a significant impact on all properties of wood. The moisture content affects both stiffness and strength of wood. Eurocode 5 introduces three service classes of moisture effect at design. The table 2-3 shows service classes of wood materials in relation to moisture content and environmental conditions.

Service class 1 characterized the moisture content that occurs in the wood when the surrounding air has a temperature at 20^o C and relative humidity exceeding 65 % only for a few weeks. Service class 2 characterized the moisture content that occurs in the wood when the surrounding air has a temperature at 20^o C and a relative humidity that exceeding 85 % only at a few weeks. Service class 3 characterized by climate condition that has a higher moisture content than in service class 2 (Bell et al., 2015; Porteous & Kermani, 2013).

Service Class	Average moisture content U_m	Environmental conditions
1	$U \leq 12\%$	20^o C und 65% rel. humidity
2	$U \leq 20\%$	20^o C und 85% rel. humidity
3	$U > 20\%$	Higher humidity compared to service class 2.

Table 2-3: Service classes in relation to moisture content and environmental conditions (Porteous & Kermani, 2013)

2.2.1.5 Load duration classes

Wood has a lower strength when it is loaded over a long period of time. Therefore Eurocode 5 defines five load duration classes as shown in the table 2-4 which is necessary in the design of timber construction.

The impact of the load duration to woods strength is determined by the value of a modification factor K_{mod}. This factor K_{mod} is the function of load duration classes and service classes. The factor is a reduction factor for the characteristic strengths of wood and it varies between 0,2 and 1,1. The variation of this factor for solid wood and glulam is between 0,5 and 1,1. Table 2-4 shows values of K_{mod} factors for solid wood and glulam (Porteous & Kermani, 2013).

Material	Standard	Service class	Load-duration class				
			Permanent action	Long term action	Medium term action	Short term action	Instantaneous action
Solid timber	EN 14081-1	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90
Glued laminated timber	EN 14080	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90

Table 2-4: Values of K_{mod} (Porteous & Kermani, 2013)

2.2.2 Serviceability limit state (SLS)

Serviceability limit state design needs to be done to ensure that the designed structure is comfortable enough for human use. It should only perform the SLS design calculation after the structure has passed the ULS design calculation. Therefore, the structure capacity should be checked first before checking of the comfort.

Generally, the output of the SLS design calculation is a deflection control, a cracking control etc. which are important to ensure that the designed structure will not interfere with deflection or crack. The rules for how the construction can be deflected or cracked are generally determined on the building code which can vary from country to country. The country's building code should be checked to find out the limit for each of these controls (Andrew Sugianto, 11.Mars.2017).

2.2.2.1 Deflection

Design in the serviceability limit state (SLS) mainly means that the cross-sectional size and strength class should be chosen so that local deformations do not exceed the limit values stated in Eurocode EN 1995-1-1. The figure 2-5 shows the components of deflection resulting from load combinations.

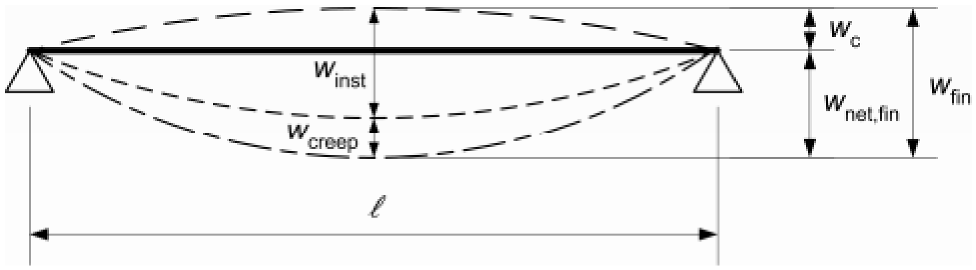


Figure 2-5: Component of deflections (TEMTIS, May.2008)

The net deflection $W_{net,fin}$ can be calculated by the following equations:

$$w_{net,fin} = w_{inst} + w_{creep} - w_c = w_{fin} - w_c \tag{2.2}$$

$$w_{fin} = w_{inst} * (1 + k_{def}) \tag{2.3}$$

$$w_{inst} = w_G * \textstyle\sum_{i\geq 1} \psi_{2,i} * w_{Q,i} \tag{2.4}$$

$$w_{creep} = k_{def} * w_{inst} \tag{2.5}$$

Where w_{inst} is the instantaneous deflection, w_{creep} is the creep deflection, w_{fin} is the final deflection, w_c is the pre-camber (if applied), k_{def} is the coefficient of deformation and $\Psi_{i,2}$ is the factor for quasi-permanent value of live loads. The table 3.1 shows some examples of recommended range of limiting value for deflections of beam with span L (TEMTIS, May.2008).

	w_{inst}	$w_{net,fin}$	w_{fin}
Beam on two supports	L/300 to L/500	L/250 to L/355	L/150 to L/300
Cantilevering beams	L/150 to L/250	L/125 to L/175	L/75 to L/150

Table 2-5: Examples of limiting values for deflections of beams (TEMTIS, May.2008)

2.2.3 Ultimate limit state (ULS)

The ultimate limit state is the design for the safety of a structure and its users by limiting the stress that materials experience i.e. the calculation that must be done to ensure that the structure being built is stable and strong enough against any loads such as dead loads, live loads, snow loads, earthquake loads, wind loads, their combination and so on. In order to meet engineering requirements for strength and stability during design loads, ULS must be fulfilled as an established state.

The ULS as a purely elastic state is usually located at the upper part of its elastic zone. This contrasts with the ultimate state which involves excessive deformations approaching structural breakdown and lies deep within the plastic zone. A structure will only satisfy the ULS criterion if all factors as bending, shear, tensile and compressive stresses are below the calculated resistances. Safety and reliability can be assumed if these criteria are fulfilled (Andrew Sugianto, 11.Mars.2017).

2.2.3.1 Beam and Column systems

In its simplest and most common form, a carrier system consists of a freely arranged beam on two columns. A column as a building element transfer weight from an overlying structure to the ground or an underlying structure. Under designing of columns, it must be checked that they satisfy specified requirements for bending, shear, axial forces (compression and tension), buckling and their combination.

Beams are normally straight wood components with rectangular cross-section. They are used as floor beams, ceiling beams, purlins, in bridges etc. Under designing of beams, it must be checked that they have enough capacity for bending and shear and they satisfy specified requirement for deflections and vibrations. For laying beams, it must be checked that the pressure perpendicular to the grain do not exceed design strength. The length of the beam is often decisive for which requirements become dimensioning. Usually bending becomes critical for medium-long spans, while shear can be dimensioned for shorter spans (Bell et al., 2015). The figure 2-6 shows to different beam-column system.

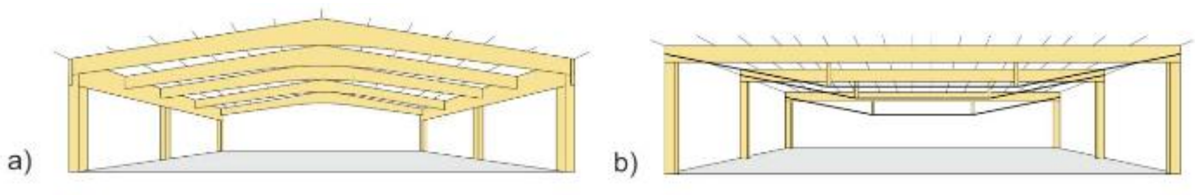


Figure 2-6: Beam-column systems, a) Boomerang beam, b) Straight beam (Bell et al., 2015)

2.2.3.2 Bending

According to technical beam theory, the normal tension in a solid rectangular cross section caused by a bending moment M about the strong (y-y) axis that given by:

$$M_{Ed} = \frac{q_d * L^2}{8} \tag{2.6}$$

And resistance moment be calculated by the following equation:

$$W_y = \frac{b \cdot h^2}{6} \tag{2.7}$$

Where q_d is the effected load, L is the length of the beam, b is the cross-sectional width and h is the cross-sectional height.

The maximum design bending stress can be calculated by the following equation:

$$\sigma_{m,y,d} = \frac{M}{W_y} \tag{2.8}$$

And design bending strength is given by the following equation:

$$f_{m,d} = \frac{f_{m,k} * k_{mod} * k_{sys} * k_h}{\Upsilon_M} \tag{2.9}$$

Where $f_{m,k}$ is the characteristic bending strength, k_{mod} is the strength factor, Υ_M is the partial factor for the material properties, k_{sys} is the system-strength factor and k_h is height factor.

If a component is affected by bending about both axes (strong and weak) at the same time must the design criteria be satisfied by:

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{2.10}$$

$$k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{2.11}$$

Where $\sigma_{m,y,d}$ and $\sigma_{m,z,d}$ are design bending stresses about the main axes, while $f_{m,y,d}$ and $f_{m,z,d}$ are the corresponding design bending strengths. K_m is the modification factor that consider the redistribution of stresses. For rectangular cross-sections of glulam and structural timber are K_m = 0,7, and for all other cross-sections are k_m=1 (Porteous & Kermani, 2013; TEMTIS, May.2008).

2.2.3.3 Shear

All components that exposed to bending will normally also have shear stresses parallel to the longitudinal axis of the component. These stresses have their greatest value at the natural axis of the cross section, while they are equal to zero at the top and bottom of the cross-section. The maximum shear stress (τ) for rectangular cross sections is given by:

$$\tau_d = \frac{3 * V_{Ed}}{2 * b * h} \tag{2.12}$$

Eurocode 5 recommended to use an effective width b_{ef} instead of width b. It is due to cracking as a result of moisture-induced stresses (drying and swelling). b_{ef} is less than b.

$$b_{ef} = k_{cr} * b \tag{2.13}$$

Eurocode 5 suggests the value of crack factor k_{cr} equal to 0,67, but this is a value that can be determined nationally. In Norway a different value is used for crack factor. For glulam and structural timber has been decided to set k_{cr}=0,8 and for other wood-based products, k_{cr}=1.

Design value of the shear force calculated by:

$$v_{Ed} = \frac{q_d * L}{2} \tag{2.14}$$

Where q_d is the effected load, L is the length of the beam, b is the cross-sectional width and h is the cross-sectional height.

Design shear strength is given by the following equation, where $f_{v,k}$ is the characteristic value of shear strength.

$$f_{vd} = \frac{k_{mod} * f_{vk}}{\gamma_m} \tag{2.15}$$

And the design criteria for shear shall be satisfied by: (Norges Standardiseringsforbund, 1995).

$$\frac{\tau_d}{f_{vd}} \leq 1 \tag{2.16}$$

2.2.3.4 Tension

The following expression shall be satisfied by tension parallel to the grain of timber:

$$\sigma_{t,0,d} \leq f_{t,0,d} \tag{2.17}$$

Where $\sigma_{t,0,d}$ is the design tensile stress along the grain and $f_{t,0.d}$ is the design tensile strength along the grain.

The design of tensile stress is given by the following equation:

$$\sigma_{t,0,d} = \frac{F_{t,0,d}}{A_{ef}} \tag{2.18}$$

And the design value of tensile strength is given by the following equation:

$$f_{t,0,d} = \frac{f_{t,0,k} * k_{mod} * k_h}{\gamma_m} \tag{2.19}$$

Where $F_{t,0,d}$ is the design tensile load along the grain , $f_{t,0,k}$ is the characteristic tensile stress along the grain and A_{ef} is the effective contact area as calculated as:

$$A_{ef} = L_{ef} * b \tag{2.20}$$

For tensile perpendicular to the grain, the following expression shall be satisfied: (Norges Standardiseringsforbund, 1995).

$$\sigma_{t,90,d} \leq f_{t,90,d} \tag{2.21}$$

2.2.3.5 Compression

Design for pressure perpendicular to the grain of the timber shall satisfy the following requirements:

$$\sigma_{c,90,d} \leq k_{c,90} f_{c,90.d} \tag{2.21}$$

Where the design compressive stress perpendicular to the grain is given by the following equation:

$$\sigma_{c,90,d} = \frac{F_{c,90.d}}{A_{ef}} \tag{2.22}$$

And the design compressive strength perpendicular to the grain is given by the following equation:

$$f_{c,90,d} = \frac{f_{c,90,k} * k_{mod} * k_h}{\gamma_m} \tag{2.23}$$

Where $F_{c,90,d}$ is the design compressive load perpendicular to the grain and $k_{c,90}$ is a factor that takes to account the load configuration, probability for splitting.

For compression parallel to the grain, the following expression shall be satisfied: (Norges Standardiseringsforbund, 1995).

$$\sigma_{t,0,d} \leq f_{t,0,d} \tag{2.24}$$

2.2.3.6 Combination of axial forces and bending

Straight timber components can be affected by axial forces (tensile or pressure) or they may be exposed to a combination of bending and axial force.

Straight component that affected by both axial tensile force and bending must satisfy the following requirement: (Norges Standardiseringsforbund, 1995).

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \qquad (2.25)$$

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \qquad (2.26)$$

2.2.3.7 Axial buckling

The Euler column is the basic case for column buckling. It is a mathematically straight, prismatic, freely structured and centrally loaded rod. The rod is sufficiently slender to crack without the stress level anywhere at the cross-section exceeding the material’s ultimate strength. The buckling load is defined as:

$$P_E = \pi^2 \frac{E*I}{L^2} \qquad (2.27)$$

Where EI is the bending stiffness of the column and L is the length of the column.

The Euler load P_E is the reference value that usually uses to indicate the buckling load or critical load of the column. For other boundary conditions than ideally free structured can the buckling load be defined as:

$$P_E = \pi^2 \frac{E*I}{(K*L)^2} \qquad (2.28)$$

Where K*L is an effective length (buckling length) that defines the length between two points on the deformed buckling shape where the curvature is zero.

The figure 2-7 shows theoretical and recommended K-values for some idealized column cases where rotations and displacements in the end points are either completely retained or completely free (Bell et al., 2015; Norges Standardiseringsforbund, 1995).

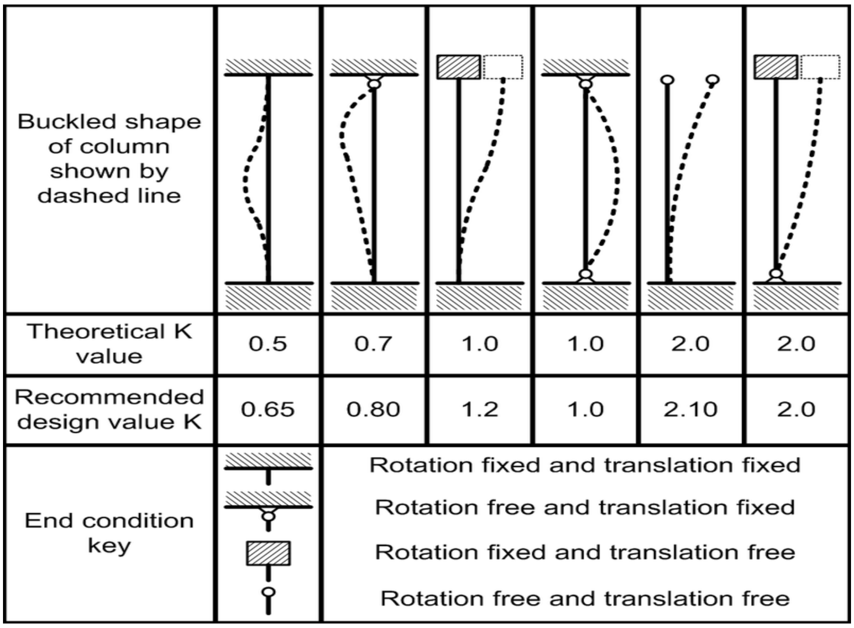


Figure 2-7: Theoretical and practical buckling length for columns with varying end condition (Wikipedia, 01.Feb. 2019)

2.2.3.7.1 The column subjected to pure axial compression

This is how the minimum requirements of column strength can be determined by Eurocode 5. As for the other Eurocodes is the design of the column according to Eurocode 5 based to linearized buckling analysis. The non-linear effects are considered by introducing a reduction factor k_C. For purely axial pressure must the following requirements be satisfied:

$$\sigma_c = \frac{P}{A} \leq k_c * f_{c,d} \tag{2.29}$$

Where σ_c is the design compressive stress, $f_{c,d}$ is the design compressive strength, A is the total cross-sectional area of the column and k_c is a buckling coefficient that takes into account the buckling risk.

The calculation of k_c is based to a numerical simulation of many columns with different shape deviations and material properties that obtained from observations of real columns. The expression of the k_c values is given in modern design standards as a function of a relatively slenderness ratio λ_{rel} and defined as:

$$\lambda_{rel} = \frac{\lambda}{\pi}\sqrt{\frac{f_{c,0.k}}{E_{0,05}}} \tag{2.30}$$

And slenderness of column is given by following equation:

$$\lambda = \frac{K*L}{i} \tag{2.31}$$

Where $f_{c,0,k}$ is the characteristic compressive strength parallel to the grain.

$E_{0,05}$ is the 5% fractile of elastic modulus parallel to the grain and i is the radius of gyration.

The expression of the buckling coefficient k_c as a function of the relative slenderness is defined as the following equation:

$$k_c = \begin{cases} 1 & for \quad \lambda_{rel} \leq 0{,}3 \\ \frac{1}{k+\sqrt{k^2-\lambda_{rel}^2}} & for \quad \lambda_{rel} > 0{,}3 \end{cases} \tag{2.32}$$

Where

$$k = 0{,}5 * (1 + \beta_c * (\lambda_{rel} - 0{,}3) + \lambda_{rel}^2 \tag{2.33}$$

And β_c is a factor for structural parts within the limits of straightness that is defined as the followings:

$$\beta_c = \begin{cases} 0{,}2 & for \quad solid\ timber \\ 0{,}1 & for \quad glued\ laminated\ timber\ and\ LVL \end{cases} \tag{2.34}$$

2.2.3.7.2 The column subjected to a combination of compression and bending

Compression and bending can occur simultaneously to a wood component that can causes two possibilities for failure.

Case $\lambda_{rel} \leq 0{,}3$- in this case buckling is not applicable and any failure will occur as result of the compressive strength being exceeded to the component. In this relative slenderness area, there is no danger for buckling and thus no strength reduction. In this case must the following requirement be satisfied:

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{2.35}$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{2.36}$$

Case $\lambda_{rel} > 0{,}3$: in this case buckling may occur and any failure will result from the compressive strength multiplied by the buckling coefficient k_c being exceeded to the component. For this case, buckling and non-linear effects must be taken into account. The failure load reached when the stresses in extreme grains reach the ultimate strength of the material. In this case, the following requirement need to be satisfied:

$$\frac{\sigma_{c,0,d}}{k_{c,y}*f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{2.37}$$

$$\frac{\sigma_{c,0,d}}{k_{c,z}*f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{2.38}$$

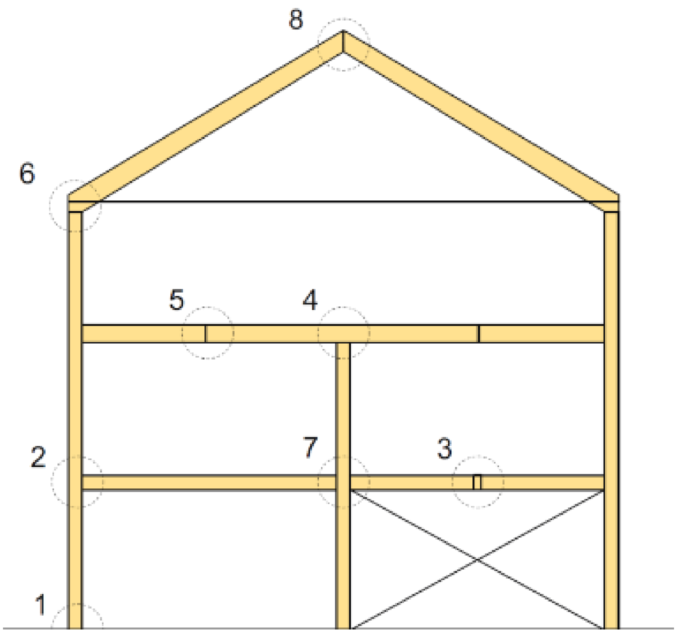
Where $k_{c,y}$ and $k_{c,z}$ are buckling coefficients that take into account the risk of buckling, the y index indicate buckling around y-axis and index z indicate buckling around z-axis (Bell et al., 2015; Norges Standardiseringsforbund, 1995).

2.2.3.8 Connections

The connections are often the weak point of the wooden structure and it is often these parts that determine the construction's bearing capacity. In addition, an unfortunate designed connection can lead to failure i.e. occur suddenly without any form of warning. It is therefore not only the strength of the connection that is important but also its flexibility (ductility) which is crucial to the dangerous situations. Most connections in timber structures are steel plates in combination with nails, dowels, bolts or screws. Self-drilling screws and threaded rods also are used both in connections and for reinforcement.

The design of timber structure involves a number of conditions, which the steel and concrete do not need to take into account. These conditions are load duration, relative humidity and orthotropic properties (angle between force and grain). The orthotropic material properties and the hygroscopic properties of the wood are important properties that the designer should be familiar with.

Figure 2-8 shows eight different types of connections that normally required in a single frame type construction.



1- Column-to-concrete connection
2- Beam-to-column connection
3- Beam-to-beam connection
4- Column top
5- Beam connection
6- Corner connection
7- Brace connection
8- Trusses connection

Figure 2-8: Typical connection types in a frame-type construction (Bell, Liven, & Norske limtreprodusenters, 2015)

Wood is a living material in the sense that it swells or shrinks by increasing or decreasing of the relative humidity in the surrounding air. Connections in timber constructions will involve the metal parts which, unlike the timber do not respond to moisture variations. This condition must be considered at designing of the connection in a timber construction (Bell et al., 2015).

2.2.3.8.1 Splitting failure

A construction timber has approximately the same strength of tension and compression parallel to the grain, while the compressive strength perpendicular to the grain is 5 to 10 times as great as the tensile strength. Therefore, it is an overall goal to reduce tensile stresses perpendicular to the grain as much as possible when designing and dimensioning of timber constructions.

The risk of splitting failure should always be investigated when the load forming an angle to the grain. According to Eurocode 5 the following criterion must be satisfied:

$$F_{V,Ed} \leq F_{90,Rd} \tag{2.39}$$

Where $F_{v,Ed}$ is the design shears force of the connection and $F_{90,Rd}$ is the design splitting capacity that calculated from the following characteristic splitting capacity:

$$F_{90,Rk} = 14b * \sqrt{\frac{h_e}{\left(1-\frac{h_e}{h}\right)}} \tag{2.40}$$

Where h is the height of timber member, b is the thickness of the member and h_e is the distance from the loaded edge to the center of the connector that is furthest in same side of loaded side.

Design shear force of the connection calculated by the following equation:

$$F_{v,d} = \frac{k_{mod} * F_{V,RK}}{\gamma_m} \tag{2.41}$$

Where $F_{v,Rk}$ is the characteristic load-carrying capacity of the connection. For determination of the characteristic capacity with metal dowel-type fasteners shall the contribution of the yield strength, the withdrawal strength and the embedment strength of the fastener be considered. Different material types that form shear planes with each other by connection have different expression to determine the characteristic load- carrying capacity.

- Timber-to-timber and panel-to-timber connections.
- Steel-to-timber connections (Association, 11.Nov.2014; Bell et al., 2015).

2.2.3.8.1.1 Connector strength

The resistance of each connector is estimated according to the theory of Johansen presented in 8.2.2 EN 1995-1-1 for panel-to-timber connections (Single shear). The figure 2-9 shows different failure modes of fastener.

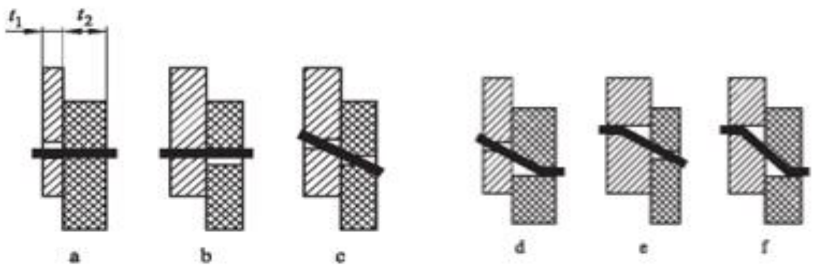


Figure 2-9: Failure modes for timber and panel connections (Norges Standardiseringsforbund, 1995)

The characteristic load-carrying capacity for nails, staples, bolts, dowels and screws per shear plane per fastener, should be taken as the minimum value found from the following expressions:

$$F_{v,Rk,a} = f_{h,1,k} \cdot t_1 \cdot d \tag{2.42}$$

$$F_{v,Rk,b} = f_{h,2,k} \cdot t_2 \cdot d \tag{2.43}$$

$$F_{v,Rk,c} = \frac{f_{h,1,k} \cdot t_1 \cdot d}{1+\beta} \cdot \left[\sqrt{\beta + 2\beta^2\left[1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1}\right)^2\right] + \beta^3\left(\frac{t_2}{t_1}\right)^2} - \beta\left(1 + \frac{t_2}{t_1}\right)\right] + \frac{F_{ax,Rk}}{4} \quad (2.44)$$

$$F_{v,Rk,d} = 1{,}05 \cdot \frac{f_{h,1,k} \cdot t_1 \cdot d}{2+\beta} \cdot \left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} \cdot d \cdot t_1^2}} - \beta\right] + \frac{F_{ax,Rk}}{4} \quad (2.45)$$

$$F_{v,Rk,e} = 1{,}05 \cdot \frac{f_{h,1,k} \cdot t_2 \cdot d}{1+2\beta} \cdot \left[\sqrt{2\beta^2(1+\beta) + \frac{4\beta(1+2\beta)M_{y,Rk}}{f_{h,1,k} \cdot d \cdot t_2^2}} - \beta\right] + \frac{F_{ax,Rk}}{4} \quad (2.46)$$

$$F_{v,Rk,f} = 1{,}15 \cdot \sqrt{\frac{2\beta}{1+\beta}}\sqrt{2 \cdot M_{y,Rk} \cdot f_{h,1,k} \cdot d} + \frac{F_{ax,Rk}}{4} \quad (2.47)$$

In the expressions above, the first term on the right-hand side is the load-carrying capacity according to the Johansen yield theory, whilst the second term $\frac{F_{ax,Rk}}{4}$ is the contribution from the rope effect (Norges Standardiseringsforbund, 1995).

The characteristic withdrawal capacity of nails, $F_{ax,Rk}$, should be taken as the smaller of the values found from the following expressions:

For smooth nails:

$$F_{ax,Rk} = \begin{cases} f_{ax,k,tip}\, d\, t_{pen,frame} \\ f_{ax,k,head}\, d\, t + f_{head,k}\, d_h^2 \end{cases} \quad (2.48)$$

For nails with improved adherence:

$$F_{ax,Rk} = \begin{cases} f_{ax,k,tip}\, d\, t_{pen,frame} = f_{ax,k,350}\left(\frac{\rho_{k,tip}}{350}\right)^{0.8} d\, t_{pen,frame} \\ f_{head,k}\, d_h^2 = f_{head,k,350}\left(\frac{\rho_{k,head}}{350}\right)^{0.8} d_h^2 \end{cases} \quad (2.49)$$

2.2.3.8.2 Column-to-concrete connection

Column end that rests directly on the concrete should be protected by moisture barrier, e.g. oil-hardened or eventual a rubber membrane. The column foot of outdoor columns or columns in areas that are exposed to water must be designed to protect as much as possible, so that it can easily be dried.

In order to avoid direct contact between the column foot and the foundation a standard or specially designed column shoe can be used. A joint-bearing as column shoes transmits horizontal and vertical forces between column and foundation but not moments. However, it is an advantage if the fastener has enough moment stiffness to keep the column stable during assembly. Since moments can easily lead to splitting then the connection should be designed as well as it does not obstruct a slight incline.

The simplest and most common used form of the joint-bearing is to attach the column foot to two externally flat steel plates. The steel plates are attached to the column foot with nails, screws or bolts. Such connections can handle both small and large horizontal forces. The figure 2-10 shows some types of anchors which used to attach columns to the foundation.

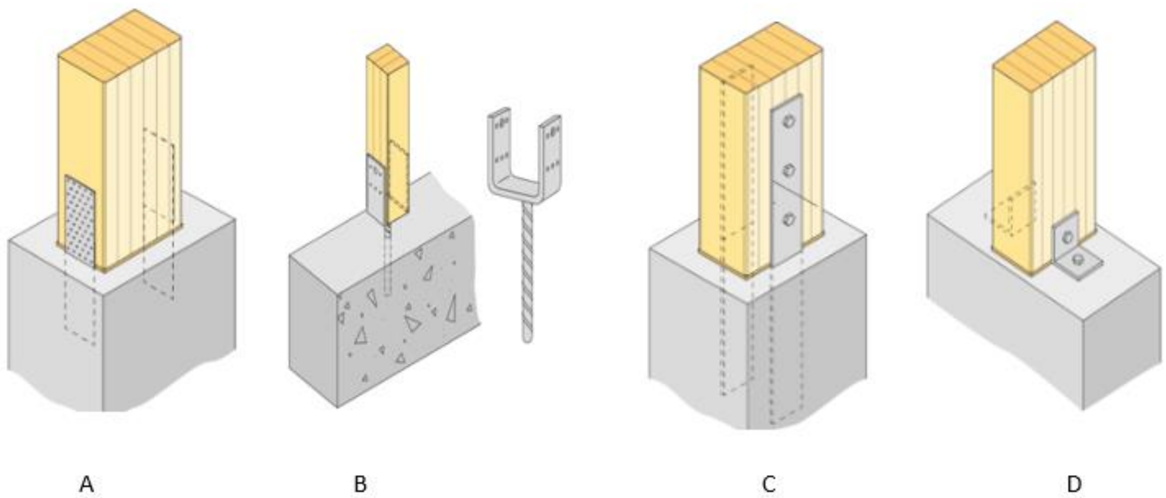


Figure 2-10: Fitting of column foot to foundation by some joint-bearing (Group)

A) *Nailing plate for column foot*: hinged fixing of the column foot with nail plates on both sides. This is a simple solution and suitable for both horizontal and vertical forces.

B) *Post anchor for column foot*: Post Anchor is another column shoe that used for potting or mounting in concrete to attach the timber columns, such as fences and terraces.

C) *Steel plate for column foot*: this is hinged or clamped fixing of the column foot with flat steel that is an equivalent option to using of nailing plates. It is bolts or wood screws that transfer the load.

D) *Steel angle for column foot*: usually, the steel angles are used symmetrical on each side of the columns and they attached to the column food by screws (Group).

2.2.3.8.2.1 Hold-down connection resistance

The design resistance R_d of the hold-downs is determined as the minimum value among the resistances relating to the following failure modes:

- Nailing failure
- Hold-downs steel failure
- Failure of the anchor
- Extraction of the anchor

And In this case must the following requirement be satisfied:

$$T_{a,d} \leq R_{a,d} = min.\left(R_{c,d}; R_{s,d}\right) \tag{2.50}$$

$$T_{p,d} \leq R_{p,d} = min.\left(R_{t,d}; R_{pull,d}\right) \tag{2.51}$$

$R_{a,d}$ is the design value of the hold down resistance, assumed to be the lower of the values of the design resistance of all the failure mechanisms associated with it. $R_{p,d}$ is the design value of the anchor resistance, assumed to be the lower of the values of the design resistance of all the failure mechanisms associated with it. $T_{a,d}$ is the design value of the tensile force acting on the hold down and $T_{p,d}$ is the design value of the tensile force acting on the anchor (Jaspart & Weynand, 2016).

Nailing resistance:

The design value of the load-bearing capacity of the nailing is given by the following expression

$$R_{c,d} = \frac{k_{mod} \cdot R_{c,k,dens}}{\gamma_M} \tag{2.52}$$

$R_{c,k,dens}$ is the characteristic value of the nailing resistance. This value can be is reduced by the k_{dens} factor when the density of the material used is less than 350 kg /m^3. k_{dens} can be evaluated using the formula:

$$R_{c,k,dens} = R_{c,k} \cdot \left(\frac{\rho_k}{350}\right)^2 \tag{2.53}$$

k_{mod} is the modification factor taking into account the effect of the duration of load and moisture content and γ_M is the partial factor for the connections.

Hold-down steel resistance:

The tensile design strength of the hold-down can be evaluated according to the following formula:

$$R_{s,d} = \frac{R_{s,k}}{\gamma_{M2}} \tag{2.54}$$

Where $R_{s,k}$ is the characteristic value of the resistance of the angle bracket and γ_{M2} is the partial factor for resistance of cross-sections in tension to fracture.

Tension resistance of the anchor:

The tension resistance of the anchor is evaluated by the following formula:

$$R_{t,d} = \frac{0.9 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} \tag{2.55}$$

Where f_{ub} is the ultimate tensile strength of the anchor, A_s is the resistant area of the threaded portion of the shank of the anchor and γ_{M2} is the partial factor for resistance of cross-sections in tension to fracture.

Pull-out resistance of the anchor:

The design pull-out resistance can be calculated as:

$$R_{pull,d} = \frac{R_{pull,k}}{\gamma_{Mc}} \tag{2.56}$$

$R_{pull,k}$ is the characteristic value of the pull-out resistance and γ_{Mc} is the corresponding partial safety factor (Jaspart & Weynand, 2016).

2.2.3.8.2.2 Angle bracket connection

The design resistance R_d of an angle bracket is determined as the minimum value among the resistances relating to the following failure modes:

- Shear failure of the angle and/or of the group of fasteners of the connection.
- Shear failure of the anchors connecting the concrete.

Design for shear resistance of an angle bracket shall satisfy the following requirements:

$$V_{a,d} \leq R_{a,d} \tag{2.57}$$

$$V_{p,d} \leq R_{p,d} \tag{2.58}$$

Where $V_{a,d}$ is the shear force acting on the angle bracket and $V_{p,d}$ is the shear force acting on the most stressed anchor (Jaspart & Weynand, 2016).

Angle bracket bearing capacity:

The design value of the shear bearing capacity of the angle bracket can be estimated from the characteristic value by means of the following expression:

$$R_{a,d} = \frac{k_{mod} \cdot \mathrm{R_{a,k,dens}}}{\gamma_M} \tag{2.59}$$

Where $\mathrm{R_{a,k,dens}}$ is the characteristic value of the nailing resistance. This value is reduced by the k_{dens} factor when the density of the material used is less than 350 kg/m^3. k_{dens} can be evaluated using the formula $\mathrm{R_{a,k,dens}} = \mathrm{R_{a,k}} \cdot \left(\frac{\rho_k}{350}\right)^2$.

Anchor bearing capacity:

The design value of the shear strength of the anchor is evaluated as:

$$R_{p,d} = \frac{R_{p,k}}{\gamma_{Ms,V}} \tag{2.60}$$

Where $R_{p,k}$ is the characteristic value of the shear strength of the anchor, $\gamma_{Ms,V}$ is the partial safety factor (Jaspart & Weynand, 2016).

2.3 Snow avalanche

Snow avalanche is a natural phenomenon that occurs every winter in all areas where there is enough snow and the terrain is suitable to snow. Snow avalanche is a fast movement of snow down a mountain slope. Density, velocity and depth of the flow are extremely variable, depending on various parameters such as topography, snow characteristics and vegetation. Snow avalanche is triggered by a complex interaction between snow, weather and terrain. In addition avalanche can be triggered by human activities such as scooter driving or skiing (McClung & Schaerer, 2006; Norges Geotekniske Institutt, 11.Mars.2019).

Snow avalanche and avalanche accidents have many consequences. The main consequences are loss of human life and physical and mental injuries to people. Those who are usually exposed to avalanche hazard are people who either live or work in avalanche areas as following:

- Resident and cabin people in avalanche area
- Road users
- Operating- and maintenance personnel
- Construction worker
- Military personnel, skiers, mountaineers

The number of avalanche victims has decreased from the 19th to the 20th century. The decline may be due to several factors. For example, the most vulnerable settlements have gradually been vacated and avalanche activity is reduced by climate change. Greater understanding of avalanche hazard and better planning of construction areas has also led to a markedly reduction in the loss of human life and destruction of buildings (Kristensen, April.2003).

2.3.1 Avalanche path (start to end)

A terrain feature where avalanche occurs is usually divided into three main parts as figure 2-11.

Figure 2-11: The avalanche path (Atkins), 27.Dec.2017)

Release zone is the zone where the avalanche starts. The starting zone is the top of the avalanche starting point i.e. the fracture edge and the lower boundary of the slipway. The most important factors that should be mapped in the starting zones are:

- The terrain slopes

- The terrain slope in relation to rainy wind direction
- The terrain slope in relation to solar radiation
- Vegetation and roughness

All areas on a mountain side or on a slope that is steeper than 30 ^{o} and not covered by dense forests are possible areas for avalanches.

Avalanche track is the middle part of the avalanche path where the avalanche passes without leaving any significant avalanche masses. The avalanche track is divided into two main groups, channeled and open track.

In channeled track, the heavy part of the avalanche follows the channel and it is easy to predict where the bulk of avalanche will hit. It often happens that larger avalanches have widths that are larger than the channeling, the parts of the avalanches that go beyond the canal take completely new tracks. In open track, the avalanches will have approximately the same width as the starting zone.

Deposition or Run-out zone is the lower part of the avalanche path where the avalanche is braked and stops. And where most of the avalanches are deposited (Kristensen, April.2003).

2.3.2 Run-out distances and Return period

Establishing of run-out distance for avalanches is an important task in connection with planning and securing of cabins, roads and other infrastructure. The avalanche run-out is usually defined as the limit of the outermost avalanche deposit. The extension of the avalanche can be described by two parameters. Either by the length L or by the run-out angle α as shown in the figure 2-12.

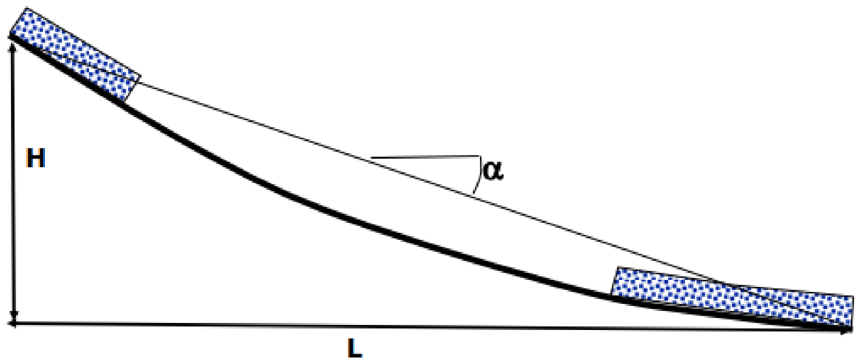


Figure 2-12: The extent of the avalanche (Norem, 2011)

Knowledge about the run-out distance is build up by collecting data on previously known avalanches. The mapped avalanches provide knowledge about the extent of avalanche deposits, the release zone and the profile of the avalanche path. This data has made it possible to do statistical comparisons to develop models for estimating of the run-out distance. The models can be divided into two main groups, topographic and dynamic models.

The topographic models are characterized by the fact that only topographical factors are included in the model, while the dynamic models are based on the numerical velocity models and calculates the run-out distance by fixed values for parameters that are included in the models.

There is currently no method for calculating the run-out distance that provides accurate answers as to how far avalanche with a certain return period can go. As a first control, both the α-β model (topographic model) and the energy line method (dynamic model) should be used. If there is access to a numerical model or a database of previously registered avalanches, these possibilities should also be used (Norem, 2011).

In some avalanche paths avalanche happens every year and in others, it may happen once in decades. The further out in the avalanche path is, the less probability to being hit by the avalanche. In terms of workplaces and building where people live or usually reside, the law states that the probability of being hit by avalanches should be less than 1/1000 per year. The stipulated requirements for design return period for houses with human activity are 1000 years and for houses with little activity are 100 years, for example garages and storehouses. The run-out distance can be expressed as a function of return period. The figure 2-13 shows the relationship between run-out distance and return period (Kristensen, April.2003).

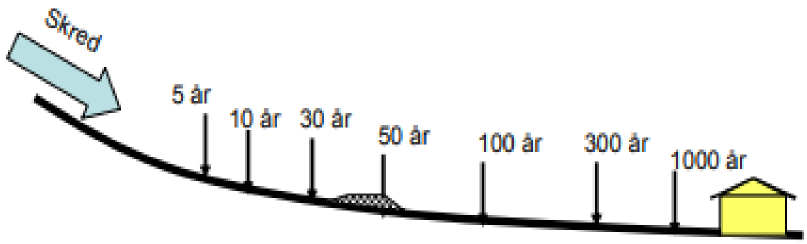


Figure 2-13: The relationship between run-out distance and the return period of avalanche (Norem, 2011)

2.3.3 Avalanche velocity

It is important to be able to estimate the velocity of design avalanche because the velocity is major importance to the choice of security measures against cabins in mountains and how these should be designed.

A volume of avalanche masses that moving down a valley side as in figure 2-14 will be affected by both accelerating and decelerating stresses. There are weight components of the avalanche masses parallel to the ground that make up the accelerating stresses, while the various forms of friction constitute the decelerating.

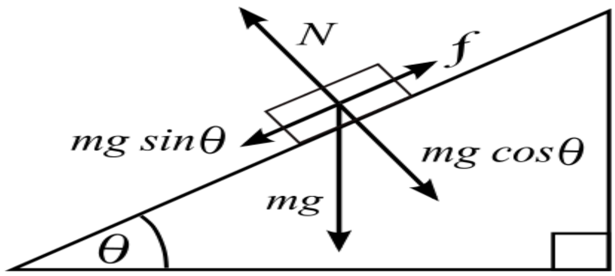


Figure 2-14: Forces acting on a control volume of avalanche masses (July Thomas, 4.May.2016)

If the control volume includes a unit area, the following equation can be set up:

$$m * a = \rho * h * \frac{dv}{dt} = P - f = \rho * g * h * sin\theta - (f_c + f_d) \quad (2.61)$$

Where m is the mass, a is the acceleration, ρ is the density, h is the flow height, v is the velocity, t is the time, P is the accelerating force, f is the frictional force, f_c is the velocity-dependent friction and f_d is the velocity-independent friction.

In order to calculate the velocity of avalanche, it is usual to group the friction force into two main groups. A group of friction that is independent of the velocity, f_c and a friction type which increases with the velocity, f_d. If all the friction is independent of the velocity, the avalanche will never reach a constant velocity and if all the friction is dependent of the velocity, all avalanches will only stop after they come out in flat ground. Since all avalanches could stop downhill and they could also achieve a constant velocity in steep terrain, so the friction must consist of both dependent and independent of velocity (Norem, 2011).

2.3.3.1 Numerical models

There are several numerical models for calculating of velocity of most avalanche types. The complexity of the models varies greatly. All existing models are based on the friction which is dependent on velocity and independent on velocity.

Equation 2.61 can be written in its simplest form as following:

$$m * a = P - (f_c + f_d) = \rho * g * h * sin\theta - \left(C + \mu * \rho * g * h * cos\theta + k\left(\frac{v}{h}\right)^2\right) \qquad (2.62)$$

Where C is the cohesion, μ is the friction coefficient, k represents the viscosity and v/h is the velocity gradient.

When the avalanche moves downward a mountain with a constant slope, it will achieve a constant velocity and at this velocity the acceleration is equal to zero. The constant velocity can be found by setting the acceleration equal to zero in the equation 2.62 and this gives the following equation (Norem, 2011).

$$v_c = \left(\frac{\rho * g * h^3 * (sin\theta - \mu * cos\theta}{k}\right)^{\frac{1}{2}} \qquad (2.63)$$

2.3.3.2 Graphic models

When the avalanche moves downward the avalanche path, there is an exchange of energy. In the first phase of the path, the avalanche has primarily potential energy. Eventually, it is transmitted to the velocity energy and heat. Where the heat represents the energy loss. When the avalanche has stopped, the potential energy and velocity energy are equal to zero, and all potential energy has converted to heat.

Bernoulli's equation states that the sum of potential energy, velocity energy and energy loss of thin currents is equal in each location of the flow path. These types of energy can be expressed by energy height.

$$H_e = H_z + H_k \qquad (2.64)$$

Where H_e is the energy height, H_z is the location height that represents the potential energy corresponds to the height at any point in the avalanche path. H_K is the velocity height and it is expressed by the following equation:

$$H_k = \frac{v^2}{2g} \qquad (2.65)$$

Then the velocity of each point can be calculated from the velocity height in this point by: (Norem, 2011).

$$v = \sqrt{2gH_k} \qquad (2.66)$$

2.3.4 Avalanche load on structures

Load from avalanche against structures will mainly include the following types of loads:

1. Press against structures that are perpendicular to the avalanche direction.
2. Shear stresses against structures that are placed parallel to the avalanche direction.
3. Loads against surfaces that make an angle with respect to the avalanche direction.

Knowledge about pressure against constructions has been collected over many years based on:

- Field trials, mounting of instrumented constructions in the avalanche path
- Recalculation of structures that are destroyed by avalanche
- Theoretical analyzes
- Knowledge about pressure against structures of other materials than snow but have comparable physical properties (Norem, 2011)

2.3.4.1 Determining design avalanche pressure

In general, the pressure exerted on the constructs is dependent on which layer of the avalanche charges them. There are four flow regimes that are distinguished for the determination of the impact force on the wall-like structures. The figure 2-15 shows the four regimes.

- Snowpack (light green)
- Dense flow/the heavy part of avalanche (red)
- Fluidized flow (yellow)
- Suspension flow/powder part (green)

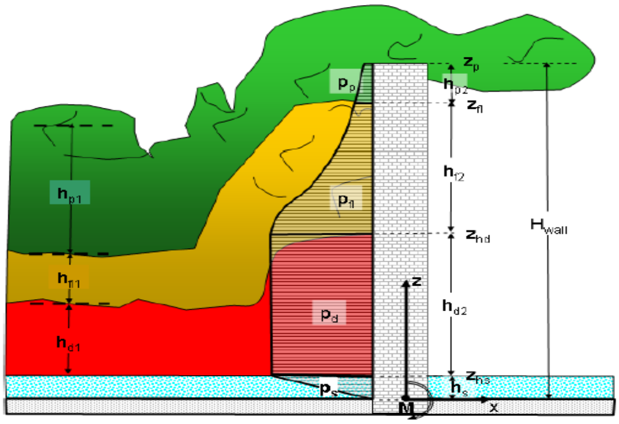


Figure 2-15: Avalanche pressure distribution on the wall (Jóhannesson, Gauer, Issler, Lied, & Hákonardóttir, 2009)

2.3.4.1.1 Snowpack

Snowpack is the amount of snow that lying on the ground before the avalanche occurs. This is important for two reasons. First, the thickness of this layer has an impact on how high the avalanche hits on the wall structure. The second, there are large shear stresses in the boundary between this layer and the avalanche. These shear stresses are transmitted downwards in the snowpack, which leads to pressure against eventual structures. This type shear stresses are registered as pressure on mast structures in the Ryggfonn project, Norem (1985) and Norem (1990). For simplicity, a linear pressure distribution will be assumed within the snowpack. The pressure transmitted through the snowpack can be calculated by the following equation:

$$P_s(z) = P_d \frac{z}{h_s} \tag{2.67}$$

The force normal onto a rectangular wall exert by an avalanche through the snowpack is:

$$F_{sx} = \frac{b * h_s}{2} * p_d \tag{2.68}$$

And the corresponding moment about the y-axis is:

$$M_{sy} = b * P_d \int_0^{z_s} z * \frac{z}{h_s} dz = \frac{2}{3} h_s F_{sx} \tag{2.69}$$

Where P_d is the dynamic pressure of the avalanche at the lower boundary, z is the height of the wall, b is the width of wall construction that is affected by snowpack and h_s is the height of the snowpack (Jóhannesson et al., 2009; Norem, 2011).

2.3.4.1.2 Dense flow

The largest avalanche pressures are registered from the dense flow. The pressure is depending primarily on:

- The avalanche velocity, maximum velocity and the velocity distribution with the height
- The density of the avalanche
- The width and height of the construction
- The thickness of the avalanche layer
- The height in the avalanche, pressure is measured

The avalanche velocity can be calculated by using the numerical models or a simple graphical solution as presented in chapter 2.3.3.

The pressure in the dense flow is assumed to be evenly disturbed along the wall. This is a simplification as there are most likely an increasing pressure with depth. When the avalanche flowing towards a large extent, the load on it will correspond to the stagnation pressure which is expressed by the following equation:

$$P_d = \frac{1}{2} * \rho * k * C_d * v^2 \tag{2.70}$$

When the avalanche hits the structure, a force will exert against this structure which can be expressed by:

$$F_{dx} = \frac{1}{2} * \rho * k * C_d * v^2 * h_d * b \tag{2.71}$$

In the case of not perpendicular between the avalanche pressure and the wall construction as the figure 2-16, the avalanche pressure to the wall will be calculated by the following equation:

$$P_d = \frac{1}{2} * \rho * k * C_d * (v * sin\alpha)^2 \tag{2.72}$$

Where ρ is the density of the avalanche, v is the velocity of the avalanche, b is the width of the wall construction that is affected by the dense flow avalanche, h_d is the height of the dense flow, C_d is the drag coefficient and k is the constant.

The constant k increases the value of the drag coefficient beyond the default values. k is estimated equal to 2 for completely dry avalanche and up to 6 for wet avalanche. The constant k can be selected equal to 3 as an average for most design avalanche.

The drag coefficient is dependent on the shape of the structure and the type of the flow. In some textbooks about hydrodynamics, there are tables for which values should be chosen for the drag coefficient. These values apply to turbulent flows i.e. liquid with the high velocity. But the drag coefficient for avalanche will be greater than liquid due to high viscosity and high velocity (Jóhannesson et al., 2009; Norem, 2011).

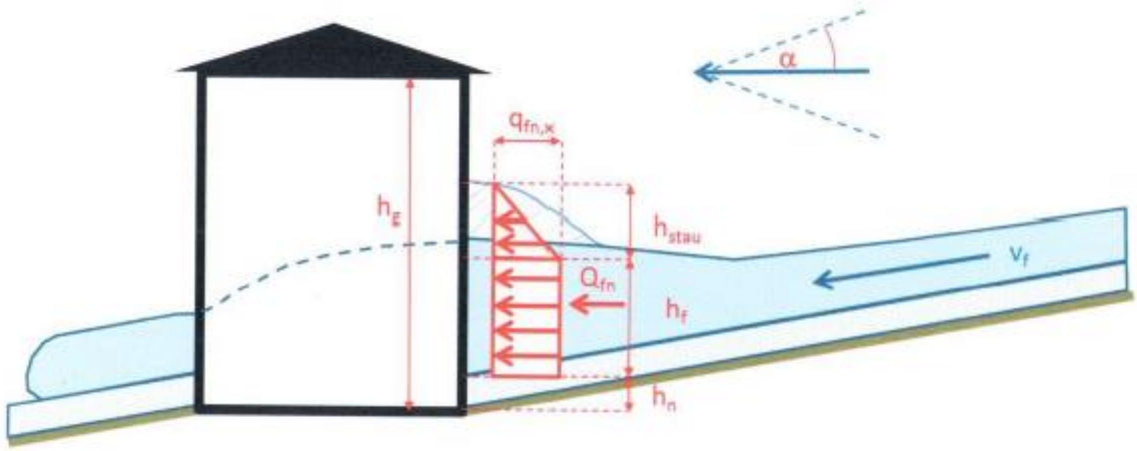


Figure 2-16: Design avalanche pressure on the wall construction (NMBU & Thiis))

2.3.4.1.3 Suspension flow/powder part

In the powder part of the avalanche, there is turbulence in the air which keeps the snow particles floating. The turbulence is generated in the boundary between the top of the dense flow and the air above. The turbulence is reduced with increasing distance from the boundary layer. Field experiments, model experiments and theoretical analyzes have shown that both velocity and density of snow particles are reduced by the height. Norem (1990) proposed to calculate the avalanche pressure from the powder part in the area where the avalanche has maximum velocity by the following criteria:

1- At the lower interface, the pressure is given by the following equation:

$$P_{p.l} = \frac{1}{2} * \rho_p * v^2 \qquad (2.73)$$

Where v is the avalanche velocity and ρ_p is the density of the powder part in the lower part. This density varies between 4 and 10 kg/m^3 and it can be selected 8 kg/m^3 as an average value.

2- It is assumed that the pressure decreases rapidly with height. Thus, the pressure at the upper part of the suspension flow is given by the following equation:

$$P_{p.u}(z) = P_{p.l} * \left(\frac{h_p - z}{h_p}\right)^{\frac{1}{3}} \tag{2.74}$$

Where h_p is the height of the powder part and z is height of the wall that is affected by the suspension flow (Jóhannesson et al., 2009; Norem, 2011).

2.4 Wall characteristics

2.4.1 Timber frame wall

The term timber frame typically describes a system of panelized structural walls with small section timber studs. Most wooden houses in Norway are built of timber frame structure which is a load bearing wooden structure and transmits vertical and horizontal load to the foundations.

Timber frame wall construction is very similar to the concrete cavity wall construction so that the outer leaf remains as masonry and a cavity is installed. The only difference between these two walls is that the timber frame construction has an internal leaf. The timber frame wall construction consists of timber stud frame with insulation between each stud, which is coated by either Oriented Strand Board (OSB) or plywood. Breathing membrane and air tightness membrane be also fixed to the plywood or OSB (Wiki, 08.July.2018).

Figure 2-17: Timber frame wall construction (Ltd; WA)

2.4.2 CLT wall

Cross Laminated wall consists of sawn, planks, layered wood and glued. Each layer is located perpendicular to the previous layer. Structural rigidly of the panel in both directions is obtained by assembling the wood layers perpendicular to each other. This is similar to plywood but with thicker components. The panel has great tensile and compressive strength in this way. Generally, CLT panels are mounted and cut in their production and the joints and openings of walls are specified during the design. Figure 2-18 shows examples of CLT wall constructions. Since the solid panel consist of a single material, the structure requires no cladding. This reduces both the need for labor and materials for its final appearance (Franco, 20.May.2018).

Figure 2-18: CLT wall construction (Companies; Franco, 20.May.2018)

2.4.3 Shear-wall

The timber-frame wall as a building element is a part of the stabilizing structure. In such a case the timber frame shear-wall works out as a support system, not only for transferring of the vertical loads but also for transferring of the horizontal loads which act on the building. For example, in the case of wind, the horizontal load is transmitted to the timber frame shear-wall by diaphragm action from the floor. The racking force on the shear-wall is divided over the length of the wall as distributed load on the top of the wall and this load will be transmitted to the foundation by the shear-wall.

The timber-frame wall is a composite element that consists of connections, timber frame elements, wood or gypsum-based panels and fasteners (nail, screw or stable). The fasteners are located along the perimeter of the sheeting, these help the shear force on the top of the wall elements be transferred to the sheeting. The sheeting acts as a brace to the timber frame wall and makes able shear-wall transfer of the racking force (Hoekstra, 2012).

2.4.3.1 Diaphragm

Diaphragms are typically horizontal as floor or roof system which transmits lateral load to the vertical resisting elements as frames and shear walls. The figure 2-19 shows the diaphragm design and load distribution of the lateral load.

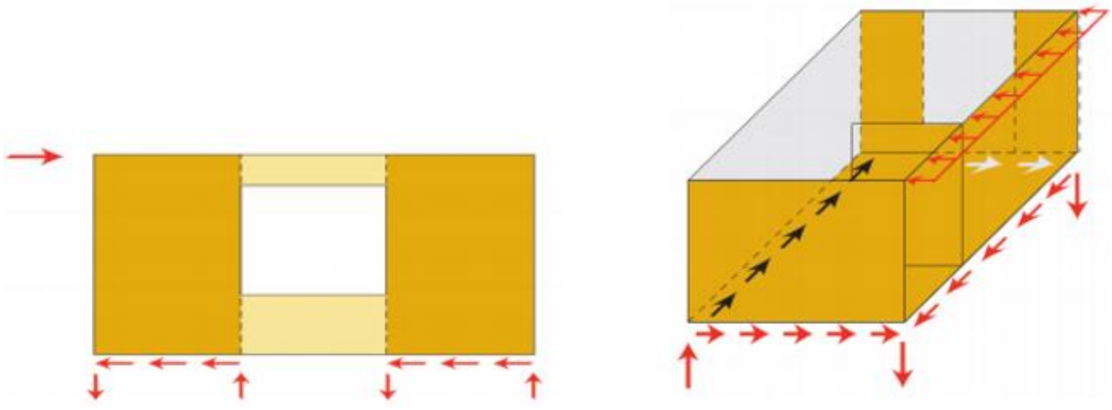


Figure 2-19: The diaphragm design and load distribution of the lateral load (Bjørnfot, 30.Okt.2018)

Distribution of the horizontal load over the shear walls by diaphragm, in addition to the stiffness of the shear walls may also depend on the stiffness of the horizontal diaphragm. Diaphragm can be either rigid or flexible. Figure 2-2 shows differences force distribution over the shear-walls in case of a flexible diaphragm and rigid diaphragm. Thus, to characterize a horizontal diaphragm is rigid or flexible, the magnitude of the expected deflection of the diaphragm relative to the shear wall deformation can be used (Nandeesh.N.H, Jan.2018).

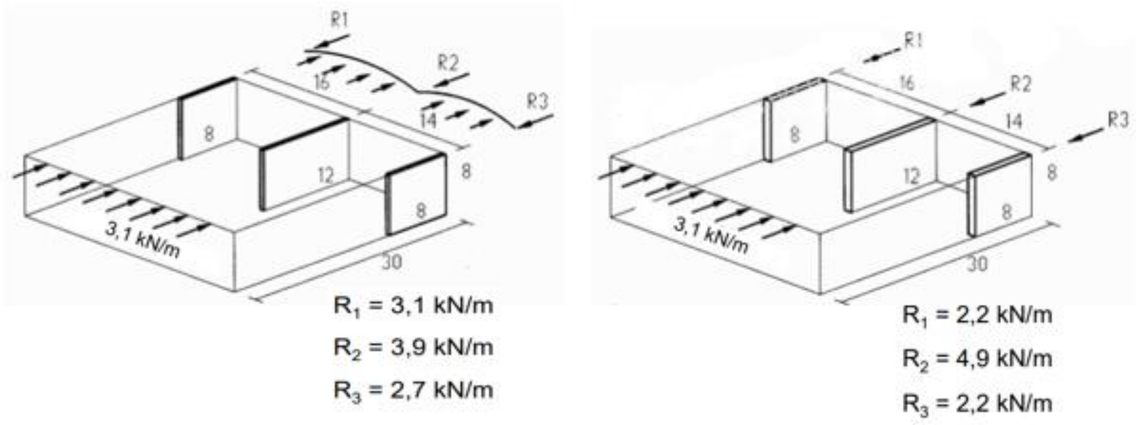


Figure 2-20: Differences force distribution over the shear-walls in case of a flexible diaphragm (left) and rigid diaphragm (right) (Bjørnfot, 30.Okt.2018)

According to the international building code, the diaphragm is rigid if the deformation of a diaphragm is less than twice the average racking deformation of the shear wall. The diaphragms for most timber-frame building is considered to be rigid because shear walls in a wood-frame building have significantly greater deformations than floor diaphragm (Nandeesh.N.H, Jan.2018).

2.4.3.2 Arrangement of shear-wall

It is very important to have correct number and correct distribution of shear walls in buildings. The location of shear walls must be such that the structure can distribute the horizontal forces in all directions. It must also maintain horizontal displacements in the building's longitudinal and transverse directions and as well as rotation about the building's vertical axis. It is relatively easy to hold the displacement but to prevent rotation, the shear walls must be positioned so that they or their extensions do not intersect each other at one point.

To achieve the global stability, at least three shear walls are required. Three shear walls provide a statically determined system and a force distribution which is independent of the wall's stiffness. More or less than three shear walls provide a static indefinite system and a force distribution depends on the stiffness of the walls. The figure 2-21 shows examples of shear walls that give a stable building and figure 52-22 shows examples of shear walls that give an unstable building (Kristiansen, 2016).

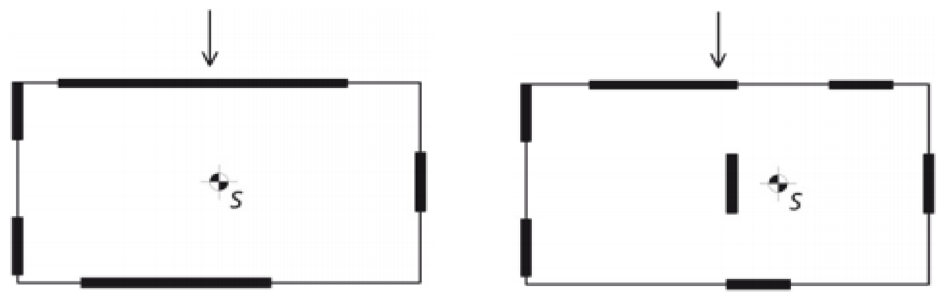


Figure 2-21: Suitable arrangement of shear wall (Wallner-Novak, Koppelhuber, & Pock, 2014)

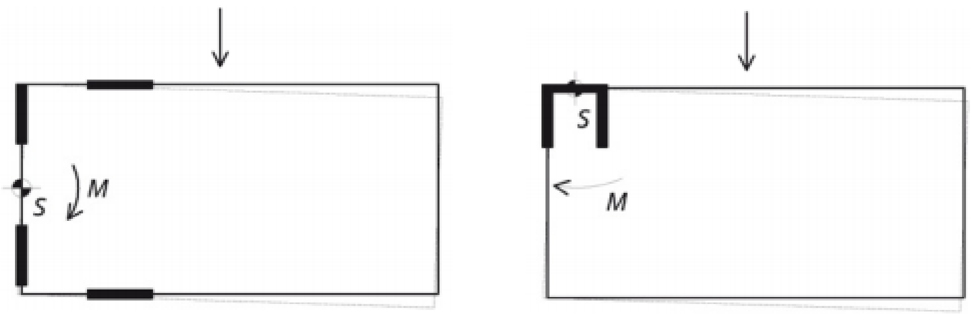


Figure 2-22: Unsuitable arrangement of shear wall (Wallner-Novak et al., 2014)

2.4.3.3 Influence of fasteners on the stiffness of shear wall timber frame

The stiffness of the wall is strongly dependent on the stiffness of the fasteners. The frame will be unstable without fasteners and the sheeting. The disproportionate influence assessment of the fasteners was made in 1985. According to William McCutcheon, the fasteners attach the sheeting to the frame and their strength and stiffness plays an important role to racking behavior of the timber shear wall. The fasteners can be determined by testing of individual nails and when the load displacement characteristics of the individual nail is known, the performance of the wall can be predicted (Hoekstra, 2012).

2.4.3.4 Influence of hold-down on the stiffness of shear wall timber frame

Figure 2-23 shows a wall panel which is subjected to a horizontal load $F_{v,E}$. During the in-plane racking force, the fasteners will distort, and the top rail of the shear wall will displace horizontally. A tension force $F_{t,Ed}$ will be developed in the tensile stud if no vertical load is applied. In order to prevent uplift of this stud, the hold-down anchoring is needed (Hoekstra, 2012).

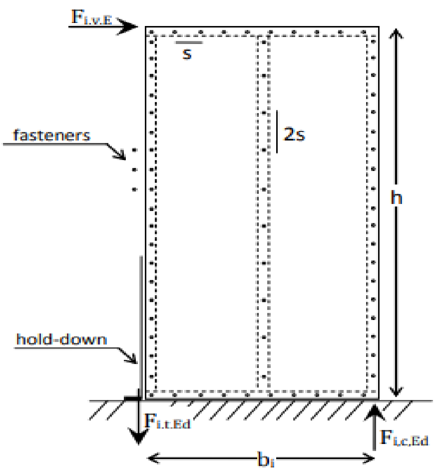


Figure 2-23: Timber-frame shear-wall panel with fastener spacing (Hoekstra, 2012)

Figure 2-24 also shows the importance of the hold-down connections and angle brackets connections in relation to the various deformation modes which a wall can be subjected during the horizontal loads. It shows two different types of anchors that can be used on the bottom to anchor timber walls. Angle-bracket anchors be used for shear force and for sliding protection and hold-down anchors be used for tension or hold down forces and to prevent lifting.

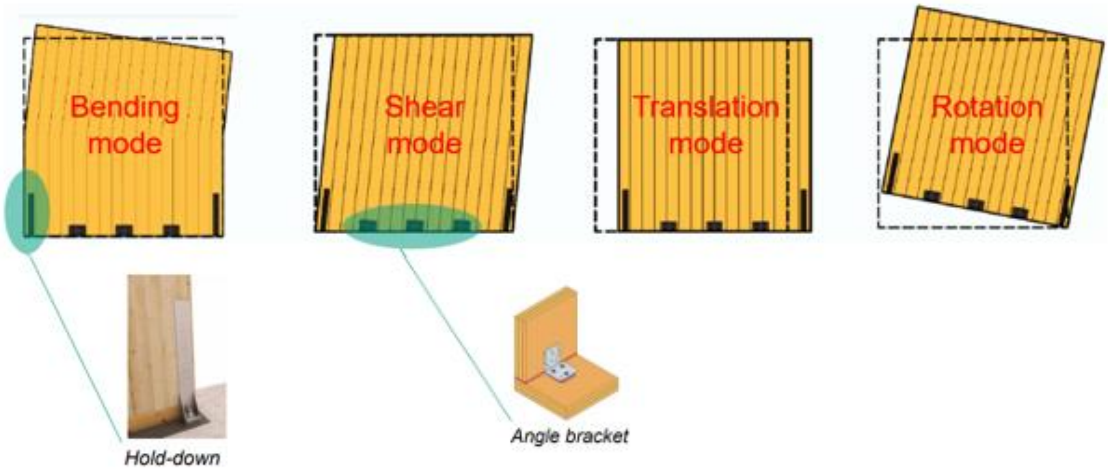


Figure 2-24: Separate deformation modes of the total horizontal load deformation of the CLT shear-wall (Bjørnfot, 30.Okt.2018)

2.5 Wood and Circular economy

Circular economy is a useful concept to investigate how sustainable resources use can be pursued. Since the building industry accounts as a significant share of resource consumption, it is relevant to focus on this.

More recently, several different approaches to the concept have emerged but most researchers agree that the circular economy represents a solution for achieving sustainable development. The purpose of a transition to circular economy is to change the current production-and consumption towards more sustainable use of resources with less pollution. Such changes may include improving the efficiency of material and energy use, the use of renewable resources and the use of waste as a resource (Utkilen, 2018).

2.5.1 Circular economy

The concept of circular economy has its roots from the 70s. It was developed by a group of academics who used the term in connection with industrial ecology. Circular economy can be described as a system in which industrial compounds act as an ecosystem that renews itself constantly (Preston, 2012).

A linear economic model is based on extracting resources, producing, using and disposing of them through landfill /incineration but a circular economy is based on reuse, repair, refurbishment/ improvement and material recycling in a circle where fewest resources are lost (Moum, Skaar, & Midthun, 2017).

A circular economy is justified both economically and environmentally and the focus is on reuse and the least possible environmental impact both during production and disposal. A circular economy can be divided into technical and biological cycles. It is only in the biological cycle that consumption should occur. This is either through digestion of food or by composting of degradable products.

The foundation of circular economy is many places in nature and in recent times, the digital technology has provided the opportunity to gain a much more overview of what happening in the production lines and possibility of feedback to be able to adjust the processes (Puntervold & Hanssen, 2018). The figure below shows the difference between linear economy and circular economy and how the product and materials can cycle in the technical and biological cycles in the circular economy.

Figure 2-25: The different between linear economy and circular economy (Moum et al., 2017)

2.5.2 Circular economy in construction industry

In the initial descriptions of the concept of circular economy, better management of resources is central. There will be examples of four focus areas in the construction industry, where the way of managing resources is important for the transition from a linear to a circular economy.

- Focus area 1: Handling of building's materials and components after use
- Focus area 2: Use of resources in the construction's processes
- Focus area 3: Area utilization
- Focus area 4: Energy efficiency of the buildings

Each area represents different activities and handling of various forms of resources in the construction industry, simultaneous they are part of a whole where everything is connected. The purpose of breaking down the vision of circular economy in this way is to concretize and show the breadth of what a conversion to a circular thinking can provide opportunities in the construction industry (Moum et al., 2017).

2.5.2.1 Handling of building's materials and components after use

Many in the building industry may think first to a better handling of components and building's materials when the building's lifetime is over in terms of conversion to circular economy. In order to ensure that the building's components and materials remain a value/resource in a circular system, the vision of the future may look like the following:

- The waste pyramid is turned upside down as figure 2-27 shows. Waste is greatly reduced, and the construction industry maximizes resources.

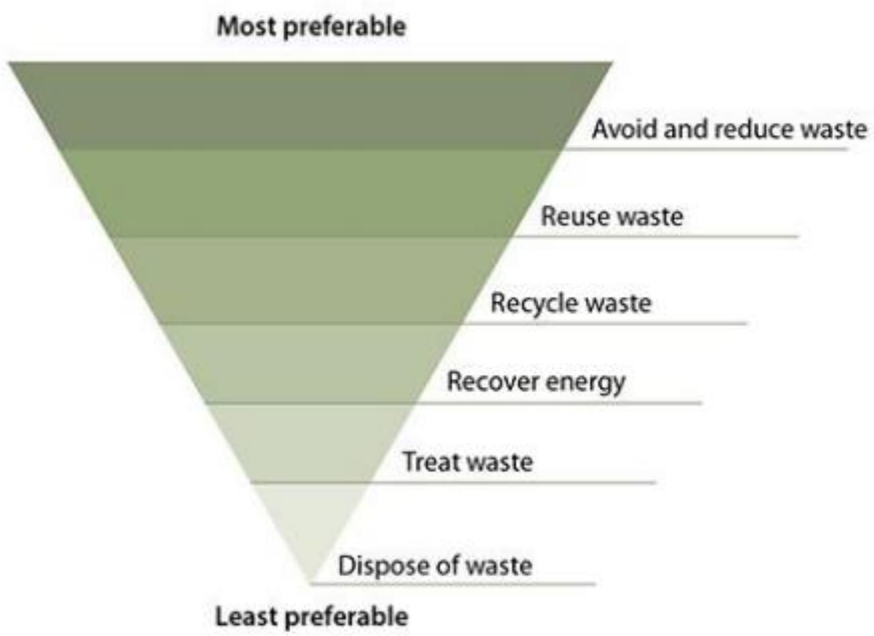


Figure 2-26 The waste pyramid turned upside down (Moum et al., 2017).

- At the same time, the extraction of new raw materials has been greatly reduced.
- The project group develops and uses design solutions that facilitate a good handling of the building's components after their use. For example, they focus on:
  - To develop robust solutions.
  - Choice of components with long life and good durability.
  - Choice of flexible solutions and components that are easy to disassemble.

- To avoid recycling solutions that involve loss of value.
- Do not use toxic or hazardous materials or constituents, or materials with a high CO_2.

▪ Urban mining is an established and profitable practice-that made possible by new infrastructure, digital technology and new business models.

▪ There are good systems/support services for the return, collection and redistribution of used buildings materials and components.

▪ There are companies that offering services to dismantle/rip and recycle complex integrated components (Moum et al., 2017).

2.5.2.2 Resource use in the construction processes

The construction process means the processes from a requirement for a new building (or a rehabilitation) to programming and concept development, detailed design and construction takeover and use to disposal.

There is great potential for a better resource utilization in these processes, which are claimed to be characterized by inefficiency and discontinuity. This leads to unnecessary loss/waste of value for example in the form of labor, money, time and quality. A construction process in a future circular system could be characterized by:

▪ Actors understanding the building and their task as part of a larger system

▪ Focus on rebuilding, adapting and reusing rather than tearing and build new

▪ Procurement and execution models that enable:
  - Continuity and clear distribution of responsibility and ownership of the building and its products
  - New forms of cooperation, for example a multi-disciplinary design team from the project's early phase

▪ A high degree of industrialization by:
  - Use of digital tools throughout the value chain that ensure good flow of information and resources through the entire building process and the life cycle of the building
  - Good flow and logistics management in the processes.
  - Standardization and reuse

  - Automated production by 3D printing, robot or standardized processes that reduce construction waste
- Design in early phase for increased life of the buildings and components, with choice of durable products and materials, flexible design solutions, focus on modularity, etc.
- Easily accessible depots and databases to obtain or return used materials and components
- Companies share/rent resources such as equipment and infrastructure on construction sites
- Building with an updated inventory available, a complete overview of its material structure. For easier disassembly and urban mining (Moum et al., 2017)

2.5.2.3 Area utilization

A third resource that is relevant to look at within a circular concept is the area. According to a rapport from Growth Within, 60% of the office buildings are empty in Europe.

In Norway, many people live in bigger space than they need, based on changes in family situation and composition. Area utilization within a circular economy can be characterized by:

- Both public and private businesses could have reduced costs and a smaller environmental footprint by sharing of office space or increasing use of virtual workplaces, for example:
  - Flexible placement and work desk sharing
  - Reduce traveling by using communication technology
  - More employees per workplace and less space per employee
- Increases focus on flexibility usage:
  - Several buildings allow a combined use
  - It is possible for people to adapt their living conditions to varying needs throughout their live, thus reduce living area per person in urban areas
- Increases focus on area sharing:
  - Access to new concepts and tools for sharing area and equipment in connection with living, work and leisure
- The building is no longer be considered merely as a product, but also as a service:
  - Empty buildings are irreversible, increased use and multifunctional use
  - The buildings adapt to the user needs and increase productivity

- Sensors and data mining provide information about utilization of the individual building throughout its lifetime (Moum et al., 2017)

2.5.2.4 Energy-efficiency of the buildings

The construction industry is often referred as 40% of the industry and this represents 40% of the total energy consumption in Norway.

In the fourth focus area, the focus should be on energy as the resource/value that is to be maintained/ maximized in a circular economy. Energy efficiency of the buildings within a circular economy can be characterized by:

- Zero emission building have become standard in smart neighborhood and cities:
  - Buildings and neighborhoods produce energy and are a part of the energy production system
  - All materials are environmentally friendly, with low CO_2
  - Energy production, distribution and use are monitored and optimized
- Nearly zero emission levels are standard almost in all reconstruction and rehabilitation of existing buildings:
  - New technology and knowledge have provided tools that enable easy cost/benefit optimization
  - Increases reuse of buildings and products
- Digital technology such as sensors and data mining for data capture for energy consumption and control of the installations (Moum et al., 2017)

2.5.3 Wood's role in circular economy

The wood industry organizes producers of lumber and other wood-based building materials and organizes over 90% of the Norwegian production capacity in the area. The wood industry is a part of the construction industry. Both productions and the products that are building materials are based on renewable resources.

The goal of transitioning to a circular economy is to increase the use of renewable resources. This presupposes that Norwegian renewable resource such as forest must be a part of the solution. Today, just under 40 % of forest is harvested in Norway. Increased use of forest raw materials as a renewable resource and replacement of fossil sources is crucial for reducing of climate emissions. Everything that can be made of oil can in principle also be made of forest (Byggfakta, 12.Oct.2016).

2.5.3.1 Wood's environmental properties

Wood is a natural material based on a renewable raw material and has little negative impact on the environment, presuming it comes from certified and sustainably managed forest. Wood is one of the most environmentally friendly building materials which is available in Norway today. Wood products have the following key environmental properties:

- The raw materials are renewable resource
- Increased use of wood reduces CO_2 emissions to the atmosphere
- Coming from a sustainable forestry that is documented through certification (PEFC and FSC)
- Large proportion of renewable energy in manufacturing processes
- Provides good indoor environment
- Recycling and reuse is easy (Treindustrien)

2.5.3.2 The wood reduces CO_2 emissions

The use of wood as a building material will contribute to reduce CO_2-content in the atmosphere in two ways:

- Carbon-storage
- Substitution

2.5.3.2.1 Carbon-storage

CO_2 released when trees burn is included in the natural carbon cycle and contributes to the formation of new biomass in growing forest. Therefore, forest-based bioenergy is neutral CO_2. Wood materials which have low emissions of fossil CO_2 in the production phase help to reduce CO_2 emissions by replacing materials that have higher CO_2 emissions during the production phase. The wood contributes to reduced CO_2 emissions in the waste phase too. If energy recovery of the materials after use replaces the use of fossil fuels, the atmosphere is directly saved from CO_2. By material recovery and reuse of materials, an extended carbon-storage is achieved in the products. The greatest savings of CO_2 emissions are achieved by using wood in wood production as recycling and then used for energy. All growing biological material absorbs carbon and carbon-storage can help to reduce CO_2 level in the atmosphere. Long-life of the wood product and fast regrowth after harvesting gives a better effect of carbon-storage (Treindustrien).

2.5.3.2.2 Substitution

Several life-cycle analyses are done with varying assumptions to clarify the substitution gain when using the wood. The University of life Sciences has made a summery after a review of surveys in Sweden and Norway. If the effect is measured as saving CO_2 emissions per used m^3 lumber, their numbers shows that:

- 1 m^3 of wood that replaces concrete reduces CO_2 emissions by 0,2-2,1 ton.
- 1 m^3 of wood that replace steel reduces CO_2 emissions by 0,2-0,5 ton.

The studies also show that the use of wood materials instead of linoleum, vinyl and carpet flooring gives even greater substitution effect (Treindustrien).

2.5.3.3 Wood's role in plastic reduction

Today, there are several raw materials within wood biomass that have replaced plastic. New plastic can be created by by-products from the wood industry so that it can be recycled and reused at the end of life, unlike most of plastics. The use of wood as a bio-plastic production is a benefit for circular economy and has a large and positive impact on the environment.

A wood component called Lignin can be easily processed through extrusion machinery and be used to make plastic part such as building materials or packaging. New processes for developing 3D-printing materials and developing by-product from wood bark extraction to replace plastics and adhesive have been developed. (Professor Frederic Pichelin, 13.Sep.2018).

2.5.3.4 Wood waste

The wood waste is sorted according to whether it is clean, or surface treated. Clean wood can be reused as materials or be used as fuel. Surface treated timber is delivered to approve reception for material recovery or energy utilization. The purpose of sending the wood for burning is to produce the heat. The heat can be used to heat water at a large plant and then it is passed on through a pipe to the customers. By burning wood waste district heating is produced, which is a cheaper alternative to heat a home instead of using electricity. Simultaneous it is economical and better for the environment. The heat can be utilized independently of the source. It replaces local heating, improves air quality and saves a lot of money by not using so much power on heating (Andreas Skjærstad & Sangasari, 06.Nov.2011).

3 Method

For analysis and design of timber structures and framed walls, TimberTech Buildings software was used. Generally, the software guides definition of the geometrical and mechanical properties of:

- The walls (Timber frame and CLT wall)
- The ceilings (Joist floors, solid timber floors, and CLT floors)
- The connections for tensile and shear forces both for ground floor and upper floor Different types of connections and fasteners can also be selected in this software By changing of the strength and stiffness, new connections can be created to customize them
- The CLT wall vertical joints

The software performs the analysis and check of the whole structure for vertical load, wind load and seismic action according to the selected standard and the site where the structure is located (CESDb.com, 26.April.2018).

In this thesis, the focus was on analyzing of timber framed wall constructions against wind load corresponding to the avalanche load. So, a wooden cabin as two cases were designed in the TimberTech software. Then both cases were subjected to 20 Kpa wind load in the y-direction and then both cases were checked for stability against the horizontal load.

The geometric shape and properties of the linear elements and floor (roof) are the same in both cases but the properties of the wall structure and connections are not the same. Since wall structures and connections are the most crucial and important components against the horizontal load in low wooden buildings therefore, most of the focus was on analyzing of these components.

3.1 Geometric design of the cabin structure

The figure below shows the geomertic design of the wooden cabin that was designed in TimberTech Buildings software. The utiliy area of the designed cabin is 12m X 8m with a height of 4,5 m.

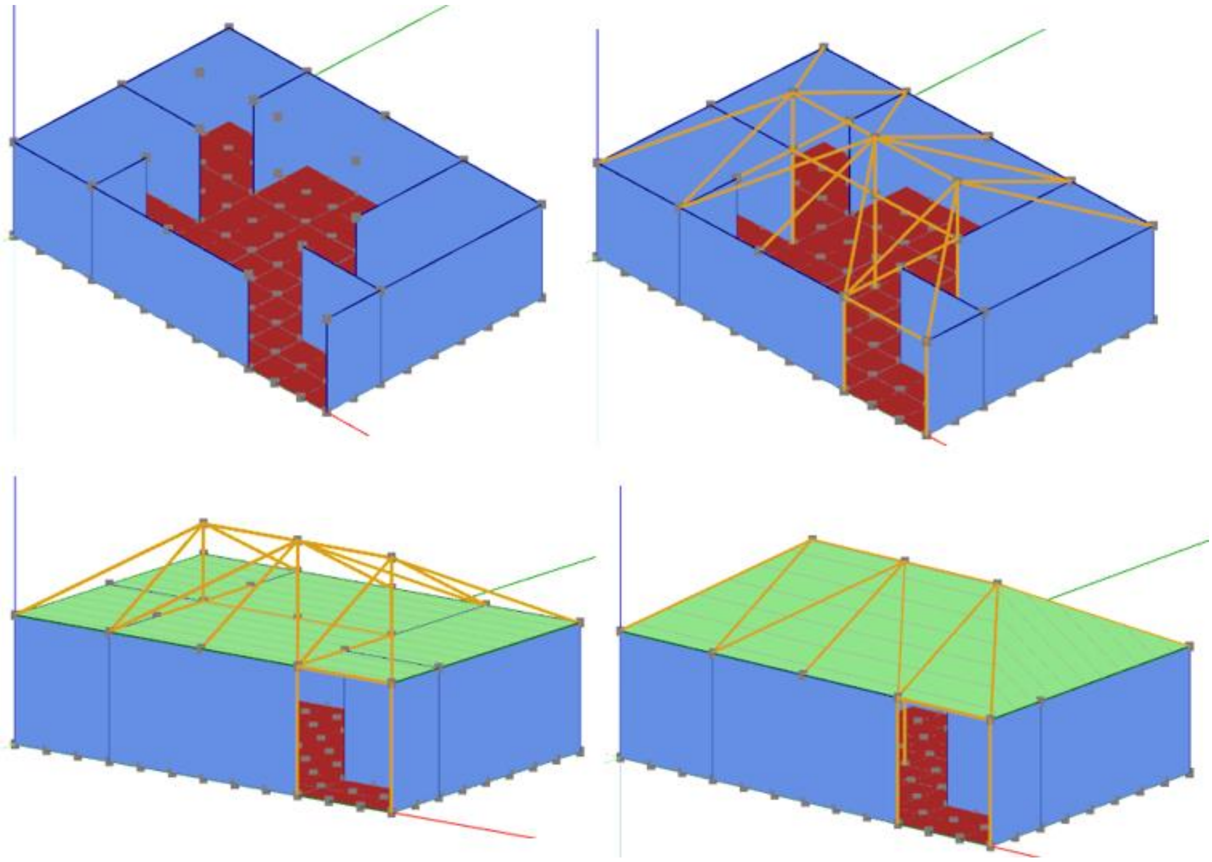


Figure 3-1: Southeast three-dimensional view of the cabin project

3.2 Wind load corresponding to avalanche load

In both cases, wind load was used corresponding to the avalanche load because the TimberTech software that was used for design of wooden cabin, not defining the avalanche load. Both cases are tested against a wind load at 20 KPa. in y-direction as shown in figure 3-2. Pressure impact on the wall constructions in both cases is the same as case A. in figure 3-3. It means that the pressure is the same everywhere on the wall structures.

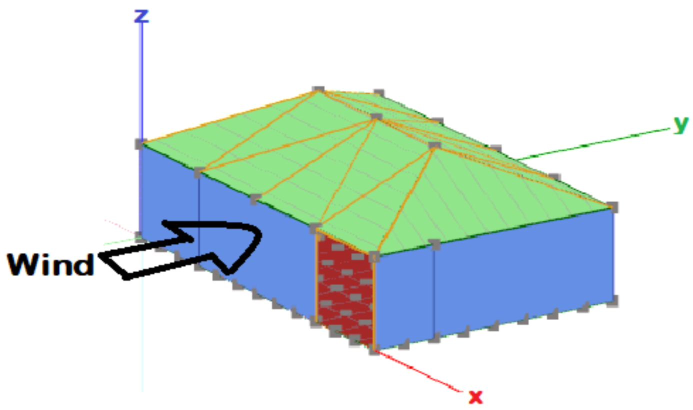


Figure 3-2: Wind load direction on the wall

3.2.1 The difference between wind load and avalanche load

3.2.1.1 Wind load

Wind impact varies with time and acts as pressure directly on the exterior surfaces of a closed structure. Wind also affects the internal surfaces due to leakage in the outer surfaces or in open structures. Wind pressure acts as perpendicular force on the surface of the structure. When the wind strikes over large areas of the construction, in addition, friction forces as tangential on the surface can be significant. The wind pressure that calculated according to NS-EN 1991-1-4 are characteristic values which are determined from basic wind velocity or wind velocity pressure values. The values of characteristic values have an annual probability of exceeding at 0,02, which corresponds to an average return period of 50 years (Norge, 2009).

The effect of wind on the construction depends on the size, shape and dynamic properties of the constructions. Figure 3.1 shows three cases (A, B and C) of wind pressure on the structures.

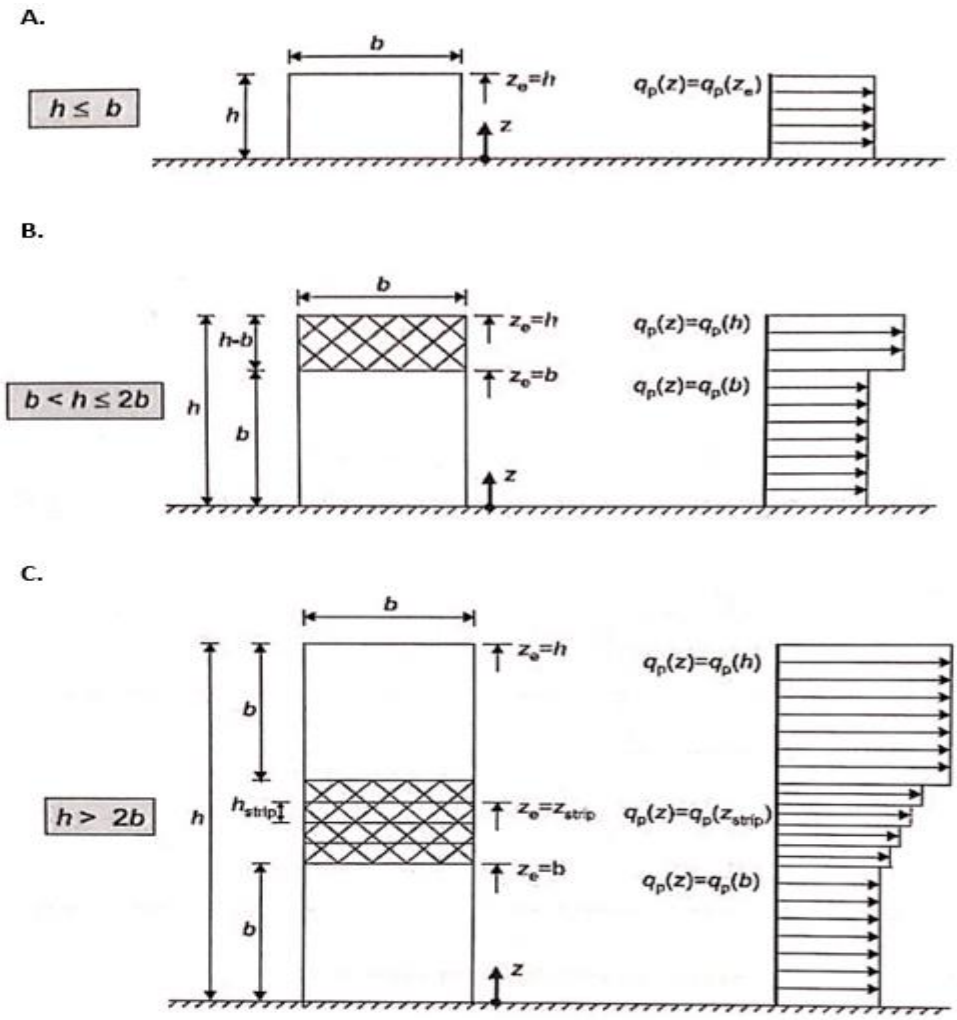


Figure 3-3: Three different cases of wind impact on structure which are dependent on height (h) and width (b) of the constructions (Norge, 2009)

Reference height (Z_e) for walls of rectangular buildings depends on the h/b ratio and are always the upper heights of the different parts of the walls.

Case A: a building with height (h) less than width (b) should be considered as a part.

Case B: a building with h greater than b but less than 2b, should be counted as two parts, a lower part from the ground up to the height equal to b and an upper part constituting the rest of the height.

Case C: a building with h greater than 2b should be considered to consist of several parts, a lower part from the ground up to the height equal to b, an upper part from the top down to a height difference equal to be and a middle area between the upper and lower part (Norge, 2009).

3.2.1.2 Avalanche load

How avalanche load effects on a construction was explained in chapter 2.3.4. It is shown that the avalanche load impact is greatest in the lower part of the construction where the heavy part of the avalanche lies, and it is called dense flow. There is almost the same pressure in the height interval where the dense flow hits. Over the dense flow part lies the fluidized flow part and powder part where the pressure decreases with the height of the construction in these two upper parts. Look at the figure 2-15 in chapter 2.3.4.

3.3 Linear elements characteristics (Beams and columns)

In both cases, linear elements were selected with the same characteristics. They have a cross section of 200x200, and the strength classes are selected of C24 and C50 materials.

The table 3-1 sets out the details concerning the cross section of every linear element.

Section name	Material	Width b [mm]	Height h [mm]	Area A [mm^2]	J_{y-y} [mm^4]	J_{z-z} [mm^4]
Section 200x200 C24	C 24	200	200	40000	1,33E8	1,33E8
200X200 C50	C 50	200	200	40000	1,33E8	1,33E8

Table 3-1: Characteristics of the linear elements

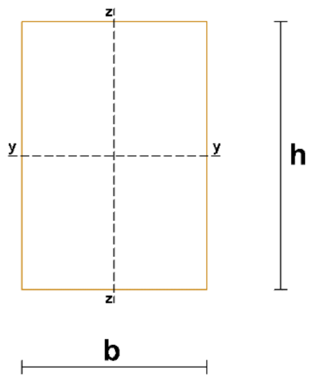


Figure 3-4: Geometric size of every timber cross section

3.4 Floor characteristics (roof)

Roof materials also have the same characteristics in the both cases. The figure 3-6 and table 3-2 show geometric characteristics of the floor with timber joists, where h_b is the cross-section height, b_b is the cross-section width and i_b is the joists spacing.

Section name	Material	Cross section height h_b [mm]	Cross section width b_b [mm]	Joists spacing i_b [mm]
Joists floor 160x200	C 30	200	160	700

Table 3-2: Characteristics of the floor with joist

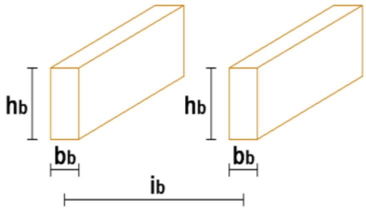


Figure 3-5: Geometric characteristics of the floor

And the naming of the roofs is shown on the figure 3-7.

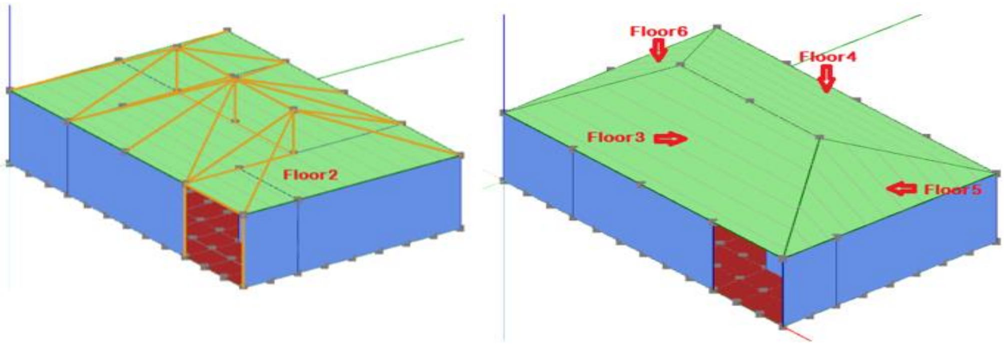


Figure 3-6: Name of the roofs

3.5 Wall characteristics

In the previous chapter, it was explained how important it is with proper placement of wall constructions in relation to rotation and stability of the building. It is also very important to have correct number and correct distribution of shear wall in building. The location of shear walls must be such that the structure can distribute the horizontal forces in all directions. The figure 3-8 shows the location and the names of the wall constructions of the designed cabin.

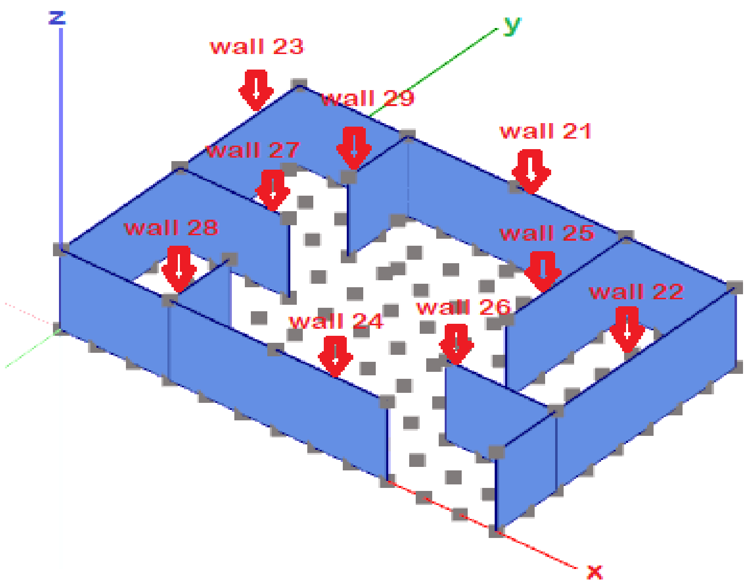


Figure 3-7: Location and the names of the wall constructions

Wall constructions consists of components such as sheeting boards, fasteners, studs (columns) and anchors such as angle brackets and hold down connection. The characteristics of wall constructions are different in each case.

3.5.1 Wall horizontal stiffness

The wall stiffness can be estimated considering the contributions of all the components. Figure 3-9 shows mechanical models for framed wall overall stiffness.

In the case of framed walls, the overall stiffness is calculated taking into account the contribution of the following components:

- Sheeting boards (k_s)
- Sheet-fasteners slip (k_c)
- Shear connections – angle brackets (k_a)
- Hold-down or tie-down (k_h)

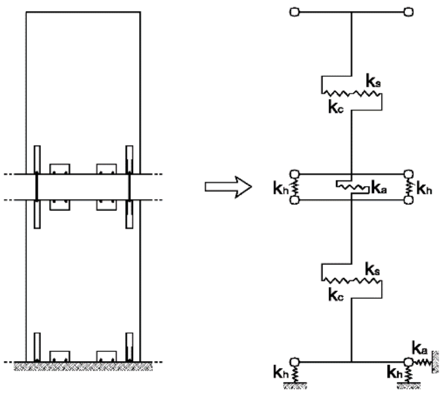


Figure 3-8: Mechanical models for framed wall overall stiffness

3.5.2 Wall elements and sign conventions

The walls, regardless of type, have the following sign conventions.

	Stress	Description	Unit of measure
In-plane stresses	**N**	Axial stress (per unit length)	kN/m
	$m_{3\text{-}3}$	Bending moment about local axis 3 (per unit length)	kNm/m
	v_2	Shear along local axis 2 (per unit length)	kN/m
Out-of-plane stresses (plate)	**$m_{2\text{-}2}$**	Bending moment about local axis 2 (per unit length)	kNm/m
	v_3	Shear along local axis 3 (per unit length)	kN/m

Table 3-3: Description of the sign convections

	Force	Description	Unit of measure
In-plane stresses	**N**	Total axial force	kN
	$M_{3\text{-}3}$	Bending moment about local axis 3	kNm
	V_2	Shear along local axis 2	kN
Out-of-plane stresses (plate)	**$M_{2\text{-}2}$**	Bending moment about local axis 3	kNm
	V_3	Shear along local axis 2	kN

Table 3-4: Description of the sign convections

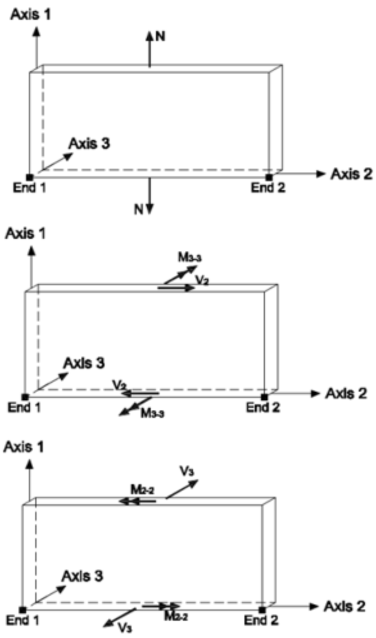


Figure 3-9: Sign conventions of the walls

3.6 Case 1

Here will be represented characteristics of wall components of case 1 and the values of load actions for each wall of case 1.

3.6.1 Sheeting boards

Table 3-5 shows the characteristics of the sheeting board and the figure 3-11shows dimensioning of sheeting board and fasteners spacing, where b_s is the sheeting boards width, $s_{c,b}$ is the spacing of fasteners along the perimeter of every sheet and $s_{c,i}$ is the spacing of internal fasteners.

Section name	Side	Material	Sheeting board thickness t_s [mm]	Sheeting boards width b_s [mm]	Frame-sheeting board fastener	Perimeter fasteners spacing $S_{c,b}$ [mm]	Internal fasteners spacing $S_{c,i}$ [mm]
Frame with OSB - 2 SIDES	1	OSB/2	12,5	1250	HH7 TX 2,8/3,1 X 65	100	200
Frame with OSB - 2 SIDES	2	OSB/2	12,5	1250	HH7 TX 2,8/3,1 X 65	100	200

Table 3-5: Characteristics of the sheeting board in case 1.

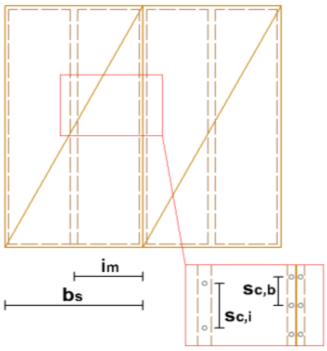


Figure 3-10: Dimensions of the sheeting boards and fasteners

3.6.2 Framed wall (Studs)

The table 3-6 shows the characteristics of the frame of each wall and the figure 3-12 shows frame geometric characteristics, where t is the thickness of the frame, h_b is the thickness of top and soleplates, $b_{s,int}$ is the width of the internal studs, $b_{s,ext}$ is the width of the external studs and i_m is the average studs spacing.

Section name	# sides with sheeting board	Material	Frame thickness t [mm]	Thickness of top and sole plates h_b [mm]	Width of the internal studs $b_{s,int}$ [mm]	Width of the external studs $b_{s,ext}$ [mm]	Average studs spacing i_m [mm]
Frame with OSB - 2 SIDES	2	C 24	160	100	100	100	625

Table 3-6: Characteristics of the frame (studs) of each wall in case 1

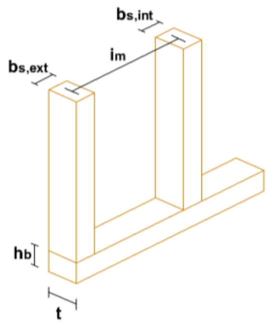


Figure 3-11: Dimension of the frame elements

The studs have a local reference system with respect to which stress/force components are shown. The sign convention adopted is shown in the figure 3-13 and their description is shown in table 3-7.

Force	Description	Unit of measure
N	Axial force	kN
$M_{3\text{-}3}$	Bending moment about local axis 3	kN m
V_2	Shear along local axis 2	kN
$M_{2\text{-}2}$	Bending moment about local axis 2	kN m
V_3	Shear along local axis 3	kN

Table 3-7 Description of the sign convections

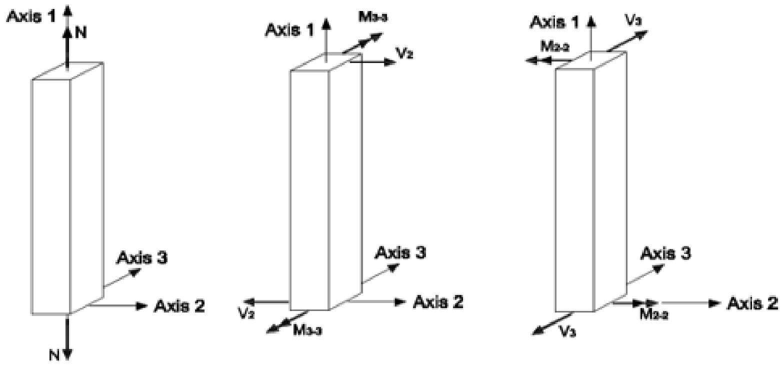


Figure 3-12: Sign conventions of the studs

3.6.3 Metal fasteners

Table 3-8 and figure 3-14 shows characteristics of metal fasteners that were used on wall constructions in case 1.

Produttore	Codice	Descr.	l [mm]	l_t [mm]	d [mm]	d_h [mm]	f_{uk} [MPa]
Rotho Blaas	HH731583	HH7 TX 2,8/3,1 X 65	65	45	2,8	4,3	600

Table 3-8: Characteristics of the nail in case 1

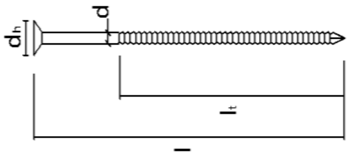


Figure 3-13: Nails with improved adhesion

Table 3-9 shows the resistances of the fasteners used to assemble the panels of the walls in case 1. Where $F_{ax,Rk}$ is the characteristic axial withdrawal capacity of the fastener. Rope effect limit is the rope effect limited to a percentage of Johansen part and $F_{v,Rk}$ is the characteristic load-carrying capacity per shear plane per fastener.

Section	Side	Fasteners	K_{ser} [N/mm]	Failure mode	$F_{ax,Rk}$ [N]	Rope effect limit	$F_{v,Rk}$
Frame with OSB - 2 SIDES	1	HH7 TX 2,8/3,1 X 65	918	d	228	50%	663
Frame with OSB - 2 SIDES	2	HH7 TX 2,8/3,1 X 65	918	d	228	50%	663

Table 3-9: Lateral load carrying capacity of the fasteners of case 1

3.6.4 Hold-down connections

Figures 3-15 and 3-16 represent hold-down connections that are used in case 1 to attach frame to the foundation.

Connection	SIM ANGOLARE AKR 135x3 L CE	
Nailing	Partial - 9 Fasteners	
Fastener type	Chiodo Anker 4x60	
Eccentricity coefficient k_t	1,00	
Anchor	M12 8.8	
Number of connections at each wall end	1	
Nailed connection resistance Rc,k	12,72	kN
Steel tensile strength Rs,k	17,21	kN
Anchor tensile strength Rt,k	67,00	kN
Anchor pull-out resistance Rpull,k	35,00	kN
Stiffness of the single connection	14887	N/mm

Figure 3-15: Characteristics of the hold-down connection of case 1

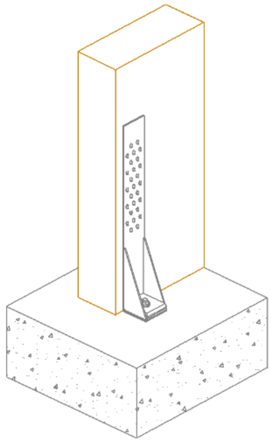


Figure 3-14: Graphical representation of a hold-down in a base connection

3.6.5 Angle brackets connections

Figures 3-17 and 3-18 represent timber-reinforced concrete connections that are used in case 1.

Connection	WKR 135	
Nailing	Total	
Fastener type	14 x Chiodi Anker LBA 4,0 X 60	
Eccentricity coefficient k_t	1,00	
Anchor	1 x M12 5.8	
Sides number	● 1 ○ 2	
Shear connections spacing i	500	mm
Connections shear resistance Ra,k	3,80	kN
Anchor shear strength Rp,k	21,00	kN
Stiffness of the single connection	24353	N/mm

Figure 3-17: Characteristics of the Angle brackets connections in case 1

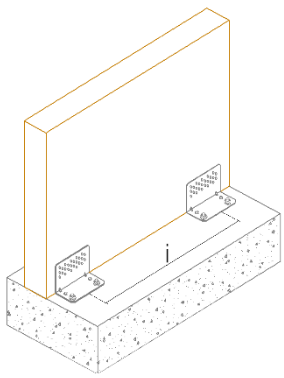


Figure 3-16: Graphical representation of the shear connection with angle brackets

3.6.6 Load acting on the walls

Figure 3-19 shows the external load direction and name of the walls and table 3-10 shows the permanent actions and the external load actions on the wall structures, where $g_{1,k}$ is the permanent action (self-weight), $g_{2,k}$ is the permanent action and $q_{wind,k}$ is the variable actions (wind load).

Wall name	Position	Load name	$g_{1,k}$ [kN/m^2]	$g_{2,k}$ [kN/m^2]	$q_{,wind,k}$ downwind [kN/m^2]	$q_{,wind,k}$ windward [kN/m^2]
Wall 21	External	External walls load1	0,39	0,6	-0,25	20
Wall 22	External	Wall load 2	0,4	0,6	-0,25	2
Wall 23	External	Wall load 2	0,4	0,6	-0,25	2
Wall 24	External	External walls load1	0,4	0,6	-0,25	20
Wall 25	External	External walls load1	0,41	0,6	-0,25	20
Wall 26	External	Wall load 2	0,41	0,6	-0,25	2
Wall 27	External	External walls load1	0,41	0,6	-0,25	20
Wall 28	External	Wall load 2	0,41	0,6	-0,25	2
Wall 29	External	Wall load 2	0,41	0,6	-0,25	2

Table 3-10: Loads on the wall structures in case 1

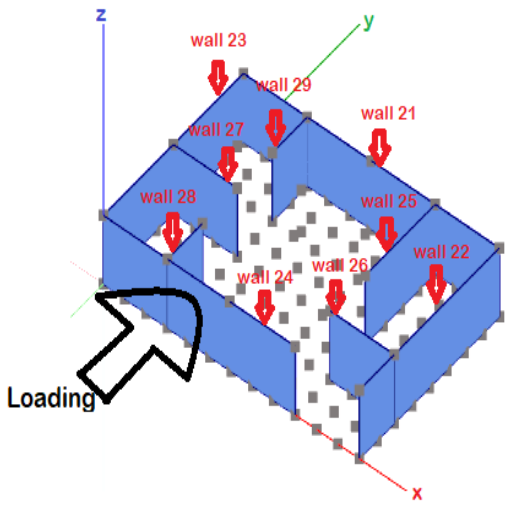


Figure 3-18: External load direction and the name of the walls

Table 3-11 shows the values of compression actions (N) parallel to the grain, bending actions (M2-2) and shear actions (V2) for each wall of case 1. It also shows that the walls 21, 24, 25 and 27 are most exposed to the bending load but the walls 22, 23, 25, 28 and 29 are most exposed to the shear force.

Wall name	Length [m]	Comb.	Dur.	N [kN]	M2-2 [kNm]	Comb.	V2 [kN]
Wall 21	12,01	ULS 36	Instantaneous	75,07	353,09	Horizontal ULS 1	17,34
Wall 22	8,01	ULS 36	Instantaneous	58,16	23,55	Horizontal ULS 4	180,93
Wall 23	8,01	ULS 36	Instantaneous	61,50	23,55	Horizontal ULS 4	185,61
Wall 24	9,01	ULS 36	Instantaneous	53,04	264,81	Horizontal ULS 1	13,53
Wall 25	3,00	ULS 36	Instantaneous	35,54	88,29	Horizontal ULS 1	3,52
Wall 26	4,01	ULS 36	Instantaneous	69,46	11,77	Horizontal ULS 4	79,66
Wall 27	3,00	ULS 36	Instantaneous	10,25	88,26	Horizontal ULS 1	3,46
Wall 28	2,00	ULS 36	Instantaneous	53,16	5,89	Horizontal ULS 4	32,26
Wall 29	2,00	ULS 36	Instantaneous	52,50	5,89	Horizontal ULS 4	32,26

Table 3-11: Values of compression, bending and shear actions for each wall in case 1

3.7 Case 2

Here will be represented characteristics of wall components of case 2 and the values of load actions for each wall.

3.7.1 Sheeting boards

Table 3-12 shows characteristics of the sheeting boards in case 2 and it is almost the same as in case 1 but the fasteners on them have different characteristics.

Section name	Side	Material	Sheeting board thickness t_s [mm]	Sheeting boards width b_s [mm]	Frame-sheeting board fastener	Perimeter fasteners spacing $S_{c,b}$ [mm]	Internal fasteners spacing i $S_{c,i}$ [mm]
Frame with OSB - 2 SIDES	1	OSB/2	12,5	1250	ELICOIDALE 5,0 x 80	100	200
Frame with OSB - 2 SIDES	2	OSB/2	12,5	1250	ELICOIDALE 5,0 x 80	100	200

Table 3-12: Characteristics of the sheeting board in case 2

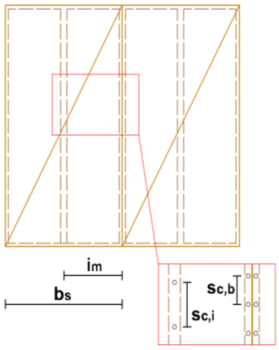


Figure 3-19: Dimensions of the sheeting boards and fasteners spacing

3.7.2 Framed wall (Studs)

The table 3-13 shows the characteristics of the frame and the figure 3-21 shows frame geometric characteristics of case 2.

Section name	# sides with sheeting board	Material	Frame thickness t [mm]	Thickness of top and sole plates h_b [mm]	Width of the internal studs $b_{s,int}$ [mm]	Width of the external studs $b_{s,ext}$ [mm]	Average studs spacing i_m [mm]
Frame with OSB - 2 SIDES	2	C 40	200	120	120	120	625

Table 3-13: Characteristics of the frame (studs) of each wall in case 2

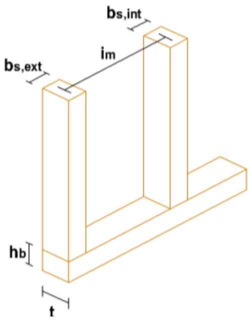


Figure 3-20: Dimension of the frame elements

3.7.3 Metal fasteners

The table 3-14 shows characteristics of the metal fasteners in case 2.

Produttore	Codice	Descr.	L [mm]	l_t [mm]	d [mm]	d_h [mm]	f_{uk} [MPa]
Würth	0478 5 80	ELICOIDALE 5,0 x 80	80	60	5	6,5	600

Table 3-14: Characteristics of the nails in case 2

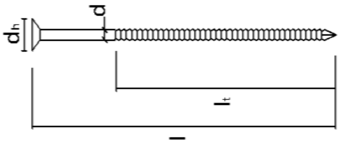


Figure 3-21: Nails with improved adhesion

And the table 3-15 shows the lateral load carrying capacity of the fasteners in case 2.

Section	Side	Fasteners	K_{ser} [N/mm]	Failure mode	$F_{ax,Rk}$ [N]	Rope effect limit	$F_{v,Rk}$
Frame with OSB - 2 SIDES	1	ELICOIDALE 5,0 x 80	1613	d	520	50%	1383
Frame with OSB - 2 SIDES	2	ELICOIDALE 5,0 x 80	1613	d	520	50%	1383

Table 3-15: Lateral-load carrying capacity of the fasteners in case 2

3.7.4 Hold-down connections

Figures 3-23 and 3-24 represents hold-down connections that are used in case 2.

Connection	WHT 340	
Nailing	Partial - 14 Fasteners	
Fastener type	Chiodi Anker LBA 4,0 X 40	
Eccentricity coefficient k_t	1,00	
Anchor	M16 5.8	
Number of connections at each wall end	1	
Nailed connection resistance Rc,k	21,98	kN
Steel tensile strength Rs,k	42,00	kN
Anchor tensile strength Rt,k	70,65	kN
Anchor pull-out resistance Rpull,k	35,94	kN
Stiffness of the single connection	19977	N/mm

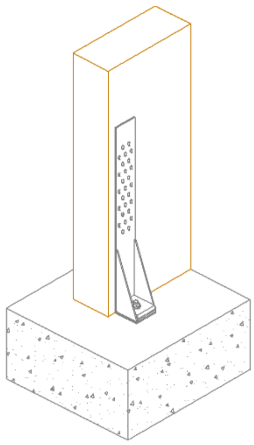


Figure 3-22: Graphical representation of a hold-down in a base connection

Figure 3-23: Characteristics of the hold-down connections in case 2

3.7.5 Angle brackets connections

Figures 3-25 and 3-26 represents timber-reinforced concrete connections that are used in case 2.

Connection	Titan TCN 200
Nailing	Total
Fastener type	30 x Chiodi Anker LBA 4,0 X 60
Eccentricity coefficient k_t	0,97
Anchor	2 x M12 5.8
Sides number	1 / 2
Shear connections spacing i	1500 mm
Connections shear resistance Ra,k	22,10 kN
Anchor shear strength Rp,k	21,00 kN
Stiffness of the single connection	52186 N/mm

Figure 3-25: Characteristics of the Angle brackets connection in case 2

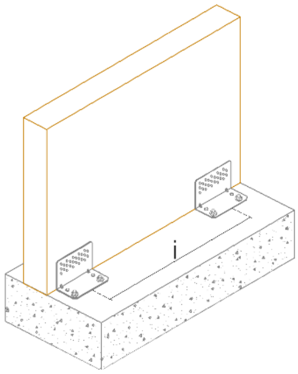


Figure 3-24: Graphical representation of the shear connection with angle brackets

3.7.6 Load acting on the walls

Table 3-16 shows the permanent loads and the external load actions on the wall structures in case 2. External load acting in both cases are the same but the self-weight of the walls in case 2 is higher than the self-weight of the walls in case 1 and it is due to walls components with different characteristics in each case. Therefore, the bending and shear effect are also the same in both cases but the axial force (compression) is different in each case which is shown in table 3-17.

Wall name	Position	Load name	$g_{1,k}$ [kN/m²]	$g_{2,k}$ [kN/m²]	$q_{,wind,k}$ downwind [kN/m²]	$q_{,wind,k}$ windward [kN/m²]
Wall 21	External	External walls load1	0,54	0,6	-0,25	20
Wall 22	External	Wall load 2	0,55	0,6	-0,25	2
Wall 23	External	Wall load 2	0,55	0,6	-0,25	2
Wall 24	External	External walls load1	0,56	0,6	-0,25	20
Wall 25	External	External walls load1	0,58	0,6	-0,25	20
Wall 26	External	Wall load 2	0,58	0,6	-0,25	2
Wall 27	External	External walls load1	0,58	0,6	-0,25	20
Wall 28	External	Wall load 2	0,58	0,6	-0,25	2
Wall 29	External	Wall load 2	0,58	0,6	-0,25	2

Table 3-16: Loads on the wall structures in case 2

Table 3-17 shows the values of compression actions parallel to the grain, bending actions and shear actions for each wall in case 2.

Wall name	**Length [m]**	**Comb.**	**Dur.**	**N [kN]**	**M2-2 [kNm]**	**Comb.**	**V2 [kN]**
Wall 21	12,01	ULS 36	Instantaneous	81,17	353,09	Horizontal ULS 1	17,17
Wall 22	8,01	ULS 36	Instantaneous	62,35	23,55	Horizontal ULS 4	179,16
Wall 23	8,01	ULS 36	Instantaneous	65,70	23,55	Horizontal ULS 4	182,03
Wall 24	9,01	ULS 36	Instantaneous	57,82	264,81	Horizontal ULS 1	13,31
Wall 25	3,00	ULS 36	Instantaneous	37,25	88,29	Horizontal ULS 1	3,35
Wall 26	4,01	ULS 36	Instantaneous	71,75	11,77	Horizontal ULS 4	79,19
Wall 27	3,00	ULS 36	Instantaneous	11,96	88,26	Horizontal ULS 1	4,02
Wall 28	2,00	ULS 36	Instantaneous	54,29	5,89	Horizontal ULS 4	35,17
Wall 29	2,00	ULS 36	Instantaneous	53,63	5,89	Horizontal ULS 4	35,17

Table 3-17: Values of compression, bending and shear actions for each wall in case 2

4 Result

Here, results will be given as check of wall components against the horizontal load. As was explained in the method, both cases have the same characteristics of roof and linear elements, but characteristics of the wall constructions are different in each case. So, the focus will be on wall components that were exposed to horizontal load.

In order to find out the capacity of wall constructions against the horizontal load, check of the following wall components are given.

- Studs (Columns)
- The shear strength of framed shear wall
  - Lateral load-carrying capacity of metal fasteners
  - Shear strength of sheeting board
- Foundation Connections
  - Hold Down – Connections at the base of the structure
  - Angle brackets with anchors- timber to concrete shear connections

4.1 Studs (columns)

Studs that be subjected to compression and bending should satisfy the following expression:

$$\frac{\sigma_{c,0,d}}{k_c \cdot f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1 \tag{4.1}$$

These elements (stud or column in a sheathed wall) are braced against buckling in the in-plane direction therefore checks are performed only in the orthogonal direction. The following tables summarizes the stability checks for the studs of the framed walls in each case.

Section: Type of cross-section of the stud.

h_{stud}: Stud height

A_{stud}: Cross sectional area of the stud

J_{stud}: Cross sectional moment of inertia of the stud

Comb.: The most severe load combination

k_{mod}: Modification factor taking into account the effect of the duration of load and moisture content

γ_M: Partial factor for a material property

$f_{c,0,k}$: Characteristic compressive strength along the grain

$f_{m,k}$: Design bending strength

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

$\sigma_{m,d}$: Design bending stress about the principal axis

4.1.1 Check of the studs in case 1

Table 4-1 shows the check of the studs for buckling. It shows that the walls 21, 24, 25 and 27 do not satisfy the requirement but the walls 22, 23,26, 28 and 29 satisfy the requirement.

Wall name	Section	Stud	h_{stud} [m]	A_{stud} [mm^2]	J_{stud} [mm^4]	kc_{stud}	Comb.	Service Class	k_{mod}	γ_M	$f_{c,0,k}$	$f_{m,k}$	N [kN]	$\sigma_{c,0,d}$ [MPa]	$\sigma_{m,d}$ [MPa]	Check
Wall 21	Frame with OSB - 2 SIDES	Internal	2,8	16000	3,41E7	0,67	ULS 36	1	1,1	1,25	21,00	24,00	9,24	0,58	43,07	207%
Wall 21	Frame with OSB - 2 SIDES	External	2,8	16000	3,41E7	0,67	ULS 36	1	1,1	1,25	21,00	24,00	4,86	0,30	27,91	133%
Wall 22	Frame with OSB - 2 SIDES	Internal	2,8	16000	3,41E7	0,67	ULS 36	1	1,1	1,25	21,00	24,00	5,26	0,33	4,31	23%
Wall 22	Frame with OSB - 2 SIDES	External	2,8	16000	3,41E7	0,67	ULS 36	1	1,1	1,25	21,00	24,00	5,19	0,32	2,33	14%
Wall 23	Frame with OSB - 2 SIDES	Internal	2,8	16000	3,41E7	0,67	ULS 36	1	1,1	1,25	21,00	24,00	5,24	0,33	4,31	23%
Wall 23	Frame with OSB - 2 SIDES	External	2,8	16000	3,41E7	0,67	ULS 36	1	1,1	1,25	21,00	24,00	5,19	0,32	2,33	14%
Wall 24	Frame with OSB - 2 SIDES	Internal	2,8	16000	3,41E7	0,67	ULS 36	1	1,1	1,25	21,00	24,00	9,12	0,57	43,07	207%
Wall 24	Frame with OSB - 2 SIDES	External	2,8	16000	3,41E7	0,67	ULS 36	1	1,1	1,25	21,00	24,00	4,73	0,30	23,26	111%
Wall 25	Frame with OSB - 2 SIDES	Internal	2,8	16000	3,41E7	0,67	ULS 36	1	1,1	1,25	21,00	24,00	12,54	0,78	43,07	208%
Wall 25	Frame with OSB - 2 SIDES	External	2,8	16000	3,41E7	0,67	ULS 36	1	1,1	1,25	21,00	24,00	1,15	0,07	23,26	110%
Wall 26	Frame with OSB - 2 SIDES	Internal	2,8	16000	3,41E7	0,67	ULS 36	1	1,1	1,25	21,00	24,00	9,83	0,61	4,31	25%
Wall 26	Frame with OSB - 2 SIDES	External	2,8	16000	3,41E7	0,67	ULS 36	1	1,1	1,25	21,00	24,00	2,91	0,18	2,33	12%
Wall 27	Frame with OSB - 2 SIDES	Internal	2,8	16000	3,41E7	0,67	ULS 36	1	1,1	1,25	21,00	24,00	2,13	0,13	43,07	203%
Wall 27	Frame with OSB - 2 SIDES	External	2,8	16000	3,41E7	0,67	ULS 36	1	1,1	1,25	21,00	24,00	1,15	0,07	23,26	110%
Wall 28	Frame with OSB - 2 SIDES	Internal	2,8	16000	3,41E7	0,67	ULS 36	1	1,1	1,25	21,00	24,00	10,19	0,64	4,31	25%
Wall 28	Frame with OSB - 2 SIDES	External	2,8	16000	3,41E7	0,67	ULS 36	1	1,1	1,25	21,00	24,00	18,75	1,17	2,76	22%
Wall 29	Frame with OSB - 2 SIDES	Internal	2,8	16000	3,41E7	0,67	ULS 36	1	1,1	1,25	21,00	24,00	9,85	0,62	4,31	25%
Wall 29	Frame with OSB - 2 SIDES	External	2,8	16000	3,41E7	0,67	ULS 36	1	1,1	1,25	21,00	24,00	18,35	1,15	2,76	22%

Table 4-1 Stability check of the frame (studs) of each wall in project 1

4.1.2 Check of the studs in case 2

Table 4-3 shows also the check of the studs for buckling of case 2, but in this case all walls satisfy the requirement.

Wall name	Section	Stud	h_{stud} [m]	A_{stud} [mm^2]	J_{stud} [mm^4]	kc_{stud}	Comb.	Service Class	k_{mod}	γ_M	$f_{c,0,k}$	$f_{m,k}$	N [kN]	$\sigma_{c,0,d}$ [MPa]	$\sigma_{m,d}$ [MPa]	Check
Wall 21	Frame with OSB - 2 SIDES	Internal	2,8	24000	8,00E7	0,81	ULS 36	1	1,1	1,25	27,00	40,00	9,56	0,40	22,97	67%
Wall 21	Frame with OSB - 2 SIDES	External	2,8	24000	8,00E7	0,81	ULS 36	1	1,1	1,25	27,00	40,00	5,09	0,21	15,07	43%
Wall 22	Frame with OSB - 2 SIDES	Internal	2,8	24000	8,00E7	0,81	ULS 36	1	1,1	1,25	27,00	40,00	5,59	0,23	2,30	8%
Wall 22	Frame with OSB - 2 SIDES	External	2,8	24000	8,00E7	0,81	ULS 36	1	1,1	1,25	27,00	40,00	5,40	0,22	1,26	5%
Wall 23	Frame with OSB - 2 SIDES	Internal	2,8	24000	8,00E7	0,81	ULS 36	1	1,1	1,25	27,00	40,00	5,57	0,23	2,30	8%
Wall 23	Frame with OSB - 2 SIDES	External	2,8	24000	8,00E7	0,81	ULS 36	1	1,1	1,25	27,00	40,00	5,40	0,22	1,26	5%
Wall 24	Frame with OSB - 2 SIDES	Internal	2,8	24000	8,00E7	0,81	ULS 36	1	1,1	1,25	27,00	40,00	9,46	0,39	22,97	67%
Wall 24	Frame with OSB - 2 SIDES	External	2,8	24000	8,00E7	0,81	ULS 36	1	1,1	1,25	27,00	40,00	4,93	0,21	12,59	36%
Wall 25	Frame with OSB - 2 SIDES	Internal	2,8	24000	8,00E7	0,81	ULS 36	1	1,1	1,25	27,00	40,00	12,89	0,54	22,97	67%
Wall 25	Frame with OSB - 2 SIDES	External	2,8	24000	8,00E7	0,81	ULS 36	1	1,1	1,25	27,00	40,00	1,36	0,06	12,59	36%
Wall 26	Frame with OSB - 2 SIDES	Internal	2,8	24000	8,00E7	0,81	ULS 36	1	1,1	1,25	27,00	40,00	10,19	0,42	2,30	9%
Wall 26	Frame with OSB - 2 SIDES	External	2,8	24000	8,00E7	0,81	ULS 36	1	1,1	1,25	27,00	40,00	3,18	0,13	1,26	4%
Wall 27	Frame with OSB - 2 SIDES	Internal	2,8	24000	8,00E7	0,81	ULS 36	1	1,1	1,25	27,00	40,00	2,49	0,10	22,97	65%
Wall 27	Frame with OSB - 2 SIDES	External	2,8	24000	8,00E7	0,81	ULS 36	1	1,1	1,25	27,00	40,00	1,36	0,06	12,59	36%
Wall 28	Frame with OSB - 2 SIDES	Internal	2,8	24000	8,00E7	0,81	ULS 36	1	1,1	1,25	27,00	40,00	10,54	0,44	2,30	9%
Wall 28	Frame with OSB - 2 SIDES	External	2,8	24000	8,00E7	0,81	ULS 36	1	1,1	1,25	27,00	40,00	19,07	0,79	1,49	8%
Wall 29	Frame with OSB - 2 SIDES	Internal	2,8	24000	8,00E7	0,81	ULS 36	1	1,1	1,25	27,00	40,00	10,20	0,43	2,30	9%
Wall 29	Frame with OSB - 2 SIDES	External	2,8	24000	8,00E7	0,81	ULS 36	1	1,1	1,25	27,00	40,00	18,66	0,78	1,49	8%

Table 4-2: Stability check of the frame (studs) of each wall in case 2

4.2 Metal fasteners

Calculation method for capacity of metal fasteners connection is explained in chapter 2.2.3. Tables 4-3 and 4-4 shows check of the bearing capacity of the walls related to the lateral load-carrying capacity of metal fasteners of each case.

$F_{v,Ed}$: Design value of the shear force acting on the fasteners

k_{mod}: The modification factor taking into account the effect of the duration of load and moisture content

γ_M: Partial factor

$F_{v,Rd}$: Design value of the fasteners resistance

Where a connection is constituted of two timber elements having different time-dependent behavior, the calculation of the design load-carrying capacity should be made with the following modification factor $k_{mod,conn,i}$:

$$k_{mod,conn,i} = \sqrt{k_{mod,studs} \cdot k_{mod,side\ i}} \tag{4.2}$$

4.2.1 Check of the metal fasteners in case 1

Table 4-3 shows the check of the metal fasteners for shear failure. It shows that the fasteners of walls 22, 23, 26, 28 and 29 failed to resist the shear force while the fasteners of walls 21, 24, 25 and 27 managed well to resist the shear force.

Wall name	**Section**	**Comb.**	**Service class**	**Dur.**	$k_{mod\ studs}$	k_{mod1}	k_{mod2}	γ_M	$F_{v,Rd}$ **[kN]**	$F_{v,Ed}$ **[kN]**	**Check**
Wall 21	Frame with OSB - 2 SIDES	Horizontal ULS 1	1	Instantaneous	1,1	1,1	1,1	1,3	140,80	17,34	12%
Wall 22	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,1	1,3	90,16	180,93	201 %
Wall 23	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,1	1,3	90,16	185,61	206 %
Wall 24	Frame with OSB - 2 SIDES	Horizontal ULS 1	1	Instantaneous	1,1	1,1	1,1	1,3	105,19	13,53	13%
Wall 25	Frame with OSB - 2 SIDES	Horizontal ULS 1	1	Instantaneous	1,1	1,1	1,1	1,3	30,05	3,52	12%
Wall 26	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,1	1,3	45,08	79,66	177 %
Wall 27	Frame with OSB - 2 SIDES	Horizontal ULS 1	1	Instantaneous	1,1	1,1	1,1	1,3	30,05	3,46	12%
Wall 28	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,1	1,3	20,47	32,26	158 %
Wall 29	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,1	1,3	20,47	32,26	158 %

Table 4-3: Capacity check of the metal fasteners in case 1

4.2.2 Check of the metal fasteners in case 2.

Table 4-4 shows the check of the metal fasteners for shear failure of case 2, but in this case shows that the fasteners of all walls resist the shear force.

Wall name	Section	Comb.	Service class	Dur.	$k_{mod\ studs}$	$k_{mod\ 1}$	$k_{mod\ 2}$	γ_M	$F_{v,Rd}$ [kN]	$F_{v,Ed}$ [kN]	Check
Wall 21	Frame with OSB - 2 SIDES	Horizontal ULS 1	1	Instantaneous	1,1	1,1	1,1	1,3	293,68	17,17	6%
Wall 22	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,1	1,3	188,06	179,16	95%
Wall 23	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,1	1,3	188,06	182,03	97%
Wall 24	Frame with OSB - 2 SIDES	Horizontal ULS 1	1	Instantaneous	1,1	1,1	1,1	1,3	219,41	13,31	6%
Wall 25	Frame with OSB - 2 SIDES	Horizontal ULS 1	1	Instantaneous	1,1	1,1	1,1	1,3	62,69	3,35	5%
Wall 26	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,1	1,3	94,03	79,19	84%
Wall 27	Frame with OSB - 2 SIDES	Horizontal ULS 1	1	Instantaneous	1,1	1,1	1,1	1,3	62,69	4,02	6%
Wall 28	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,1	1,3	42,69	35,17	82%
Wall 29	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,1	1,3	42,69	35,17	82%

Table 4-4: Capacity check of the metal fasteners in case 2

4.3 Sheeting boards

For a wall made up of several wall panels, the design racking load-carrying capacity of a wall should be calculated from:

$$F_{v,Rd} = \sum_i F_{i,v,Rd} \tag{4.3}$$

Where $F_{i,v,Rd}$ is the design racking load-carrying capacity of the wall panel in accordance with 9.2.4.2(3) and 9.2.4.2(5) of EN 1995-1-1. The load-carrying capacity of a sheeting panel $F_{i,v,Rd}$ is:

$$F_{i,j,v,Rd} = f_{j,v,d} \cdot b_i \cdot t_{i,j} \tag{4.4}$$

Where $F_{i,j,v,Rd}$ is the shear strength of the single sheet, $f_{j,v,d}$ is the shear strength of the single sheeting board, b_i is the panel width and $t_{i,j}$ is the thickness of the sheeting board.

Tables 4-5 and 4-6 summarizes the shear strength capacity of the wall panels of case 1 and case 2. The check of shear strength of sheeting boards in both cases is almost the same because of the same sheeting boards characteristics.

4.3.1 Shear strength of the sheeting boards in case1

Table 4-5 shows the check of the sheeting boards for shear failure. It shows that the sheeting boards of all the walls satisfy the requirement.

Wall name	Section	Comb.	Service Class	Dur.	k_{mod} side 1	k_{mod} side 2	γ_M	γ_{M2}	$F_{v,Rd}$ [kN]	$F_{v,Ed}$ [kN]	Check
Wall 21	Frame with OSB - 2 SIDES	Horizontal ULS 1	1	Instantaneous	1,1	1,1	1,3	1,3	1727,59	17,34	1%
Wall 22	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,3	1,3	1078,85	180,93	17%
Wall 23	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,3	1,3	1078,85	185,61	17%
Wall 24	Frame with OSB - 2 SIDES	Horizontal ULS 1	1	Instantaneous	1,1	1,1	1,3	1,3	1258,65	13,53	1%
Wall 25	Frame with OSB - 2 SIDES	Horizontal ULS 1	1	Instantaneous	1,1	1,1	1,3	1,3	359,62	3,52	1%
Wall 26	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,3	1,3	539,42	79,66	15%
Wall 27	Frame with OSB - 2 SIDES	Horizontal ULS 1	1	Instantaneous	1,1	1,1	1,3	1,3	359,62	3,46	1%
Wall 28	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,3	1,3	287,98	32,26	11%
Wall 29	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,3	1,3	287,98	32,26	11%

Table 4-5: Shear strength check of the sheeting boards in case 1

4.3.2 Shear strength of the sheeting boards in case 2

Table 4-6 shows the check of the sheeting boards for shear failure. The results in this case is almost the same as the result in case 1.

Wall name	Section	Comb.	Service Class	Dur.	k_{mod} side 1	k_{mod} side 2	γ_M	γ_{M2}	$F_{v,Rd}$ [kN]	$F_{v,Ed}$ [kN]	Check
Wall 21	Frame with OSB - 2 SIDES	Horizontal ULS 1	1	Instantaneous	1,1	1,1	1,3	1,3	1727,59	17,17	1%
Wall 22	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,3	1,3	1078,85	179,16	17%
Wall 23	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,3	1,3	1078,85	182,03	17%
Wall 24	Frame with OSB - 2 SIDES	Horizontal ULS 1	1	Instantaneous	1,1	1,1	1,3	1,3	1258,65	13,31	1%
Wall 25	Frame with OSB - 2 SIDES	Horizontal ULS 1	1	Instantaneous	1,1	1,1	1,3	1,3	359,62	3,35	1%
Wall 26	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,3	1,3	539,42	79,19	15%
Wall 27	Frame with OSB - 2 SIDES	Horizontal ULS 1	1	Instantaneous	1,1	1,1	1,3	1,3	359,62	4,02	1%
Wall 28	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,3	1,3	287,98	35,17	12%
Wall 29	Frame with OSB - 2 SIDES	Horizontal ULS 4	1	Instantaneous	1,1	1,1	1,3	1,3	287,98	35,17	12%

Table 4-6: Shear strength check of the sheeting boards in case 2

4.4 Hold-down connection

Calculation method of capacity for hold-down connections against the various failure modes are explained in chapter 2.2.3, so that the following expression should be satisfied.

$$T_{a,d} \leq R_{a,d} = min.\left(R_{c,d}; R_{s,d}\right) \tag{4.5}$$

$$T_{p,d} \leq R_{p,d} = min.\left(R_{t,d}; R_{pull,d}\right) \tag{4.6}$$

The checks of both cases are summarized in the following tables which shows the characteristic values of the resistances associated with collapse of the various components.

Name: Name of the connection in which the hold-down is used.

Comb.: The most severe combination of load.

$T_{a,d}$: Design value of the tensile force acting on the hold down.

$T_{p,d}$: Design value of the tensile force acting on the anchor.

k_{mod}: is the modification factor taking into account the effect of the duration of load and moisture content.

γ_M: is the partial factor.

$R_{a,d}$: Design value of the hold down resistance, assumed to be the lower of the values of the design resistance of all the failure mechanisms associated with it.

$R_{p,d}$: Design value of the anchor resistance, assumed to be the lower of the values of the design resistance of all the failure mechanisms associated with it.

4.4.1 Check of the hold-down connection in case 1

Table 4-7 shows the check of the hold-down connection for shear failure. It shows that the hold-down connections of walls 21, 24, 25 and 27 are almost unaffected of shear force and the hold-down connections of walls 22, 23, 26, 28 and 29 failed to resist the shear force.

Wall name	Connection name	Comb.	Service class	$T_{a,d}$ [kN]	$R_{c,k,dens}$ [kN]	$R_{s,k}$ [kN]	k_{mod}	γ_M	γ_{M2}	$R_{a,d}$ [kN]	$T_{p,d}$ [kN]	$R_{t,k}$ [kN]	$R_{pull,k}$ [kN]	γ_{Mc}	$R_{p,d}$ [kN]	Failure mode	Check
Wall 21	Connection 1	Horizontal ULS 1	1	0,00	12,72	17,21	1,1	1,3	1,25	-	0,00	67	35	1,5	-	-	0%
Wall 22	Connection 1	Horizontal ULS 4	1	36,71	12,72	17,21	1,1	1,3	1,25	10,76	36,71	67	35	1,5	23,33	Nailed connection	341%
Wall 23	Connection 1	Horizontal ULS 4	1	36,73	12,72	17,21	1,1	1,3	1,25	10,76	36,73	67	35	1,5	23,33	Nailed connection	341%
Wall 24	Connection 1	Horizontal ULS 1	1	0,00	12,72	17,21	1,1	1,3	1,25	-	0,00	67	35	1,5	-	-	0%
Wall 25	Connection 1	Horizontal ULS 1	1	0,00	12,72	17,21	1,1	1,3	1,25	-	0,00	67	35	1,5	-	-	0%
Wall 26	Connection 1	Horizontal ULS 4	1	23,01	12,72	17,21	1,1	1,3	1,25	10,76	23,01	67	35	1,5	23,33	Nailed connection	214%
Wall 27	Connection 1	Horizontal ULS 1	1	0,00	12,72	17,21	1,1	1,3	1,25	-	0,00	67	35	1,5	-	-	0%
Wall 28	Connection 1	Horizontal ULS 4	1	19,88	12,72	17,21	1,1	1,3	1,25	10,76	19,88	67	35	1,5	23,33	Nailed connection	185%
Wall 29	Connection 1	Horizontal ULS 4	1	20,20	12,72	17,21	1,1	1,3	1,25	10,76	20,20	67	35	1,5	23,33	Nailed connection	188%

Table 4-7: Check of hold-down connections resistance in case 1

4.4.2 Check of the hold-down connection in case 2

Table 4-8 shows the check of the hold-down connection for shear failure, but in this case the hold-down connections of walls 22, 23, 26, 28 and 29 managed well to resist the shear force. And the hold-down connections of walls 21, 24, 25 and 27 are the same as in case 1 where they are unaffected of shear force.

Wall name	Connection name	Comb.	Service class	$T_{a,d}$ [kN]	$R_{c,k,dens}$ [kN]	$R_{s,k}$ [kN]	k_{mod}	γ_M	γ_{M2}	$R_{a,d}$ [kN]	$T_{p,d}$ [kN]	$R_{t,k}$ [kN]	$R_{pull,k}$ [kN]	γ_{Mc}	$R_{p,d}$ [kN]	Failure mode	Check
Wall 21	Ground connection - hold down - bracket6	Horizontal ULS 1	1	0,00	21,98	42	1,1	1,3	1,25	-	0,00	70,65	35,94	2,1	-	-	0%
Wall 22	Ground connection - hold down - bracket8	Horizontal ULS 4	1	11,45	21,98	42	1,1	1,3	1,25	18,60	11,45	70,65	35,94	2,1	17,11	Anchor pullout	67%
Wall 23	Ground connection - hold down - bracket8	Horizontal ULS 4	1	11,24	21,98	42	1,1	1,3	1,25	18,60	11,24	70,65	35,94	2,1	17,11	Anchor pullout	66%
Wall 24	Ground connection - hold down - bracket6	Horizontal ULS 1	1	0,00	21,98	42	1,1	1,3	1,25	-	0,00	70,65	35,94	2,1	-	-	0%
Wall 25	Ground connection - hold down - bracket4	Horizontal ULS 1	1	0,00	21,98	42	1,1	1,3	1,25	-	0,00	70,65	35,94	2,1	-	-	0%
Wall 26	Ground connection - hold down - bracket3	Horizontal ULS 4	1	10,86	21,98	42	1,1	1,3	1,25	18,60	10,86	70,65	35,94	2,1	17,11	Anchor pullout	63%
Wall 27	Ground connection - hold down - bracket5	Horizontal ULS 1	1	0,00	21,98	42	1,1	1,3	1,25	-	0,00	70,65	35,94	2,1	-	-	0%
Wall 28	Ground connection - hold down - bracket9	Horizontal ULS 4	1	11,74	21,98	42	1,1	1,3	1,25	18,60	11,74	70,65	35,94	2,1	17,11	Anchor pullout	69%
Wall 29	Ground connection - hold down - bracket9	Horizontal ULS 4	1	11,90	21,98	42	1,1	1,3	1,25	18,60	11,90	70,65	35,94	2,1	17,11	Anchor pullout	70%

Table 4-8: check of hold-down connections resistance in case 2

4.5 Angle brackets Connection

Design for shear resistance of an angle bracket should satisfy the following expressions:

$$V_{a,d} \leq R_{a,d} \tag{4.7}$$

$$V_{p,d} \leq R_{p,d} \tag{4.8}$$

The check of angle brackets resistance for case 1 and case 2 are summarized in tables 4-9 and 4-10.

$R_{a,d}$: The design value of the shear bearing capacity of the angle bracket

$R_{p,d}$: The design value of the shear strength of the anchor is evaluated

Name: Name of the connection in which the angle bracket is used.

Comb.: The most severe combination of load.

$V_{a,d}$: Shear force acting on the angle bracket.

$V_{p,d}$: Shear force acting on the most stressed anchor.

k_{mod}: Modification factor taking into account the effect of the duration of load and moisture content.

γ_M: Partial safety factor.

4.5.1 Resistance check of angle brackets in case 1

Table 4-9 shows the check of the angle brackets connections for shear failure. It shows that the angle brackets of walls 22, 23, 26, 28 and 29 failed to resist the shear force while the angel brackets of walls 21, 24, 25 and 27 managed well to resist the shear force. The last column of the table shows the resistance check of the anchor where the anchor of all walls satisfies the requirement.

Wall name	Connection name	Comb.	Service class	$V_{a,d}$ [kN]	$R_{a,k,dens}$ [kN]	k_{mod}	γ_M	$R_{a,d}$ [kN]	Check – angle brackets	$V_{p,d}$ [kN]	$R_{p,k}$ [kN]	$\gamma_{Ms,V}$	$R_{p,d}$ [kN]	Check - anchor
Wall 21	Connection 1	Horizontal ULS 1	1	0,72	3,80	1,1	1,3	3,22	22%	0,72	21	1,5	14	5%
Wall 22	Connection 1	Horizontal ULS 4	1	11,31	3,80	1,1	1,3	3,22	352%	11,31	21	1,5	14	81%
Wall 23	Connection 1	Horizontal ULS 4	1	11,60	3,80	1,1	1,3	3,22	361%	11,60	21	1,5	14	83%
Wall 24	Connection 1	Horizontal ULS 1	1	0,75	3,80	1,1	1,3	3,22	23%	0,75	21	1,5	14	5%
Wall 25	Connection 1	Horizontal ULS 1	1	0,59	3,80	1,1	1,3	3,22	18%	0,59	21	1,5	14	4%
Wall 26	Connection 1	Horizontal ULS 4	1	9,96	3,80	1,1	1,3	3,22	310%	9,96	21	1,5	14	71%
Wall 27	Connection 1	Horizontal ULS 1	1	0,58	3,80	1,1	1,3	3,22	18%	0,58	21	1,5	14	4%
Wall 28	Connection 1	Horizontal ULS 4	1	8,06	3,80	1,1	1,3	3,22	251%	8,06	21	1,5	14	58%
Wall 29	Connection 1	Horizontal ULS 4	1	8,06	3,80	1,1	1,3	3,22	251%	8,06	21	1,5	14	58%

Table 4-9: Check of angle bracket connections resistance in case 1

4.5.2 Resistance check of angle brackets in case 2

Table 4-10 shows the check of the angle brackets connections for shear failure of case 2. It shows that the angle brackets of all walls managed well to resist the shear force. The resistance check of the anchor also shows that the anchor of all walls satisfies the requirement.

Wall name	Connection name	Comb.	Service class	$V_{a,d}$ [kN]	$R_{a,k,dens}$ [kN]	k_{mod}	γ_M	$R_{a,d}$ [kN]	Check – angle bracket	$V_{p,d}$ [kN]	$R_{p,k}$ [kN]	$\gamma_{Ms,V}$	$R_{p,d}$ [kN]	Check - anchor
Wall 21	Ground connection - hold down - bracket6	Horizontal ULS 1	1	1,43	22,10	1,1	1,3	18,7	8%	1,39	21	1,25	16,8	8%
Wall 22	Ground connection - hold down - bracket8	Horizontal ULS 4	1	11,20	22,10	1,1	1,3	18,7	60%	10,86	21	1,25	16,8	65%
Wall 23	Ground connection - hold down - bracket8	Horizontal ULS 4	1	11,38	22,10	1,1	1,3	18,7	61%	11,04	21	1,25	16,8	66%
Wall 24	Ground connection - hold down - bracket6	Horizontal ULS 1	1	1,48	22,10	1,1	1,3	18,7	8%	1,43	21	1,25	16,8	9%
Wall 25	Ground connection - hold down - bracket4	Horizontal ULS 1	1	1,68	22,10	1,1	1,3	18,7	9%	1,63	21	1,25	16,8	10%
Wall 26	Ground connection - hold down - bracket3	Horizontal ULS 4	1	15,84	22,10	1,1	1,3	18,7	85%	15,36	21	1,25	16,8	91%
Wall 27	Ground connection - hold down - bracket5	Horizontal ULS 1	1	1,34	22,10	1,1	1,3	18,7	7%	1,30	21	1,25	16,8	8%
Wall 28	Ground connection - hold down - bracket9	Horizontal ULS 4	1	8,79	22,10	1,1	1,3	18,7	47%	8,53	21	1,25	16,8	51%
Wall 29	Ground connection - hold down - bracket9	Horizontal ULS 4	1	8,79	22,10	1,1	1,3	18,7	47%	8,53	21	1,25	16,8	51%

Table 4-10: Check of angle bracket connections resistance in case 2

4.6 Roofs and linear elements

Since roofs and linear elements (timber beams) in both cases have the same properties and are affected by the same load, the check of these will also give the same result in both projects. The check of bending strength and shear strength of roof and linear elements are given in appendix C.

5 Discussion

The discussion will be divided into two parts. In the first part, the technical section of the thesis will be discussed including an interpretation of the results from the previous section.

In the second part, the role of wood materials in the circular economy will be discussed in more detail.

5.1 Technical analysis of wooden cabins against horizontal load

In the technical part of this thesis, a wooden cabin has been considered as two different cases that were affected by wind load corresponding to avalanche load. The focus was mostly on analysis of wall construction against the horizontal load. Roofs and linear elements were not exposed to the horizontal load, but they were vertically loaded. In order to reach a conclusion, an attempt will be made to answer the problem statement. Discussion of the results will be based on the following questions:

1- Which components are most exposed to the horizontal load?
2- Which parameters are crucial for the building's horizontal stability?
3- What are the solutions for horizontal stiffening?

In order to answer these questions, it is important to initially discuss the two different cases. The method section showed that wall constructions consist of components such as studs, sheeting boards and fasteners and the walls were attached to the foundation with two different types of anchors, such as hold-down connections and angle brackets connections. These components must be sufficiently resistant so that wall structures can resist the horizontal load. Therefore, a wooden cabin was studied as two separate cases.

Material properties of the components in the first case were chosen as follows:

- The strength classes of studs were selected of C24 materials and the cross-section of 100X160
- The cross-section of fasteners was chosen at 2,8/3,1X65 and the stiffness of 918 N/mm
- The stiffness of Hold-down connections was chosen 14887 N/mm and the stiffness of angle brackets connections was chosen 24353 N/mm

In the first case, after selecting the material properties of wall components and anchors, a 20 kpa. wind load was applied to the cabin in the y-direction. From the results, it was noted which components were most susceptible to the horizontal load and also which components demonstrated a resistance or did not resist to the horizontal load.

The load effect from wind in the x-direction was equal to 2,5 kpa. which is much lower than load effect in the y-direction, therefore the dimensioning that made relative to the load impact in y-direction will satisfy the requirement in the x-direction as well.

Results in case 1 showed that studs of framed walls (green walls) and components such as fasteners, hold-down connections and angle-brackets connections of shear walls (red walls) failed to resist the horizontal load, but the sheeting boards could resist the horizontal load. Figure 5-1 shows the names of the walls and the direction of wind load.

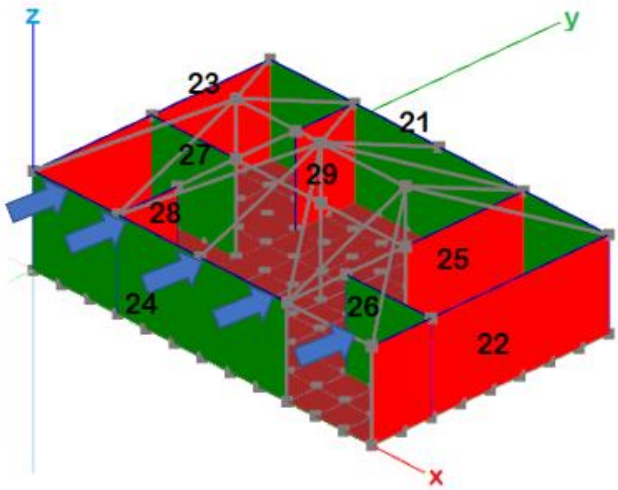


Figure 5-1 Color and name of the walls and the wind direction.

In case 1, it was observed, that the green walls (walls 21, 24, 25 and 27) which are perpendicular to the wind direction were most exposed to the bending force and less exposed to the shear force but the red walls (walls 22,23,25,28 and 29) which are parallel to the wind direction were most exposed to the shear force and less exposed to the bending.

The suds in green walls, in addition to the bending were also subjected to the axial load from the roofs, linear elements and self-weight. Therefore, these studs were examined for buckling. Due to the high bending load and low bending strength of the studs, the studs failed for buckling in case 1. The fasteners and anchors of the red walls were most exposed to the shear force and less exposed to bending force therefore, these components were examined for shear failure where they also failed against shear force.

Thus, the material characteristics of these unstable components were changed to better material characteristics and the cabin studied again as case 2.

In case 2, the strength characteristics of the studs was changed to C40 and the cross-section changed to 120X200. The cabin was loaded again with the same horizontal load. It was observed that the studs could then resist the horizontal load. This meant that the change of material strength classes and the cross-section are the same as the change of stiffness and mass, thus increased stiffness and mass is the solution for horizontal stiffening or stability of the cabin.

Fasteners were also changed to a thicker and longer fastener with a cross section of 5X80 and stiffness of 1618 N/mm. Therefore, the result in case 2 showed that fasteners with the new material characteristics also satisfy the requirement. Here, it was shown again that increased stiffness and mass of wall components can affect the total horizontal stiffening.

Prevention from wall turning is achieved with appropriate anchoring of the walls to the foundation. The material properties of the two different types of anchors that were used on the bottom to anchor timber frame walls, were also changed. Stiffness of angle-brackets connection was increased to 52186 N/mm and stiffness of hold-down connections increased to 19977 N/mm. Following increased stiffness of the two anchors, these also resisted the shear force. Consequently, stiffness of the angle-brackets will reduce acceleration and the risk of shear failure and protect the cabin from sliding. Increased stiffness of the hold-down anchors can contribute to tension or hold down-forces and prevent lifting.

Since sheeting boards resisted well against the avalanche load in case 1, the properties of these in case 2 were chosen the same as in case 1.

The two cases have been discussed and based on this, it has been determined which components were most exposed to the horizontal load and which parameters can be decisive as the solutions to increase the horizontal stiffening of the cabin.

Based on this, it can be concluded that studs of the framed walls, fasteners and anchors of the shear walls are the components that are most exposed to the horizontal load.

In case 2, following a change of the material’s strength properties and cross-section of the components, it was found they could resist the horizontal load.

Therefore, it can be concluded, that stiffness and mass are the crucial parameters for the cabin's horizontal stability. Loads that affect the building are also a crucial parameter. In both cases, a 20 kpa wind load as a dominant load in y-direction was used which is assumed to be the maximum limit that may be required.

So, the stiffness properties and mass of the components are important for the stiffening of the entire cabin. In addition to this, stiffness is linked to the movement of the construction. If the stiffness is increased, the movement will decrease. Changing of the stiffness will also affect the natural frequency of the construction, whereas an increased stiffness will lead to an increase of the natural frequency.

Another solution for horizontal stiffening is to have the correct number and proper distribution of shear walls. The placement of shear walls must be such that the structure can distribute the horizontal force in all directions and can also accept shear forces and prevent rotation. In order to stabilize the building against horizontal loads from several directions, it must have at least three shear walls, but they must not cross each other, and the walls should not be parallel.

Technical analysis of timber cabin was done in TimberTech software where wind load was used as horizontal load instead of avalanche load, this could give inaccurate results. The design of the cabin was also not perfect as the correct openings were not taken into account and this also could give some inaccuracy in relation to results.

5.2 Wood in circular economy

In this part of the chapter, the role of wood in the circular economy will be discussed. In the literature study for this thesis, the definition of circular economy was considered; circular economy in construction industry and some benefits of wood as building materials. In order to reach a conclusion, the positive and negative aspects of wood materials need consideration and also a comparison with other building materials.

The construction industry uses around 40% of all resources worldwide. An ever increasing population will increase resource use, and this requires a transition from linear to circular economy. Circular economy helps resources remain in the economy. It is based on reuse, repair, improvement and material recycling where fewest resources are lost.

Wood is a natural and renewable building material. Increased use of wood products reduces emissions of greenhouse gases to the atmosphere. When trees grow, $CO2$ is converted to biomass through photosynthesis, which is part of the natural carbon cycle. When producing wood material, the entire raw material is utilized so that 55 % of the wood becomes lumber and the remaining quantities are by-products that can be used for energy. Wood is a light material in relation to strength properties, which contributes to less transport loads and less energy consumption. The production of three products requires little energy in manufacturing, which aids in the emission reduction of $CO2$. Wood can be recycled and reused, after the primary function of the wood is over, it can be used to produce energy. Combustion of clean wood does not require a cleaning method. Wood materials also have a positive effect on the indoor environment and have the ability to regulate air. Negative aspects of the wood are that they can easily be exposed to moisture damage and be attacked by microorganisms and insects under certain conditions because they are natural materials. They are also flammable and relatively easy to ignite but they can be improved against fire by proper manufacture and treatment.

By comparing wood materials with other building material, wood has a lower environmental impact. For example, steel is suitable for reuse, but it has a high environmental impact during manufacture and has a relatively short lifetime compared to wood. Concrete is a widely used building material but provides the largest part of waste as demolition material. By looking at these wood properties and comparing with other materials, it can be summarised that wood material can be an important material for transition to the circular economy.

6 Conclusion

Here will be a brief conclusion based on the results and the discussion.

In technical analysis, a cabin was tested against horizontal load in two different cases with the purpose of finding solutions to the cabin's horizontal stability. In the analysis, wall components with different strength classes and cross-sections were tested and different types of ground connections for shear and hold-down anchoring of timber-framed walls were investigated.

Technical analysis was done in TimberTech software where wind load was used as horizontal load instead of avalanche load. This compensation could have given some inaccuracy in relation to results.

The results have shown that case1 has not satisfied the requirements, but case 2 has satisfied the requirements. From the tests, it was found that the walls that are perpendicular to the wind direction were most affected by bending forces and the walls that are parallel to the wind direction were most affected by shear forces.

Lateral forces create strong torsional and this leads to failure of the structure by shear. In order that shear-walls will provide good resistance and a less flexible plan; the correct number and proper distribution of shear-walls are important. For buildings where it is possible to have many internal shear-walls, it is a good solution for horizontal bracing.

The materials' stiffness properties and mass to components are of great importance for the building's resistance and are important parameters in relation to cabin behavior.

Attaching walls to the foundation is also an effective measure to increase the building's resistance against horizontal loads which are transmitted to the foundation by timber framed walls. With appropriate anchoring of walls to the ground, rotation and uplift of the walls will be prevented and the acceleration and horizontal displacement will be reduced.

In the economic part, it was looked at the role of wood materials as a building's material in the circular economy, where positive and negative aspects of wood materials were discussed and simultaneous it was compared with other building materials in relation to the circular economy.

Construction and the construction industry as well as large resource consumption include as a large greenhouse gas emission. By setting circular economy as a requirement will contribute to the reduction of material costs, the amount of waste, climate emissions and environmental impacts related to construction projects.

Wood materials are renewable materials and increased use of wood materials as building materials will contribute to major emission reductions. The use of wood materials has several positive effects. In addition to renewable resource, it can be reused as energy; have good durability and strength and give greater growth of the forest than it is taken out. This means that wood materials as building material has a crucial role in the transition to the circular economy.

7 Recommendation for the future research

A thesis has a limited time perspective which causes many interesting problems to be omitted. Therefore, in this final chapter, some criteria that the author proposes as a further development of this thesis will be highlighted.

In this thesis, the focus was only on analysis of timber framed wall structures against the avalanche load and other parts of the building such as roofs and beams were not taken into account. For future research, analysis of roof constructions and linear elements against avalanches can be investigated. Furthermore, analysis of Cross Laminated walls (CLT) can also be studied.

The analysis performed in TimberTech Buildings software where wind load was used corresponding to the avalanche load. In order to reduce inaccuracy and any faults that cause this load change, it is better to use avalanche load in further work.

The cabin that was studied had a rectangular geometry and one floor height. Analysis of other geometric shapes and cabins with more than one floor can also be investigated in further work so that it could be shown how different geometric conditions and the number of floors will affect the cabin’s resistance against the avalanche load.

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Appendix A

A.1 Combination Factors

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

Table A-1: Recommended values of Ψ factors for buildings (Porteous & Kermani, 2013)

Appendix B

B.1 Strength classes

Table B-1 shows strength properties of solid wood.

		Coniferous species and Poplar												Deciduous species					
		C14	C16	C18	C20	C22	C24	C27	C30	C35	C40	C45	C50	D30	D35	D40	D50	D60	D70
Strength properties in N/mm^2																			
Bending	$f_{m,k}$	14	16	18	20	22	24	27	30	35	40	45	50	30	35	40	50	60	70
Tension parallel to grain	$f_{t,0,k}$	8	10	11	12	13	14	16	18	21	24	27	30	18	21	24	30	36	42
Tension perpendicular to grain	$f_{t,90,k}$	0,4	0,5	0,5	0,5	0,5	0,5	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6
Compression parallel to grain	$f_{c,0,k}$	16	17	18	19	20	21	22	23	25	26	27	29	23	25	26	29	32	34
Compression perpendicular to grain	$f_{c,90,k}$	2,0	2,2	2,2	2,3	2,4	2,5	2,6	2,7	2,8	2,9	3,1	3,2	8,0	8,4	8,8	9,7	10,5	13,5
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	3,0	3,4	3,8	4,6	5,3	6,0
Stiffness properties in kN/mm^2																			
Mean value of modulus of elasticity parallel to grain	$E_{0,mean}$	7	8	9	9,5	10	11	11,5	12	13	14	15	16	10	10	11	14	17	20
5% value of modulus of elasticity parallel to grain	$E_{0,05}$	4,7	5,4	6,0	6,4	6,7	7,4	7,7	8,0	8,7	9,4	10,0	10,7	8,0	8,7	9,4	11,8	14,3	16,8
Mean value of modulus of elasticity pependicular to grain	$E_{90,mean}$	0,23	0,27	0,30	0,32	0,33	0,37	0,38	0,40	0,43	0,47	0,50	0,53	0,64	0,69	0,75	0,93	1,13	1,33
Mean value of shear modulus	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	0,60	0,65	0,70	0,88	1,06	1,25
Density in kg/m^3																			
Density	ρ_k	290	310	320	330	340	350	370	380	400	420	440	460	530	560	590	650	700	900
Mean value of density	ρ_{mean}	350	370	380	390	410	420	450	460	480	500	520	550	640	670	700	780	840	1080

Table B-1: Strength classes of Solid Timber (TEMTIS, May.2008)

Table B-2 shows strength properties of combined glulam.

Egenskap		Limtre fasthetsklasse						
	Symbol	GL20c	GL22c	GL24c	GL26c	GL28c	GL30c	GL32c
Bøyefasthet	$f_{m,g,k}$	20	22	24	26	28	30	32
Strekkfasthet	$f_{t,0,g,k}$	15	16	17	19	19,5	19,5	19,5
	$f_{t,90,g,k}$	0,5						
Trykkfasthet	$f_{c,0,g,k}$	18,5	20	21,5	23,5	24	24,5	24,5
	$f_{c,90,g,k}$	2,5						
Skjærfasthet (skjær og torsjon)	$f_{v,g,k}$	3,5						
Rulleskjærfasthet	$f_{r,g,k}$	1,2						
Elastisitetsmodul	$E_{0,g,mean}$	10400	10400	11000	12000	12500	13000	13500
	$E_{0,g,05}$	8600	8600	9100	10000	10400	10800	11200
	$E_{90,g,mean}$	300						
	$E_{90,g,05}$	250						
Skjærmodul	$G_{g,mean}$	650						
	$G_{g,05}$	542						
Rulleskjærmodul	$G_{r,g,mean}$	65						
	$G_{r,g,05}$	54						
Densitet	$\rho_{g,k}$	355	355	365	385	390	390	400
	$\rho_{g,mean}$	390	390	400	420	420	430	440

Table B-2: Strength classes of combined glulam (Bell et al., 2015)

Table B-3 shows strength properties of homogenous glulam.

Egenskap	Symbol	Limtre fasthetsklasse						
		GL20h	GL22h	GL24h	GL26h	GL28h	GL30h	GL32h
Bøyefasthet	$f_{m,g,k}$	20	22	24	26	28	30	32
Strekkfasthet	$f_{t,0,g,k}$	16	17,6	19,2	20,8	22,3	24	25,6
	$f_{t,90,g,k}$	0,5						
Trykkfasthet	$f_{c,0,g,k}$	20	22	24	26	28	30	32
	$f_{c,90,g,k}$	2,5						
Skjærfasthet (skjær og torsjon)	$f_{v,g,k}$	3,5						
Rulleskjærfasthet	$f_{r,g,k}$	1,2						
Elastisitetsmodul	$E_{0,g,mean}$	8400	10500	11500	12100	12600	13600	14200
	$E_{0,g,05}$	7000	8800	9600	10100	10500	11300	11800
	$E_{90,g,mean}$	300						
	$E_{90,g,05}$	250						
Skjærmodul	$G_{g,mean}$	650						
	$G_{g,05}$	540						
Rulleskjærmodul	$G_{r,g,mean}$	65						
	$G_{r,g,05}$	54						
Densitet	$\rho_{g,k}$	340	370	385	405	425	430	440
	$\rho_{g,mean}$	370	410	420	445	460	480	490

Table B-3: Strength classes of homogenous glulam (Bell et al., 2015)

Appendix C

C.1 Joist floors / Glued laminated timber floors

C.1.1 Bending strength

The checks are conducted according to EN 1995-1-1. The following expression shall be satisfied:

$$\frac{\sigma_{m,d}}{k_{crit} \cdot f_{m,d}} \leq 1$$

Table C-1 shows the checks of bending strength of cabin's roofs.

Floor name	Section	$M_{3\text{-}3\ max}$ [kNm]	W [mm^3]	k_{crit}	Comb.	Service Class	k_{mod}	γ_M	$f_{m,d}$ [MPa]	$\sigma_{m,d}$ [MPa]	Check
Floor 2	Joists floor 160x200	4,79	1066667	1,00	ULS 28	1	0,6	1,25	14,40	4,49	31%
Floor 3	Joists floor 160x200	1,79	1066667	1,00	ULS 28	1	0,6	1,25	14,40	1,68	12%
Floor 4	Joists floor 160x200	1,65	1066667	1,00	ULS 28	1	0,6	1,25	14,40	1,55	11%
Floor 5	Joists floor 160x200	2,25	1066667	1,00	ULS 28	1	0,6	1,25	14,40	2,10	15%
Floor 6	Joists floor 160x200	2,24	1066667	1,00	ULS 28	1	0,6	1,25	14,40	2,10	15%

Table C-1 Bending strength check of roofs

C.1.2 Shear strength

The checks are conducted according to § 6.1.7 of EN 1995-1-1. The following expression shall be satisfied:

$$\frac{\tau_d}{f_{v,d}} \leq 1$$

The checks of shear strength of roofs are summarized in table C-2. The values resulting from the calculations, relating to each verification, are reported in the form of a percentage.

Floor name	Section	$V_{2\ max}$ [kN]	Area [mm^2]	k_{cr}	Comb.	Service Class	k_{mod}	γ_M	$f_{v,d}$ [MPa]	$\tau_{2,d}$ [MPa]	Check
Floor 2	Joists floor 160x200	5,64	32000	0,67	ULS 28	1	0,6	1,25	1,92	0,39	21%
Floor 3	Joists floor 160x200	3,23	32000	0,67	ULS 28	1	0,6	1,25	1,92	0,23	12%
Floor 4	Joists floor 160x200	3,10	32000	0,67	ULS 28	1	0,6	1,25	1,92	0,22	11%
Floor 5	Joists floor 160x200	2,69	32000	0,67	ULS 28	1	0,6	1,25	1,92	0,19	10%
Floor 6	Joists floor 160x200	2,69	32000	0,67	ULS 28	1	0,6	1,25	1,92	0,19	10%

Table C-2 Shear strength check of the roofs

C.2 Timber beams

C.2.1 Bending strength

The checks are conducted according to EN 1995-1-1. The following expression shall be satisfied:

$$\frac{\sigma_{m,d}}{k_{crit} \cdot f_{m,d}} \leq 1$$

The checks of bending strength of timber beams are summarized in table C-3.

Beam name	Section	$M_{3\text{-}3\ max}$ [kNm]	W [mm^3]	Lateral restraints	$\sigma_{m,crit}$ [MPa]	k_{crit}	Comb.	k_{mod}	γ_M	$f_{m,d}$ [MPa]	$\sigma_{m,d}$ [MPa]	Check
Beam 8	Section 200x200 C24	0,27	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	11,52	0,20	2%
Beam 9	Section 200x200 C24	7,50	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	11,52	5,62	49%
Beam 10	Section 200x200 C24	7,62	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	11,52	5,71	50%
Beam 11	Section 200x200 C24	0,27	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	11,52	0,20	2%
Beam 12	Section 200x200 C24	0,27	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	11,52	0,20	2%
Beam 13	Section 200x200 C24	7,62	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	11,52	5,71	50%
Beam 14	Section 200x200 C24	7,59	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	11,52	5,69	49%
Beam 15	200X200 C50	10,86	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	24,00	8,15	34%
Beam 16	200X200 C50	10,50	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	24,00	7,87	33%
Beam 17	Section 200x200 C24	0,27	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	11,52	0,20	2%
Beam 18	Section 200x200 C24	0,27	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	11,52	0,20	2%
Beam 19	200X200 C50	10,49	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	24,00	7,86	33%
Beam 20	200X200 C50	10,87	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	24,00	8,15	34%
Beam 23	200X200 C50	11,16	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	24,00	8,37	35%
Beam 24	200X200 C50	8,72	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	24,00	6,54	27%
Beam 25	200X200 C50	11,17	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	24,00	8,38	35%
Beam 26	200X200 C50	11,03	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	24,00	8,27	34%
Beam 27	200X200 C50	9,03	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	24,00	6,78	28%
Beam 28	200X200 C50	11,17	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	24,00	8,38	35%
Beam 29	200X200 C50	11,72	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	24,00	8,79	37%
Beam 30	200X200 C50	12,16	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	24,00	9,12	38%
Beam 31	200X200 C50	12,27	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	24,00	9,20	38%
Beam 32	200X200 C50	12,27	1333333	No torsional buckling	-	1,00	ULS 28	0,6	1,25	24,00	9,20	38%

Table C-3 Bending strength check of timber beams

C.2.2 Shear strength

The checks are conducted according to EN 1995-1-1. The following expression shall be satisfied:

$$\frac{\tau_d}{f_{v,d}} \leq 1$$

The checks of shear strength for timber beams are summarized in table C-4.

Beam name	Section	$V_{2\,max}$ [kN]	Area [mm^2]	k_{cr}	Comb.	Service class	k_{mod}	γ_M	$f_{v,d}$ [MPa]	$\tau_{2,d}$ [MPa]	Check
Beam 8	Section 200x200 C24	0,36	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,02	1%
Beam 9	Section 200x200 C24	15,55	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,87	45%
Beam 10	Section 200x200 C24	16,71	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,94	49%
Beam 11	Section 200x200 C24	0,36	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,02	1%
Beam 12	Section 200x200 C24	0,36	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,02	1%
Beam 13	Section 200x200 C24	16,77	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,94	49%
Beam 14	Section 200x200 C24	17,86	40000	0,67	ULS 28	1	0,6	1,25	1,92	1,00	52%
Beam 15	200X200 C50	9,12	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,51	27%
Beam 16	200X200 C50	7,66	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,43	22%
Beam 17	Section 200x200 C24	0,36	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,02	1%
Beam 18	Section 200x200 C24	0,36	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,02	1%
Beam 19	200X200 C50	7,66	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,43	22%
Beam 20	200X200 C50	9,18	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,51	27%
Beam 23	200X200 C50	10,31	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,58	30%
Beam 24	200X200 C50	10,80	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,60	31%
Beam 25	200X200 C50	10,31	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,58	30%
Beam 26	200X200 C50	10,31	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,58	30%
Beam 27	200X200 C50	11,03	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,62	32%
Beam 28	200X200 C50	10,48	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,59	31%
Beam 29	200X200 C50	9,33	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,52	27%
Beam 30	200X200 C50	9,56	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,53	28%
Beam 31	200X200 C50	9,79	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,55	29%
Beam 32	200X200 C50	9,86	40000	0,67	ULS 28	1	0,6	1,25	1,92	0,55	29%

Table C-4 Shear strength check of the timber beams

Appendix D

D.1 List of figures

D.2 List of tables

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