CHALMERS
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Robustness in timber structures

Numerical study on how progressive collapse influences the floor structure in a multi-story building

Master's Thesis in Structural Engineering and Building Technology

Andreas Jonsson
Stefan Olander

Department of Civil and Environmental Engineering
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Department of Civil and Environmental Engineering
Division of Structural Engineering
Steel and Timber Structures
Chalmers University of Technology
SE-412 96 Göteborg
Sweden
Telephone: + 46 (0)31-772 1000

Cover:
Mobil crane crashes into a timber house in Stavanger, Norway (Nedrebö R., Jacobsen K, 2013).

Department of Civil and Environmental Engineering, Göteborg, Sweden, 2016

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ABSTRACT

Progressive collapse is unproportional failure in a structural system that can lead to serious consequences, in case of an accident. For houses with five stories or more one is obligated to design buildings so robust that a progressive collapse is prevented. Security is always an important aspect when designing structures and for the timber house manufacturer A-hus it is important, since they produce multi-story residential buildings up to eight stories.

The aim of this thesis is to describe the phenomenon of progressive collapse in the context of timber frame structures. To design a floor system that can in case of an accident create an alternative load path to keep the load-carrying ability and thereby prevent a progressive collapse from occurring.

In order to design a building so that in an event of an accident progressive collapse does not occur, a literature study has been made to find key aspects to use in design. One major key aspect is to have continuity between building elements. Eurocode 1 suggests designing interior and exterior horizontal ties to achieve this. For the design of these ties Eurocode 1 suggests two alternatives, one is based on recommended values for the load and the other one is modelling the removal of a wall supporting the floor structure to analyze the strength in the ties. To get better understanding of how the floor structure reacts the second alternative will be further investigated.

The method used to analyze A-hus' existing floor structure is a numerical study and the result is that there is a need to modify the floor structure to secure an alternative load path. This can be done by creating an exterior horizontal tie of plywood boards that ensures continuity between floor elements when the main support is lost.

To prevent progressive collapse in the event of an accident, it is vital to have continuity between all of the building elements. It is the authors' hope that this thesis will lead to safer timber buildings and to show that the actions needed to prevent a progressive collapse can be done with minor extra cost and effort.

Key words: Abaqus, Connection, Continuity, FE-model, Floor structure, Horizontal tie, Multi-story timber houses, Progressive collapse, Robustness, Timber structures.

Robusthet i träkonstruktioner
Numerisk studie av hur fortskridande ras påverkar bjälklaget i ett flervåningshus

Examensarbete inom masterprogrammet Structural Engineering and Building Technology

Andreas Jonsson
Stefan Olander
Institutionen för bygg- och miljöteknik
Avdelningen för Konstruktionsteknik
Stål- och träkonstruktioner
Chalmers tekniska högskola

SAMMANFATTNING

Fortskridande ras är en oproportionerligt stor kollaps i ett strukturellt system som kan leda till allvarliga konsekvenser i händelse av en olycka. För hus med fem våningar eller fler, är det föreskrivet att utforma konstruktionen så robust att ett fortskridande ras förhindras. Säkerhetsaspekten är alltid viktig vid utformning av konstruktioner och för trähus tillverkaren A-hus är det viktigt, eftersom de producerar bostadshus upp till åtta våningar.

Denna avhandling syftar till att beskriva fenomenet fortskridande ras kopplat till trähus. Samt att utforma ett bjälklagssystem som i händelse av en olycka har en alternativ lastväg som förhindrar att bjälklaget kollapsar och därigenom förhindra att fortskridande ras uppstår.

En litteraturstudie har gjorts för att hitta viktiga aspekter och för att skaffa kunskap om hur fortskridande ras förhindras. En viktig aspekt är att kontinuitet mellan byggelementen säkerställs och för att uppnå detta föreslås det i Eurokod 1 att designa inre- och yttre horisontella förband. För utformningen av dessa förband föreslår Eurokod 1 två alternativ. Det ena är en handberäknings metod och den andra är att ta bort en vägg som bär upp ett bjälklag. För att få bättre förståelse för hur bjälklaget beter sig har det andra alternativet används.

Den metod som används för att analysera den befintliga bjälklagskonstruktionen är en numerisk studie där resultatet visar att det finns ett behov att modifiera bjälklaget för att säkerställa en alternativ lastväg. Genom att designa ett yttre horisontellt dragband av plywoodskivor, säkerställs kontinuiteten mellan bjälklagselementen när upplaget försvinner och en alternativ lastväg säkras.

För att förhindra fortskridande ras i händelse av en olycka är det viktigt att ha kontinuitet i byggnadskonstruktion. Det är författarnas förhoppning att denna avhandling kommer leda till att fortskridande ras förhindras i flervåningshus i trä till blygsamma extra kostnader och ansträngningar.

Nyckelord: Abaqus, Anslutningar, Bjälklag, FE-modell, Flervåningshus i trä, Fortskridande ras, Horisontellt förband, Kontinuitet, Robusthet, Träkonstruktion.

Contents

Preface

Like the famous sculpture Le Penseur that pictures a man in deep thought made by Auguste Rodin this thesis has been a thought process to solve the problem with the floor system by designing the alternative load path. The best ideas have come to us during long walks around Mossen and in the corner of the south entrance to Forskarhuset where the sun always shines.

Great thanks to our supervisors at A-hus Linus Abrahamson and Tobias Persson who have believed in us and given us freedom to change the direction of this thesis when we found another part of their building system that was more interesting to investigate. We are also grateful that they have let us use our own knowledge to solve this problem.

Reza Haghani Dogaheh has been a big support when establishing limitations and pointing out the direction of this thesis. Otherwise we would have had a hard time limiting ourselves and would have lost focus on the important things, so thank you Reza.

During the work with FE-modelling in Abaqus there were some problems that we were unable to solve ourselves. But then came Mohsen to our rescue with his deep knowledge in modelling in Abaqus. So Thanks Mohsen Heshmati!

To write a text in a foreign language is in the best case considered to be stimulating and challenging. It can provide a deeper understanding and appreciation for the art of writing. We would like to thank Nathalie Kullberg and Selma Sinanovic for the taking on our text and deepened our knowledge in the noble art of writing.

And finally, we want to thank our opponents and friends Jakob Nordenstam and Gustav Svantesson for our shared moments in Snobberiet discussing our master's thesis over a cup of coffee.

Göteborg June 2016

Andreas Jonsson and Stefan Olander

1 Introduction

The number of multi-story timber structures have increased since 1994 when the Swedish rules changed and allowed timber houses taller than two stories (Boverket 2006). It is the authors' opinion that this late change has made timber industries fall behind in their studies on robustness. Therefore the building code is based on concrete structures. This is reflected in the amount of literature available with focus on designing timber buildings to prevent a progressive collapse. This thesis aims to provide knowledge for the timber industries regarding robustness and progressive collapse.

1.1 Background

The timber house manufacturer A-hus is a part of Derome group and produces prefabricated timber frames for multi-story buildings. With their building concept they have the possibility to produce buildings up to eight stories. A-hus' role in the building process is to supply timber frames to a contractor. Their commitments are to deliver timber frames that are time efficient to assemble and to guarantee structural demands.[1]

A-hus wants to improve their building system by evaluating the connection between exterior walls and floor, see Section 8.1. Since connecting these elements is time consuming on the building site and it is difficult to evaluate their load bearing capacity in the calculation phase, further evaluation of these joints is desirable.

From A-hus' calculations on structural stability for the whole structure a conclusion can be drawn that the governing structural phenomenon that is affecting this connection is progressive collapse. Therefore more knowledge regarding progressive collapse is needed in order to make a better connection.

When examining Eurocode's recommendations for design of progressive collapse, one alternative that is stated is to remove a load-bearing wall and evaluate the consequences of it. Due to support condition of the floor system and the connections between floor elements the authors began to question the floor structure's ability to prevent a progressive collapse.

1.2 Aims and objectives

This thesis' describes the phenomenon of progressive collapse in the context of timber frame structures. To design a floor system that can in case of an accident create an alternative load path that keeps the load-carrying ability and thereby prevent a progressive collapse from occurring.

1.3 Limitations

Besides the phenomenon of progressive collapse, other types of loads and load combinations need to be considered when designing timber frame structures, such as

[1] Meeting with Tobias Persson designer at A-hus 2016-01-21

wind, snow and imposed load. However, this thesis is limited to investigate how to design a floor structure that in the event of an accident prevents progressive collapse from occurring. Further on, many different load situations could occur in progressive collapse, but since Eurocode 1 suggests a removal of a load-bearing wall, this situation will be investigated.

In this situation walls are considered to be robust and able to carry the redistributed load. Further on, the connections of the floor structure to the interior walls are not taken into account in this thesis.

1.4 Method

This thesis begins with a literature study on progressive collapse and the regulations that govern the design of this phenomenon. To get familiar with and to evaluate A-hus' building system, there will be a short description of the load bearing structure and the different building members. To learn about how progressive collapse affects the connection that is evaluated, a finite element model (FE-model) of the floor is made with an existing building as a reference.

During the process of FE-modeling A-hus' floor structure, it was obvious that the floor was not designed with progressive collapse in mind. Therefore a study on how to improve the floor to withstand a progressive collapse was carried out. Different solutions are presented and a muli-criteria analysis is used to choose the best solution. To evaluate the capacity of the improved floor system a new FE-model was developed, from which it was possible to extract design values for the design of the external horizontal tie and the connection between the floor elements and the exterior wall.

To improve the understanding on how to connect the floor elements to the wall, a literature study is performed to find different existing solutions for connecting these elements. The study is performed by using internet, books and through contact with different companies within the building sector. In order to evaluate pros and cons with the existing solution and to identify possible improvements that can be made, a structural analysis of the original connection is performed. For further evaluation an interview with one building site assembly team is made to gain knowledge of how the installation of the connector works in reality. With all the knowledge collected, a process of improving the connection is performed.

In the thesis several references to Eurocode are made and for simplification the different parts of the code are referenced according to:

- Eurocode 0 (Swedish standard institute 2002).
- Eurocode 1 (Swedish standard institute 2008).
- Eurocode 5 (Swedish standard institute 2009).

2 Progressive Collapse

This chapter is mostly based on the study of the book *Designing concrete structures to avoid progressive collaps* by Albertsson Å. et al in 1982. This book gives a comprehensive understanding of progressive collapse in buildings. Although the book focuses on concrete structures the basic way of dimensioning the structure should be the same for timber structures. Since the book is extensive and treats many aspects of how to design robust buildings, effort has been made to cover the topics that can be of interest when designing timber structures composed of block elements.

Progressive collapse is when one or more elements in a construction lose its resistance, changes in conditions for load uptake occur. This can in an unfavorable case lead to more and more building member's breaking apart. In many cases this phenomenon can get catastrophic effects and, in the context of the cause of the damage, unreasonable consequences.

2.1 History

The development of norms to prevent progressive collapse is mostly based on lessons learned from previous collapses of buildings. There is a number of incidents that have influenced and changed the view on robustness in structures. Two of these accidents are described below, as well as one example of an accident that could have led to a progressive collapse in other circumstances.

2.1.1 Ronan Point

One of these incidents that has had a great impact is the partial collapse of a 22 stories high residential building in London in the neighborhood Ronan Point in 1968, see Figure 2.1. The house was constructed with prefabricated concrete elements where the connections between these elements were improper. The collapse was initiated by a gas explosion in a corner flat on the 18th story. The unusually strong explosion led to the collapse of the loadbearing exterior wall in one of the apartments. The loss in load carrying capacity led to the floors above instantly falling down because of the lack of coherence within the structure, and landing on the floor of the 18th story. This increase in weight became too much for underlying structures and a vertical progressive collapse was a fact.

Figure 2.1 Ronan Point 1968 (Verlaan, T. 2011)

The catastrophic event in Ronan Point led to a worldwide debate regarding structural robustness and many countries realized that there was a need for regulations that dealt with unforeseen events. The regulations that came out of these debates dealt both with how to foresee the accidental loads and how the structure should be built to avoid progressive collapse.

2.1.2 The World Trade Center

A more recent incident occurred when the first aircraft hit the World Tarde Center. It inflicted severe damage, but this was only local and did no strip a significant portion of the steel insulation. The subsequent fire would likely not have led to the overall collapse.

According to Bažant and Verdure the scenario of the collapse is like this: When the aircraft hit the World Trade Center about 13% of the total number of 287 columns were severely damaged. This caused stress redistribution, increasing the load on the columns that were still standing, reaching or almost reaching the load capacity of some of them. Since some of the steel insulation was stripped, the heat from the fire further diminished the capacity of some of the affected columns.

Thermal expansion combined with heat-induced deformation caused the floor trusses to sag and because these were fixed to the columns placed in the parameter of the building, this led to large bowing of the columns acting as large initial imperfections. This induced multistory out-of-plan buckling of the framed tube wall.

Combination of mentioned effects lead to the buckling of the columns. As a result the upper part of the tower fell through at least one floor height, impacting the lower parts of the tower. This triggered a progressive collapse since the kinetic energy created was, by an order of magnitude larger than the capacity for absorbing energy in the still standing building (Bažant, Z., Verdure, M. 2007).

2.1.3 Mobile crane crash in Stavanger Norway

In February 2013 the driver of the mobile crane lost his consciousness and crashed into a three story timber building in Storhaug Stavanger (Nedrebö R., Jacobsen K, 2013,). This accident is of interest because it is a more realistic accident for Sweden than for example an explosion from a gas leak. As seen in Figure 2.2 the corner of the house is demolished by the mobile crane, but the floor supported by the corner stays intact.

Figure 2.2 Mobile crane crashes into a timber house in Stavanger, Norway (Nedrebö, R., Jacobsen, K. 2013,).

2.2 Resistance against progressive collapse

A building's ability to withstand progressive collapse at a certain accidental load depends on, the use of the building, the type of building members, dimensioning and connections between elements.

When assessing a building's capacity against progressive collapse, several factors need to be included in the analysis, such as:

- Type of accidental load
- Load case (self-weight and imposed load)
- Number of floors
- Construction method
- Production method
- Connections

- Possible alternative pathways for load transfer
- Foundation

These factors can influence both the individual building element's capacity against accidental loads as well as the stability of the structural body after a primary damage.

2.3 Different types of progressive collapse

A progressive collapse can propagate in different directions, often can collapses be divided into vertical and horizontal direction.

A horizontal collapse is linked directly to frame rigidity and is influenced by the type of structure, design of connections and where the load bearing members are placed.

Vertical collapse can be induced for example by loss of one or more loadbearing walls on the bottom floor, which can happen when the accidental load is a collision of some sort. The overhead structure loses its support and thereby progressive collapse spreads up through the building. If the collision takes place further up in the building, the rubble from collapsed floors above can increase the weight on the floor under the collapsed part and thereby start a progressive collapse downwards through the building.

2.4 Loading conditions

If designing a building structure for a relativity large variable load in proportion to the permanent load, the risk of an accidental load happening at the same time, as the maximum variable load, is small. This means that the risk of a progressive collapse is low when an accident occurs if the building is designed for a large variable load.

2.5 Size of the building

In larger buildings with large dimensions there is a greater risk that primary damage lead to big secondary damages. Furthermore, there is often more people staying in these buildings and there is a greater risk of personal injuries.

With increased total height of the building follows an increase in vertical loads on the lower floors. In buildings with load bearing walls the thickness of the members does not increase with increased building height and they are still relatively slender. The possibility for these walls to withstand more loads than the function load and also take an additional accidental load is therefore limited.

In buildings with many stories a wall structure often have the possibility to create alternative load paths after a primary damage. If this possibility is used the load bearing members affected by the alternative load must be designed accordingly.

2.6 Wall structures

In a wall affected by an accidental load in the form of a concentrated hit, from for example a vehicle, a primary damage occurs through punching a hole in the wall or a more extensive damage of the wall. The size of the damage is in the latter case decided by the smallest span of the wall. Walls with large length with respect to floor height can thereby partly still stand after a collision but walls with smaller span might be totally lost.

2.7 Continuity in the structure

Experience from occurred damage has shown that continuity in building structures is a factor that can greatly contribute to decrease the consequence of an accidental load.

Continuity means in this context moment- and force transmitting capability of the building elements themselves and their connections. Continuity must be enough to transmit the forces that can arise from an accidental load. In many cases these forces can be totally different from the forces at normal loading situations, both in size and direction.

Cohesion and continuity are favorable factor with regard to the robustness of the building structure affected by an accidental load. A prerequisite for this is that the capacity for bending the building parts is greater than the bending effort affecting the connections between the elements when the moment is redistributed. An example of this is a wall structure composed of elements acting as a high beam when losing its support beneath, leading to great forces in the connections of the elements, see Figure 2.3.

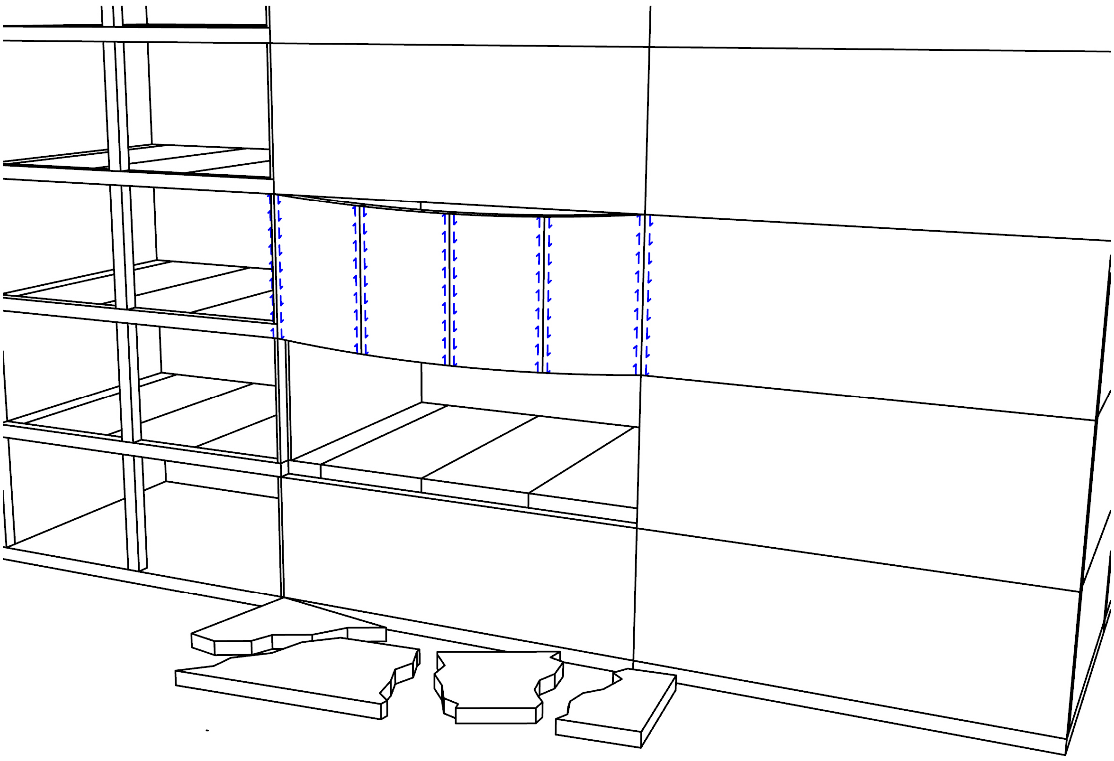


Figure 2.3 Wall structure acting as a high beam after losing support from the wall beneath.

2.8 Connections between building members

Studies of damages in buildings where a construction part is taken out imply that a building's sensitivity to damage is mostly affected by the design of the connections between the elements and not so much affected by the individual building members' strength.

2.8.1 Plates

Connection between plates (floors and walls) is normally affected by shear force and diaphragm action in the floor utilized. In element built structures, see Figure 2.4 the lengthwise joints must be able to hold the floor together and transition shear, see Figure 2.5. These joints can in the situation of damage to the primary load bearing structure be affected by forces in other directions than the main loading direction, see Figure 2.6. These loads can be of great magnitude and need to be considered when the floor structure is a part in the stabilization of the building structure.

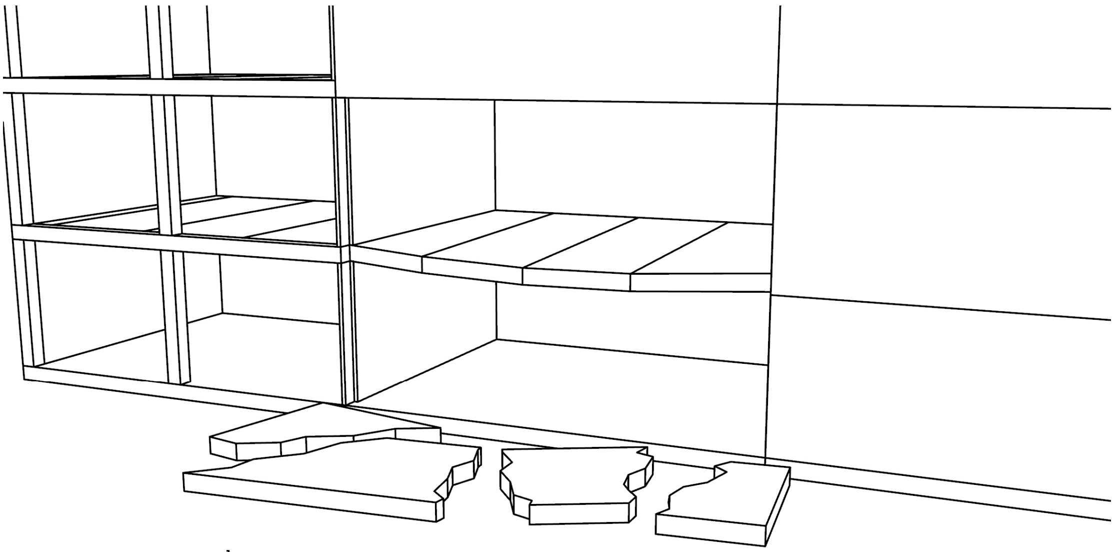


Figure 2.4 Damaged construction composed of building elements.

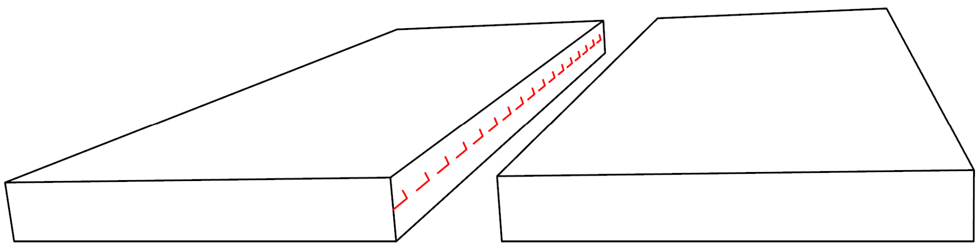


Figure 2.5 Shear force between floor elements, normally dimensioned in the design process when accounting for structural stability.

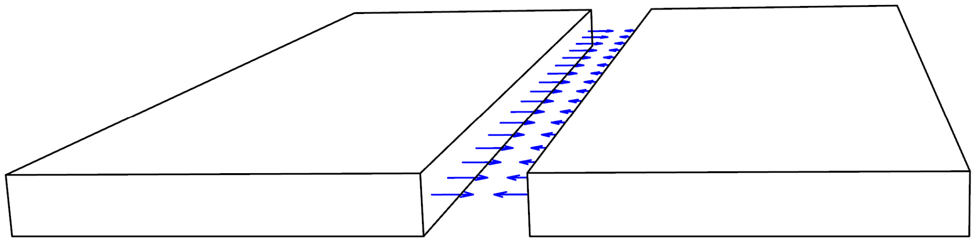


Figure 2.6 Normal forces between floor elements that occurs when the main load provider is lost.

2.8.2 Reserve supports

A possibility to prevent a progressive collapse is to design the building structure to act as a secondary load provider when the main support is lost. This can be done in walls by seeing them as high beams distributing the load to secondary support, see Figure 2.7.

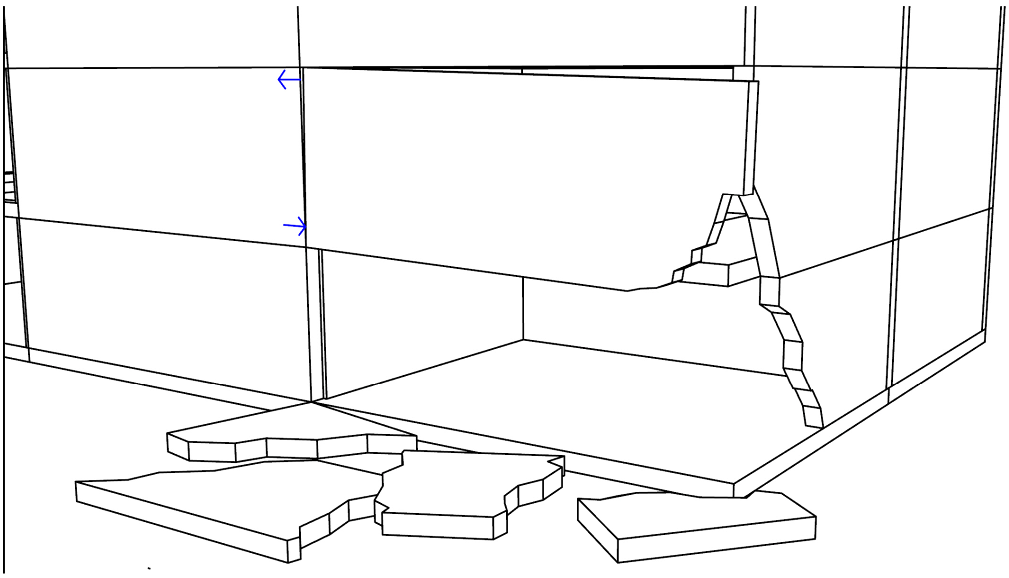


Figure 2.7 Example of a wall acting as a cantilever beam.

Between floor plates connectors can be used to distribute loads orthogonally to the intended loadbearing direction, see Figure 2.8.

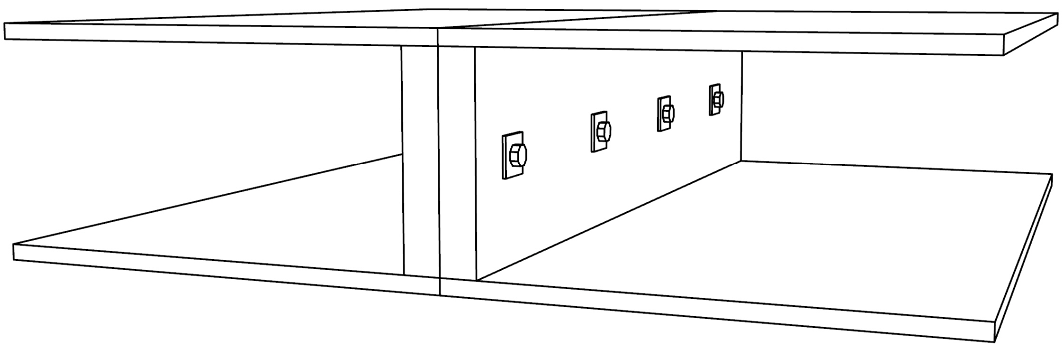


Figure 2.8 Connectors between floor elements.

Beams can be used in the end of the floor plates to act as secondary load providers, distributing load to secondary supports. Here the designer should take great care so that the floor structure does not collapse if the beam is damaged in a possible accident see Figure 2.9.

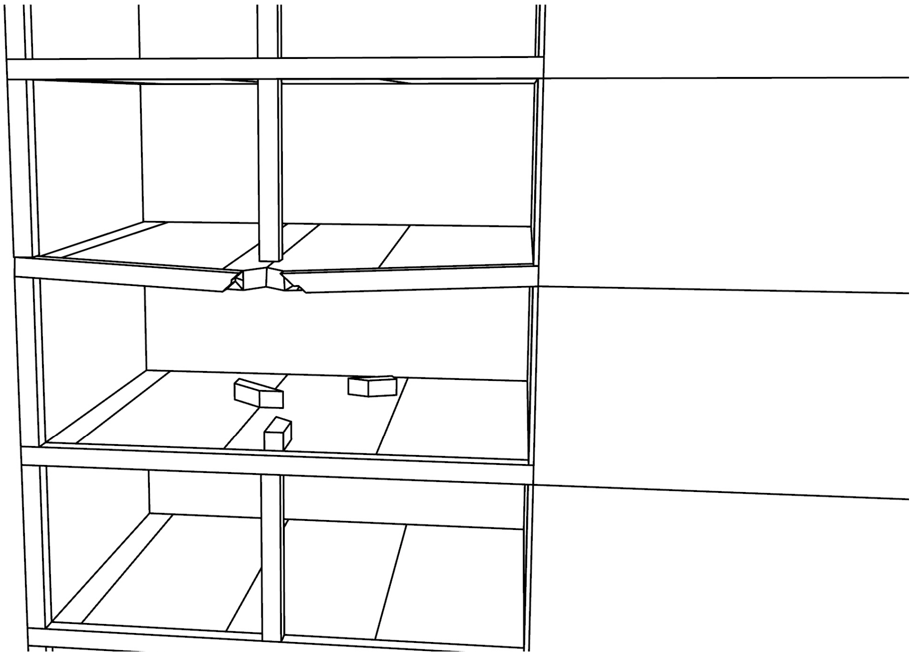


Figure 2.9 Damage to the edge beam.

Interior walls could be designed as secondary support if dimensioned for the loads that can be diverted to it, see Figure 2.10. If the interior wall are dimensioned and used as secondary supports the rest of primary loadbearing structure could be less robust, which can decrease the span in case of a lost exterior wall.

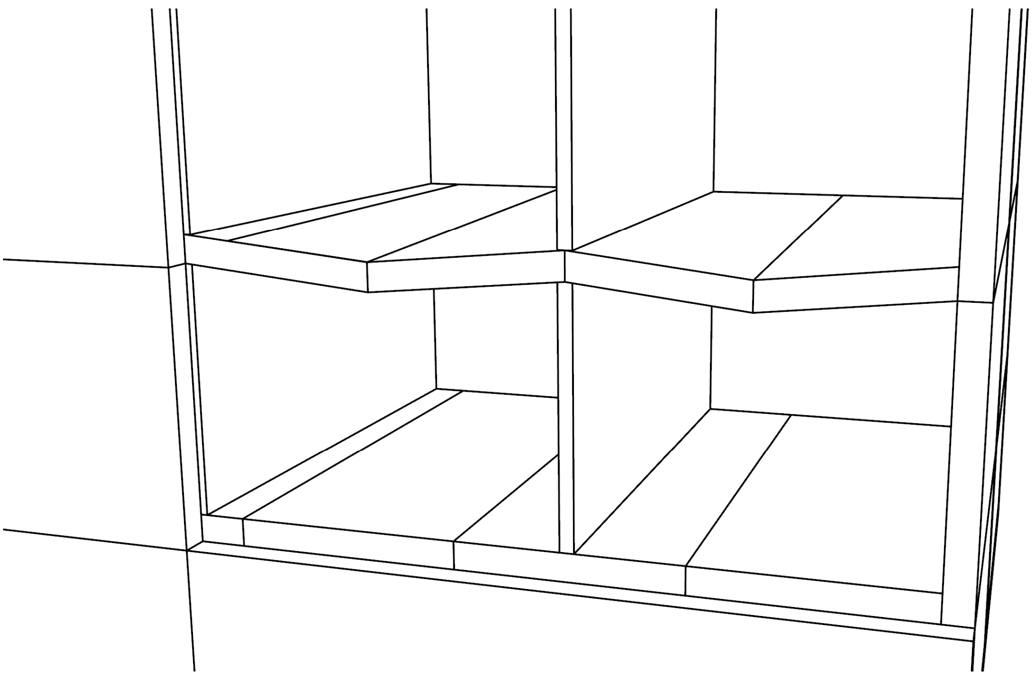


Figure 2.10 Interior wall acts as secondary support when the main support is lost.

2.9 Foundation

An accidental load that affects the foundation might lead to settlements that can cause displacement in the building structure causing a progressive collapse, see Figure 2.11. This scenario is however not so likely since the slab or cellar is normally dimensioned for the large load from the structure itself and when a load bearing wall is lost the building's self-weight is eased on the foundation and only the accidental load remains.

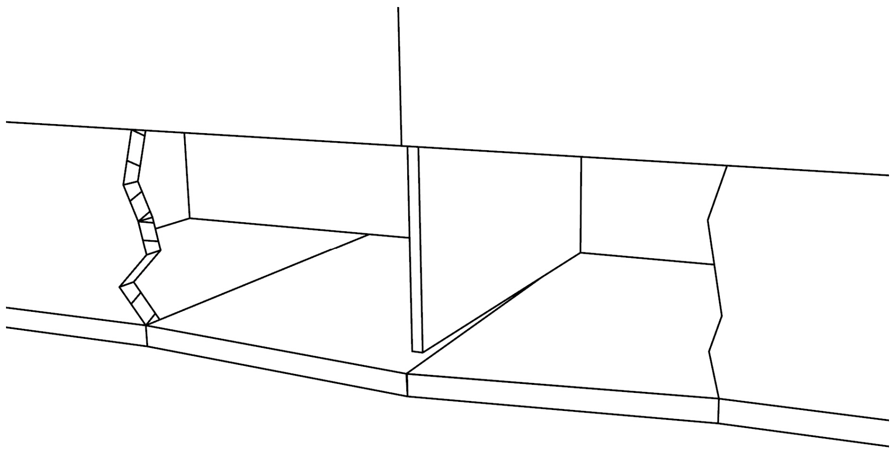


Figure 2.11 Settlement in foundation leading to decreased support to the loadbearing wall.

2.10 Division into accidental zones

When a building has been damaged because of an accidental load, the damage spreads uncontrollably throughout the building and a progressive collapse occurs or is contained when the surrounding structure remains stable. In the latter case, with regard to the injury progress, the different zones in the damaged building can be identified, see Figure 2.12. This division into zones can be an appropriate method when deciding how to analyze the damaged building's structure.

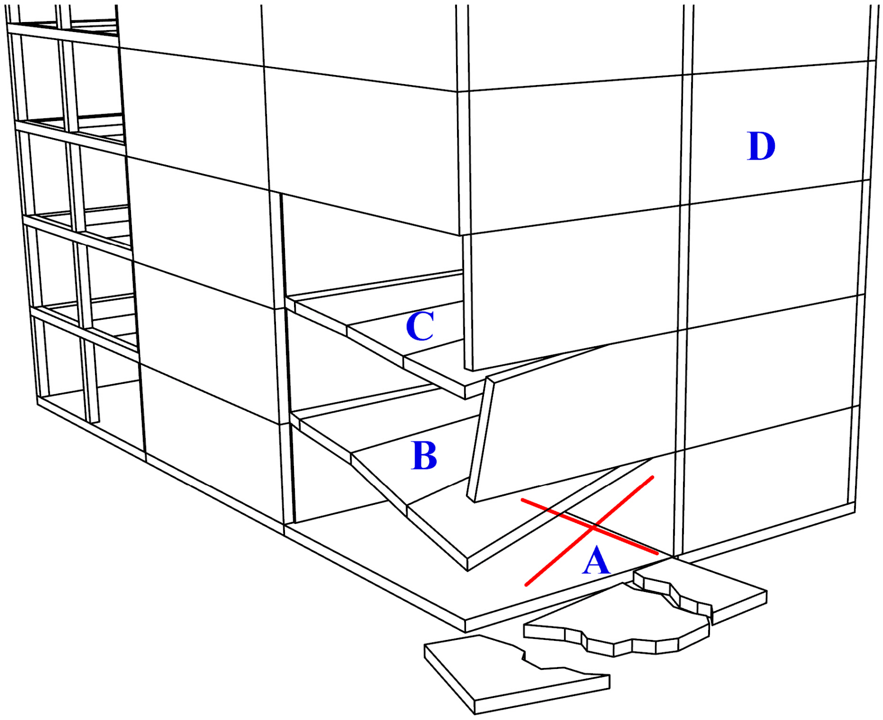


Figure 2.12 Zones in a damaged building due to accidental load. Area for primary damage caused by accidental load, zone A. Secondary damage area with severe risk of personal injury, zone B. Area with large permanent deformations but with insignificant risk of personal injury, zone C. Area were the construction largely remains unaffected by the accidental load and the primary damage, zone D.

In most accidental impacts the effect is relatively local. This applies specially to, for example, shock loads from vehicles. The primary damage, thereby means the damage that is created in the accident, for example a hole in a wall is in this case the affected part of the wall and it is denoted as the primary damage area, zone A.

In some cases the primary damage propagates to the nearest surrounding. Such an added secondary damage, where there is severe risk of personal injury, it is denoted as the secondary damaged area, zone B. Example of secondary damage area is the downfall of a beam when its support is destroyed in an accidental impact. Within the secondary damaged area the construction parts can remain load bearing, lose their load bearing capacity and be an extra load on the damaged structure or totally disappear from the damaged area depending on the circumstances.

Outside the secondary damage, zone C, parts of the construction can have large permanent deformations but there is insignificant risk for human injury. In a zone D, the structure's load bearing capacity remains largely unaffected by the accident.

2.10.1 Primary damage

When considering alternative ways for the structure to distribute loads, estimation about the imaginary damage extent has to be done. When an accidental incident gives limited damage, the area subjected to the damage can be assumed and for this area the control of the building's capacity against progressive collapse can be done.

There are basically three different ways to estimate the size of the damaged area:

1) Consider the structural properties and the dynamic impact of the building members affected by an assumed accidental load.
2) From the weakness and inhomogeneity in the building structure identify possible modes of fracture. Example of this can be junctions between plates and holes in the structure.
3) The proportion between the size of the damage and the building size should be within a certain range. This range has to be nuanced considering which kind of building members that the building is comprised of.

2.10.2 Secondary damage area

To prevent the spreading of the primary damage there is need for drastic measures, which are often impossible or unreasonably demanding. It is however essential that the structural members in the surroundings of the damage, contribute in preventing a further spread that leads to a progressive collapse. In plate constructions the secondary damage area can for example include one or more room units that surround the primary damaged structural part. The area's limitations are comprised of intact joints.

The recommendations for the limits of the secondary damage vary for different kind of buildings. Recommended size is the smallest of 15 % of the floor area or 100 m^2 in each of two adjacent stories. The secondary damage extent in sideways can be assumed to coincide with the natural weaknesses in the structure for example between free edges and joints to other structural members. The extent in sideways can often be assumed to coincide with the primary damage in one floor.

The more building members in the secondary damage area that lose their load bearing capacity, the harder it becomes to achieve alternative load pathways and thereby secure the robustness of the overall structure. Within the secondary damage area can severe personal injuries can be expected due to beams and floors falling down when these building members collapse. To limit the risk of personal injuries the secondary damage area should not be allowed to become too large, even though it does not jeopardize the overall structural stability.

3 Design for robust buildings

To avoid the risk for progressive collapse there is a need to consider the robustness of the whole structure. This means the structure's ability to redistribute loads when the primary load taker is incapable of transferring its associated load or the load taker itself is dimensioned to withstand the accidental load. Guidelines and methods for the analysis of a building's capacity to resist progressive collapse are presented in Eurocode 1 section 7.

3.1 Eurocode 1

Section 7 gives basically two alternative methods for analysing the robustness of a structure. The first alternative is to use a method based on known accidental load. The second method is based on limiting a local rupture. Since the magnitude of the eventual accidental load is unknown the methods for limiting a local rupture are given in Annex A, will be further described.

3.2 Annex A

Annex A gives advice and instructions on how to design buildings to withstand a local rupture without unproportioned large collapse. The recommended size of a local rupture is the smallest of 15 % of the floor area or 100 m^2 in each two adjacent floors, see Figure 3.1.

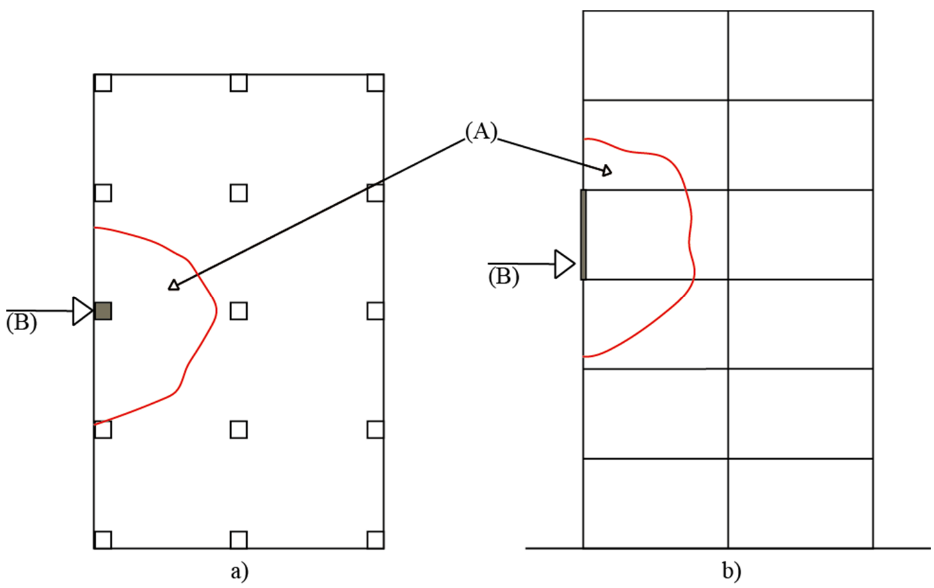


Figure 3.1 Recommended limits for allowed damage.

a) Plan b) Section

A. Local rupture that do not exceed 15 % of the floor area in two adjacent storeys.

B. Column that is assumed to be removed

Buildings equal or higher than five stories fall into consequence class 2b, which gives that in addition to the methods in consequence class 1, two separate choices for designing with progressive collapse in mind.

First choice for a building with load bearing walls, the floor should have continuous horizontal ties in the outer edge of the floor system, within a 1200 mm wide area measured from the edge of the floor, see Figure 3.2.

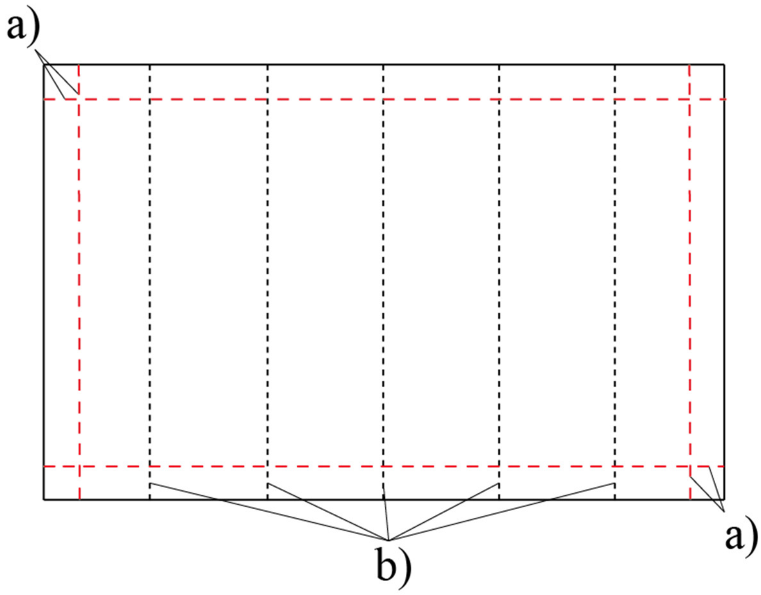


Figure 3.2 Displaying horizontal tie: a) External ties 1200 mm from the edge of the floor and b) Internal ties.

Horizontal internal ties should be evenly distributed in the floor system in two orthogonal directions. Dimensioning tension force in ties should be calculated in the following way:

For horizontal internal ties T_l is equal to the larges value of F_t kN/m and

$$\frac{F_\text{t}(g_\text{k}+\Psi * q_\text{k})}{7.5}\frac{z}{5} \quad \text{kN/m} \tag{1}$$

For horizontal exterior ties $T_\text{p} = F_\text{t}$ (2)

Were:

F_t is the smallest of 60 kN/m and $20 + 4n$ kN/m

n is number of floors

z is the smallest of: Five times floor height H, and the largest distance in meter in direction of the horizontal ties measured between center of a column or other structural components for vertical loads. Independent of whether this distance spans over a single plate or a system of beams and plates, see Figure 3.3.

The partial factor Ψ should be taken equal to $\Psi_1 = 0.5$ in equation (6.11b) in Eurocode 0 according to (Westerberg, B. 2010).

$q_\mathrm{k} = 2.0$ kN/m For residential buildings according to Eurocode 1.

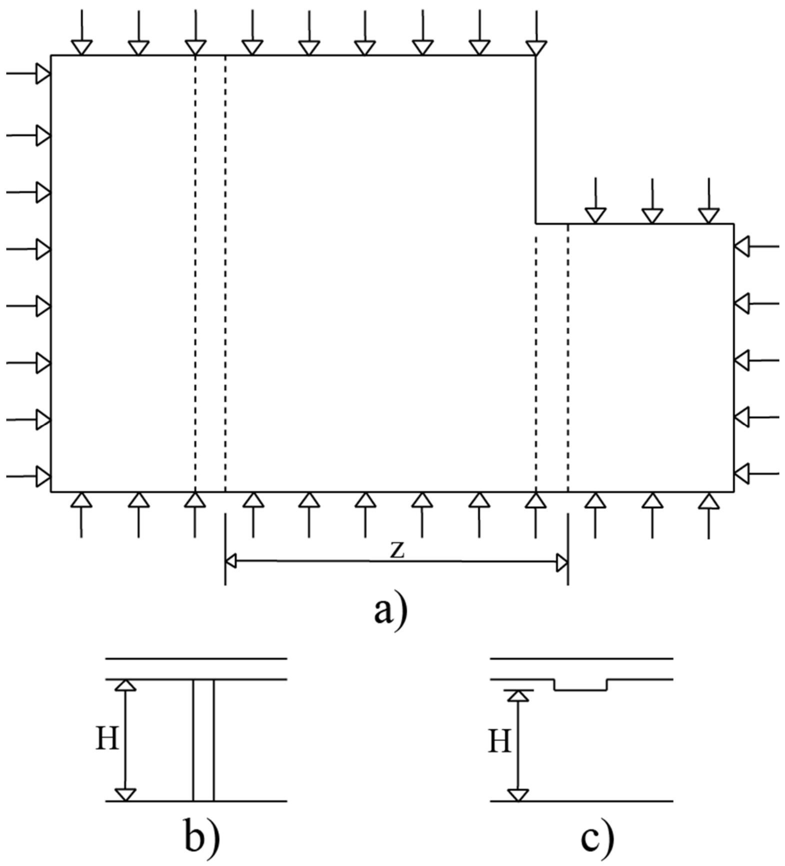


Figure 3.3 Illustrations of the factors H and z

Second alternative is to control that the building remains stable and that a local rupture does not exceed a certain level for the hypothetical case. For timber stud walls the recommended length of the wall to be removed is between two edge supports from other vertical building members, see Figure 3.4. The appropriate load to use when analysing the floor system is:

$$q_\mathrm{d} = \mathrm{g_k} + \Psi * \mathrm{q_k} \quad (3)$$

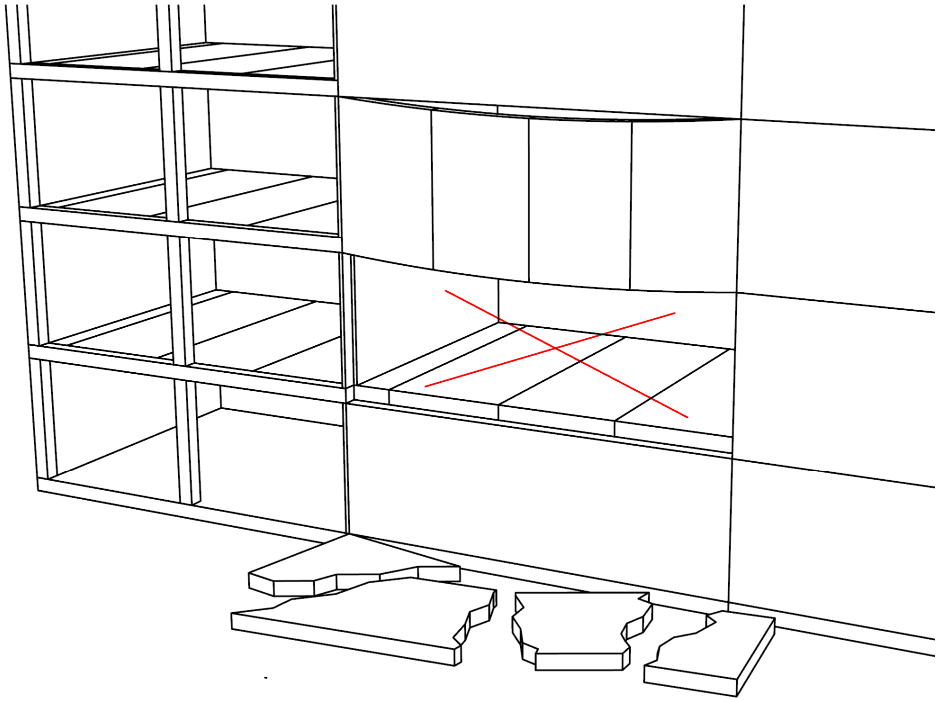


Figure 3.4 Eliminated wall between two vertical edge supports.

3.3 Massive-timber handbook

It is normally not reasonable to design a building system in massive-timber, to cope with a direct impact from an accidental load. Instead the structural system should be design so that separate building elements could be disabled without causing progressive collapse. When design for progressive collapse the overall stability of the structure should be controlled (Martinsons 2006).

Total stability of massive-timber structures after a primary damage is often satisfactory since there normally is a number of load carrying elements. The possibility for load redistribution is therefore often enough (Martinsons 2006).

The building's cohesion can be controlled in, design with two in plane perpendicular shear and normal forces at the joints between building members. These forces are for concrete structures *N*=20 kN/m and *T*=20 kN/m (Boverket 1994) according to the old and outdated building code in Sweden. For buildings of massive timber can these forces can be scaled down due to the in comparison low self-weight of massive-timber structures (Martinsons 2006). The proportionating can be done accordingly:

$$N = T = 20 * \frac{g_{\text{k\_timber}} + \Psi * q_{\text{k}}}{g_{\text{k\_concrete}} + \Psi * q_{\text{k}}} \qquad (4)$$

4 Description of the reference project.

Since A-hus uses the same principle building system in their multi-story buildings, the problematic areas are likely to occur at the same places in all the buildings using this system.[2] To analyze the floor system an existing building is evaluated. It is a five story apartment building and hence subjected to consequence class 2b according to Eurocode 1, which implies that the structure should remain stable even if a loadbearing wall is removed.

4.1 Building system

The building system A-hus uses to build multi-story buildings is prefabricated blocks built in their own factory. There are mainly three groups of elements, exterior wall elements, load bearing interior wall elements and floor elements. These building blocks are presented below.

4.1.1 Exterior walls

The exterior walls are built as an ordinary stud-wall and are composed of a 45x170 mm sole plate and top rail and 45x170 studs at spacing 600 mm. The frame is cladded on both sides with 11 mm thick OSB and filled with insulation, see Figure 4.1. The height of these walls varies between 2700 mm and 3000 mm. Maximum length of these wall elements are 7000mm.

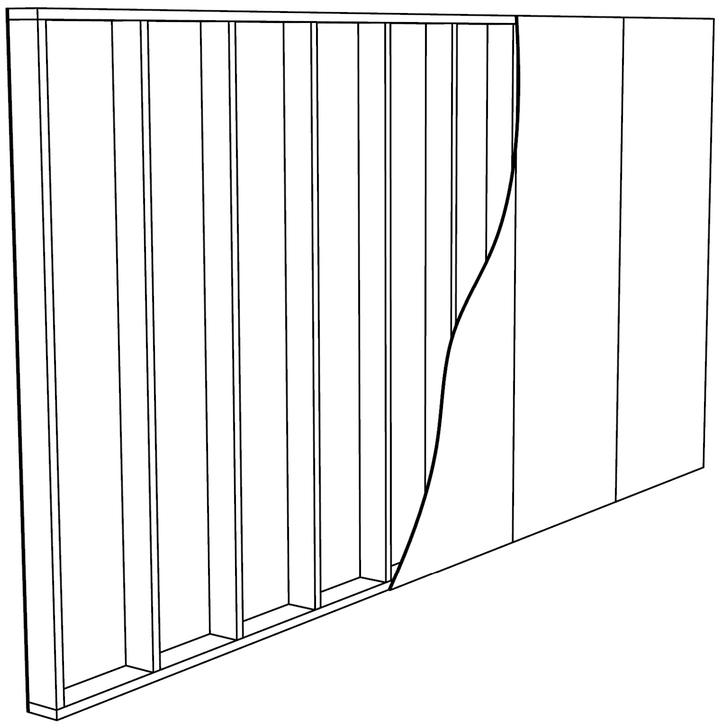


Figure 4.1 Exterior wall element.

[2] Meeting with Linus Abrahamsson designer at A-hus 2016-01-27

4.1.2 Loadbearing interior walls

The load bearing interior walls are made with a 38 mm thick particleboard as the core, on both sides of the particleboard 45 x 95 mm studs at spacing 600 mm are glued and nailed. These walls can have the same height and width as the exterior walls, see Figure 4.2.

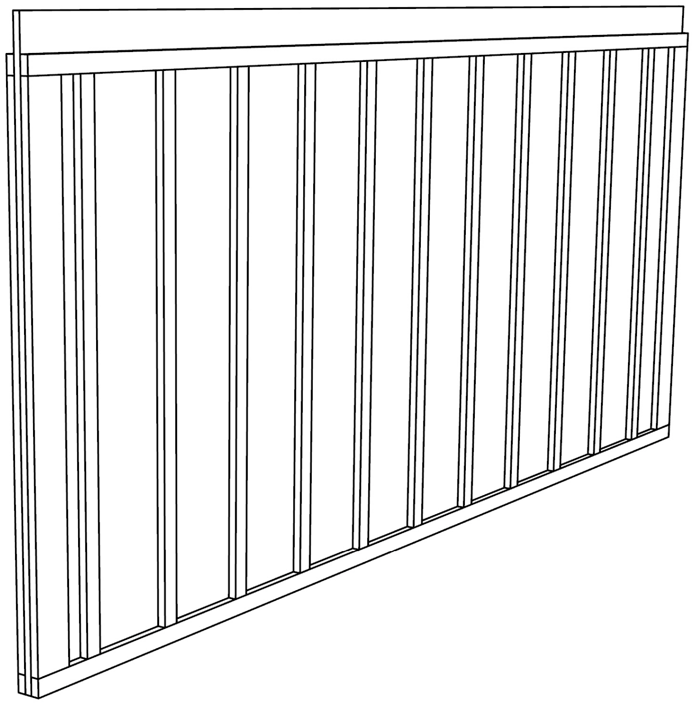


Figure 4.2 Load bearing interior wall element.

4.1.3 Floor system

Floor system is composed of elements that are mounted on the building site, these elements are lowered in place by a crane and weigh 1.0 kN/m^2. Joints between elements are screwed and along the whole edge of the floor screws are mounted to the walls, see Figure 4.3.

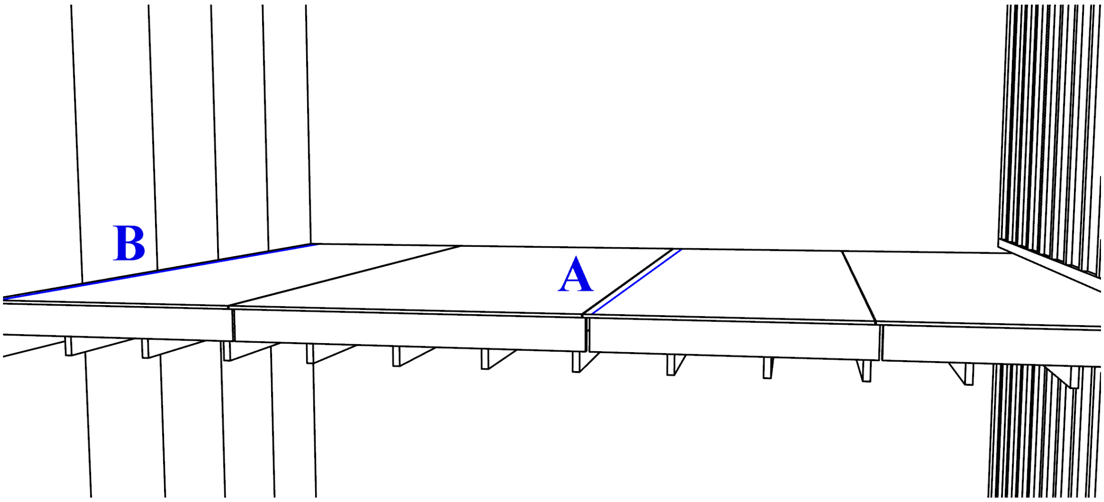


Figure 4.3 Floor structure. Attachment between floor elements (A) and attachment along the edge of the floor (B) to the exterior wall.

Floor blocks consists of 45x360 mm Kerto-S beams covered on the top with a 22 mm particleboard that are glued and insulated with 220 mm insulation. The maximum span of these elements are 6000 mm and maximum width are 2400 mm, see Figure 4.4.

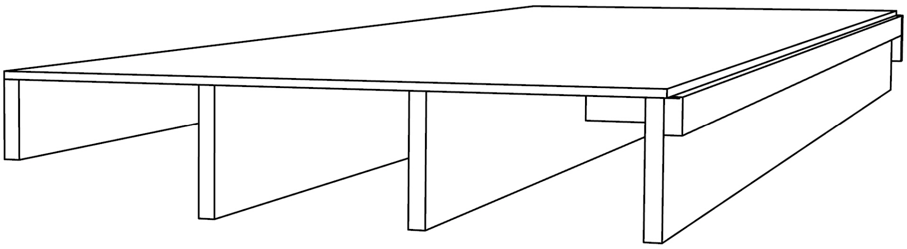


Figure 4.4 Edge floor element.

4.2 Analysis of reference building

The analysis of the building’s ability to cope with a removal of a wall will be done on the floor structure since there is an obvious lack in continuity between the elements that constitutes the floor, see Figure 4.5.

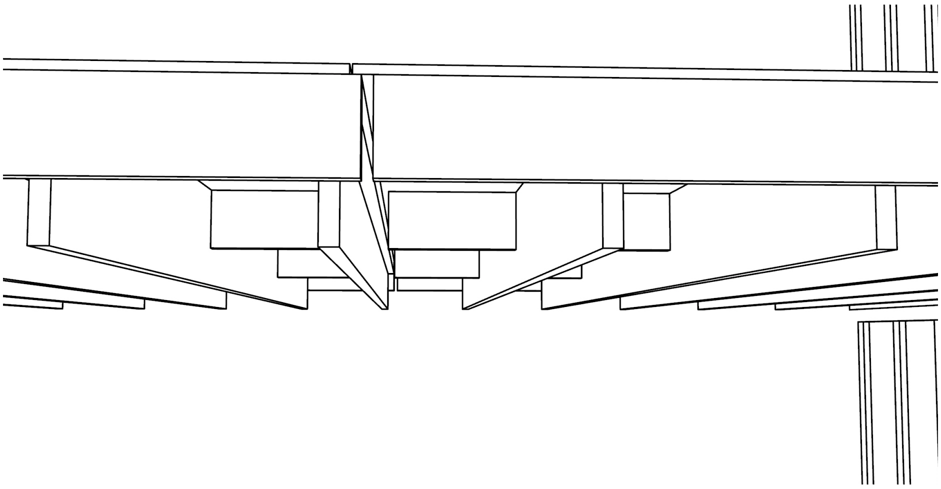


Figure 4.5 Lack in continuity between floor elements.

Especially in the wall that constitutes the support, see Figure 4.6, when the wall structure is removed there will be no timber beam to redistribute the loads on the floor via an alternative pathway, see Figure 4.7.

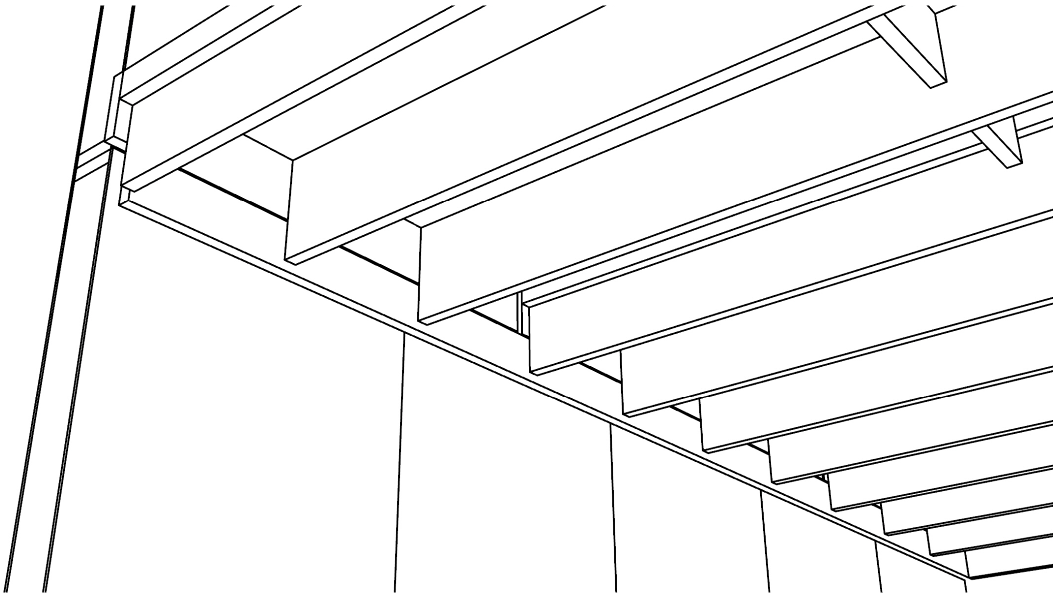


Figure 4.6 Displaying support for beams that constitutes the floors load bearing structure.

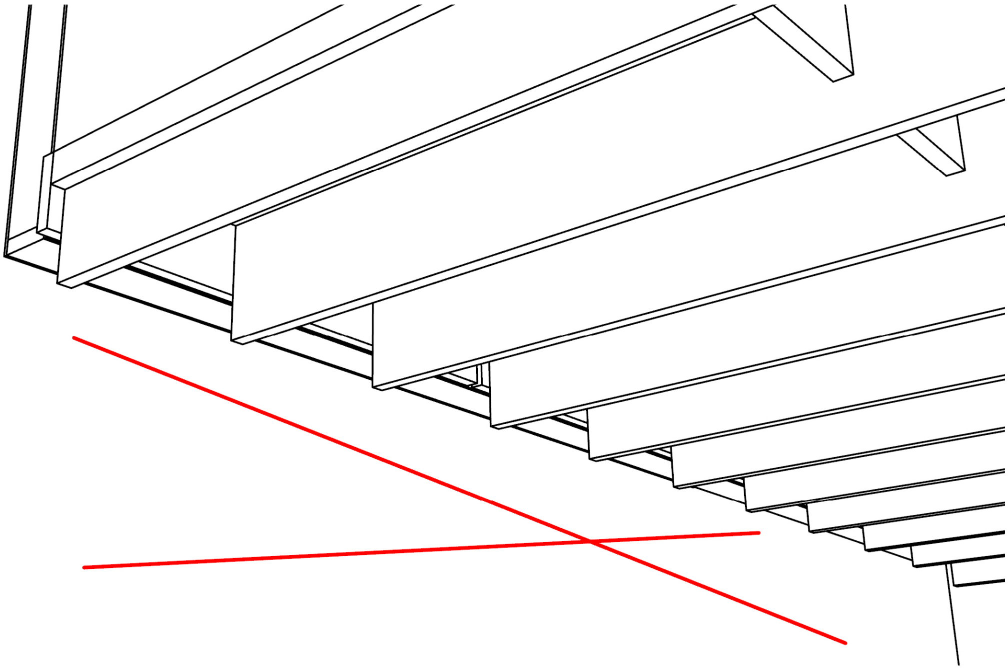


Figure 4.7 Support is lost and there is no edge beam left to redistribute the load.

The floor structure can be seen as a secondary damage area with severe risk of personal injury. Since joints between the floor elements govern the capacity for creating an alternative pathway for load redistribution a further analysis is desirable, to make sure that in the event of an accidental load damaging the building progressive collapse does not occur.

5 FE-model of existing floor

The FE-program Abaqus/CAE 6.13-3 is used to make a model of the existing floor, to investigate the consequence if one load bearing wall were to be removed, according to Eurocode 1, see Section 3.2. In previous Section 4.1.3 the floor system is described in detail, but when modelling some simplifications were made.

5.1 Description of the model

The total dimensions of the floor are 7800 x 6000 mm and all the parts will be modelled as 3-dimesional deformable shell elements. Since shell elements do not have any width, the height of Kerto-S beams, noggins and edge beams have to be increased with half of the thickness of the particle board.

The particle board is modelled as a whole plate with dimensions of 7800 x 6000 mm. All the Kerto-S beams are modelled as 371 mm high and 6000 mm long and they are placed with a distance of 600 mm. Noggins are placed between the Kerto-S beams with a distance of 1800 mm from edge. Since the floor is made up of four floor elements, the noggins between floor elements are shortened with 50 mm to 550 mm. This is due to how the elements are attached to each other, see Figure 4.5. The same principle is used when modelling edge beams on the long sides of the floor. The floor is supported on edge beams made of Kerto-S 371 mm high on two sides and timber C24 231 mm high on the other two, see Figure 5.1.

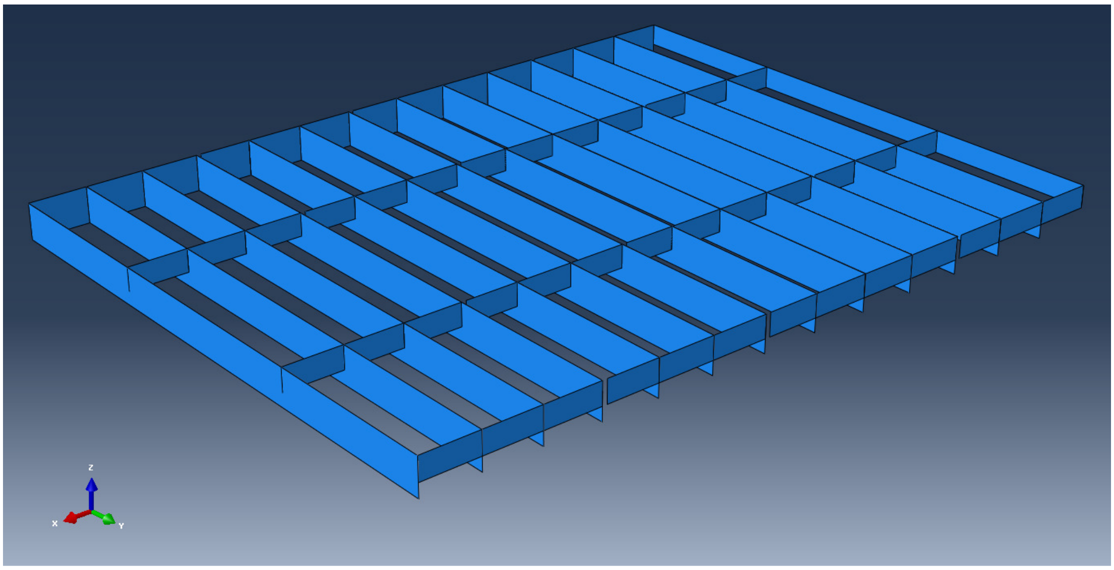


Figure 5.1 Model from Abaqus where the particle board has been removed to make it easy to see how all parts are attached to each other.

The material properties are given in Table 5.1, where Kerto-S and Timber C24 are modelled as a lamina material and the particle board is modelled as an isotopic material. The recommend values of elastic modules and poisons ratio for the particle boards are 1800 GPa and 0.35 (Brandin, J., Oscarsson, A. 2015). The local coordinates for Kerto-S and the timber beams are x-axis is along the length and y-axis is along the height of the beam.

Table 5.1 The different materials and their properties. E1 correspond to the elastic modules in x-direction and E2 y-direction.

Material	Mean density [kg/m^3]	E1/E [GPa]	E2 [GPa]	Nu12/ν [-]	G12 [GPa]	G13 [GPa]	G23 [GPa]
Kerto-S	510	13.8	0.43	0.02	0.6	0.15	0.05
Timber C24	420	11	0.37	0.03	0.3	0.69	0.03
Particle board P5	632.5	1.8		0.35			

The step module is static general for every simulation that has been made during this project. To assemble all the parts together constraints type has been chosen to tie constraints. Walls are assumed to be rigid hence boundary conditions are pined connections on four sides for verification of the model and three sides for the studied case. The load is uniformly distributed according to Equation (3) $q_d = 2.0\ kN/m^2$ load acting in negative z-direction on the particle board. Mesh type is quad structured mesh.

5.2 Verifying with hand-calculations

To see if the model works, a simple hand-calculation of how large the deflection at the centre of the floor. In the calculation some simplifications are made, such as only looking at a 600 mm section of the floor with a top flange of particle board and below it' a Kerto-S beam. Both deflection from bending and shear are calculated and the sum of them is 7.0 mm, the whole calculation can be found in Appendix A. The result from Abaqus is a deflection of 6.9 mm (mesh size 50 mm), see Figure 5.2 which is near the result from the hand-calculations.

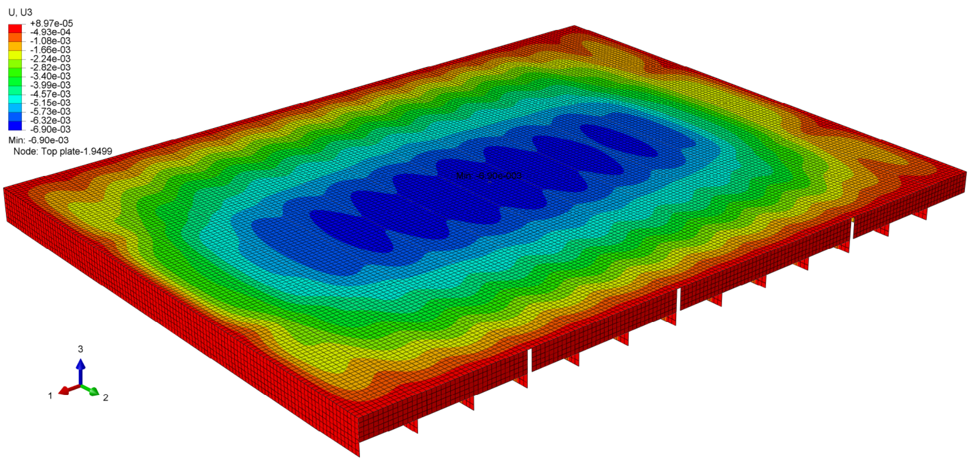


Figure 5.2 Floor simply supported on four sides loaded with a uniformed distributed load of 2.0 kN/m^2. The colour indicate how large the deflection is.

5.3 Mesh and convergence

A convergence study is carried out by decreasing element size by half for every analysis and calculate the difference between new and old maximum deflection.

Based on Table 5.2 the model converges when the element size is 50 mm and therefore this size is chosen for the rest of this report.

Table 5.2 Values from the convergence study.

Mesh size [mm]	200	100	50	25	12.5
Deflection [mm]	7.83	7.04	6.90	6.85	6.84
Diff. [%]		10.0	2.0	0.7	0.15

5.4 Result from the analysis

Removing the boundary condition on one side of the floor from the verified model, will result in a case that is interesting for this thesis. Contour plot of the stress shows that the stresses in the particle board are high at the place where floor elements are joined together. Based on EN 12369-1:2001 particle boards can withstand a tension stress of 6.9 MPa and from the FE-model maximum tension stress in the particle board is 22 MPa, see Figure 5.3.

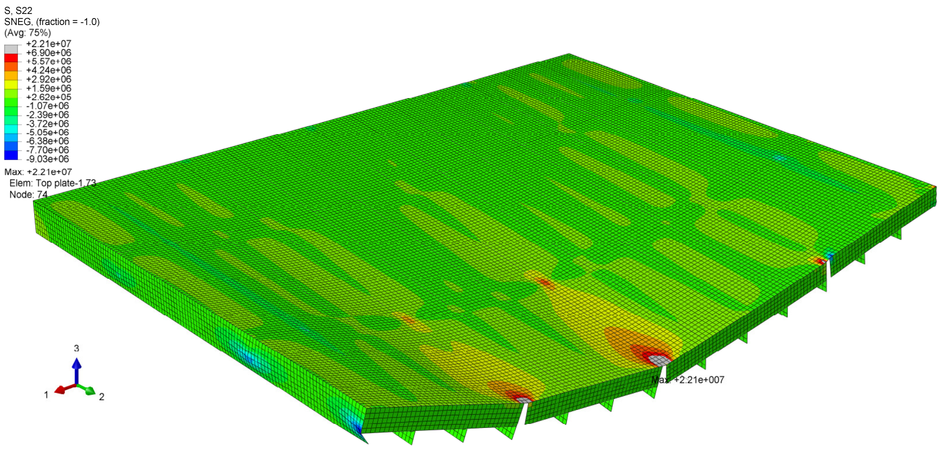


Figure 5.3 Contour plot from Abaqus where one support is removed, the color indicates how large stresses there are in the floor in 2-direction. The deflection is amplified by a factor of five.

6 Evaluation of results from FE simulation

From the results in the FE analysis it is clear that in an eventual collapse of the wall that support the floor, there will be large stress concentration at the joints between the floor elements. This stress concentration is larger than the tensile stress capacity of the particle board, which implies that it will fail. Since the particle board is modelled as one continuous unit and not as it built is in reality. Joints that experience large stress concentration are not continuous. Therefore have better structural capacity than they would normally have.

Due to the large stress concentrations and the lack in continuity it is likely that the joints will fail in some way. Although there are uncertainties in how large this failure will be and how it will influence the particleboard's ability to redistribute load. It will certainly affect the particleboard and it is questionable if the floor system has a secure secondary load path.

6.1 Improvements

There is a need to ensure that there is continuity between the floor elements so that a secondary load path can be created in case of an accident that severely damages the main support.

6.2 Different ways to achieve continuity between elements

There are several different ways to achieve continuity between the floor elements. Three solutions are presented below and to evaluate their suitability a multi-criteria analysis is performed. For guidance in the process, criteria for the improved floor system is developed.

6.2.1 Criteria for improvements to the floor system

The aim with these criteria is to fulfill the demands from both the theoretical design process as well as provide a reasonable approach that can be implemented in the building system. Criteria for development of the improved floor system are:

1) Fulfill the recommendations outlined in Eurocode 1, Annex A. To ensure that there is continuity in the event of an accident a zone 1200 mm from the edge of the floor system, must be able to transmit the forces that will act on it and prevent a progressive collapse.
2) Provide sufficient availability to fit installations inside the floor elements. For example pipes for pluming.
3) Resemble the original solution as much as possible, so that large changes in the production and design of the floor elements will not be necessary.
4) Easy to manufacture.
5) Easy to assemble at the building site.

6.2.2 Solution A

Changing the frame to contain only Kerto-S beams and placing noggins narrower at the edge, creates a stiffer floor system that might be able to carry the load transverse to the primary load path, see Figure 6.1. In this solution it is vital that the connectors between each single building-element can transmit the forces that arise if the floor's primary support is lost. For example connectors between floor joists and noggins must be designed to have enough withdrawal capacity to create a secondary load path.

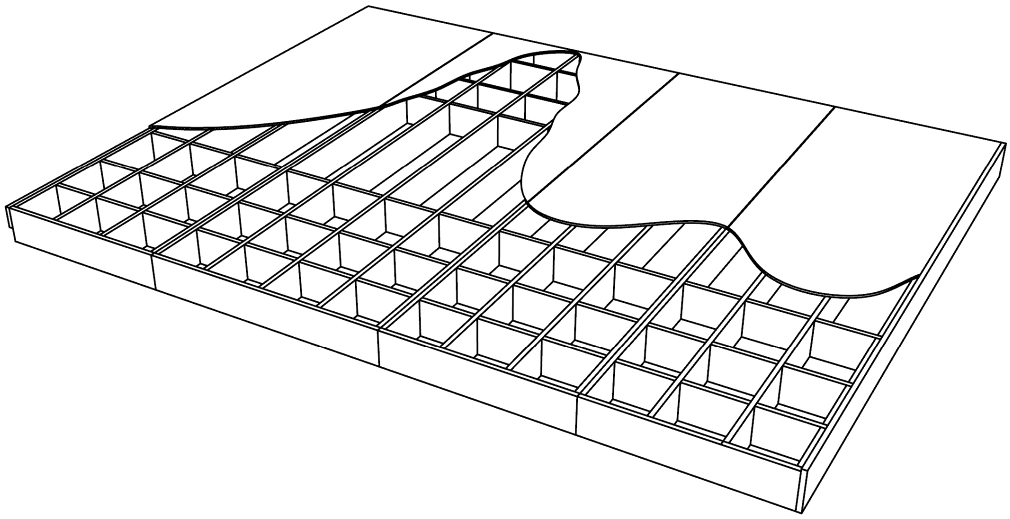


Figure 6.1 Floor system composing of Kerto-S beams.

6.2.3 Solution B

Secondary solution is to have weir ropes running through the floor elements, spread at a distance of 1200 mm from the edge, supporting it if a progressive collapse occur, see Figure 6.2. In the event of an accident to the main support, all the force acting on the floor will be transmitted to the supports and the wire has to be anchored accordingly.

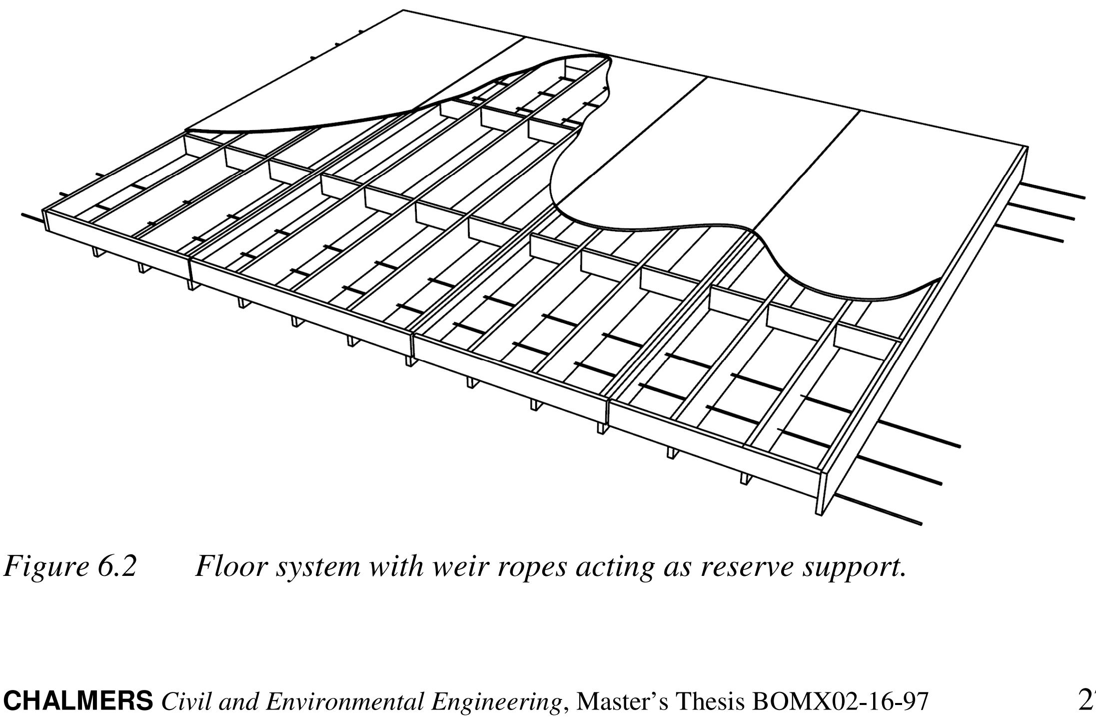


Figure 6.2 Floor system with weir ropes acting as reserve support.

6.2.4 Solution C

Third solution is plywood boards mounted under the floor structure, redistributing the forces to the supports ensuring a secondary load path, see Figure 6.3. Since the plywood boards come with a standard length, joints between each single board must be designed for shear force.

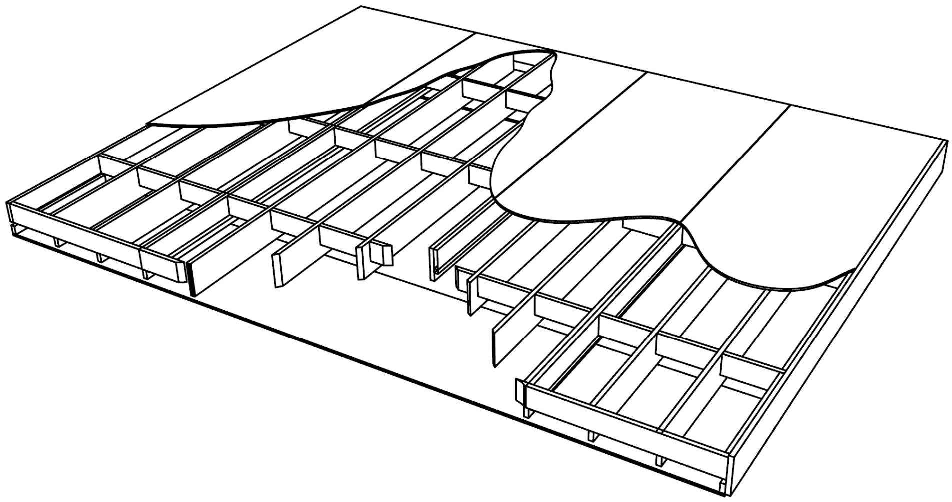


Figure 6.3 Floor system reinforced with plywood boards.

6.3 Evaluation of solutions

As parameters in the evaluation of the three solutions, the five criteria listed below are used. First the parameters are ranked with respect to each other to get an order of priority, see Table 6.1

Table 6.1 Ranking of chosen parameters.

Criteria						Ranking	Weight
Fulfill the recommendations outlined in Eurocode 1, Annex A.	0	+	+	+	+	1	0.4
Provide sufficient availability to fit installations inside the floor elements.	-	0	+	+	+	2	0.275
Resemble the original solution as much as possible.	-	-	0	-	-	5	0.05
Easy to manufacture.	-	-	+	0	-	4	0.1
Easy to assemble at the building site.	-	-	+	+	0	3	0.175

For guidance in the choice of floor system the three solutions are graded in a scale from 1 to 3 and with the weights from Table 6.1, see Table 6.2.

Table 6.2 Graded solutions.

Solution	A	B	C
Fulfill the recommendations outlined in Eurocode 1, Annex A.	1	3	3
Provide sufficient availability to fit installations inside the floor elements.	1	3	3
Resemble the original solution as much as possible.	2	2	2
Easy to manufacture.	1	2	3
Easy to assemble at the building site.	2	1	3
Summation	1.225	2.5	2.95

The main drawbacks of solution A is the difficulty to achieve structural robustness. Connectors between floor joists and noggins have to have a large withdrawal capacity, which will be hard to achieve since they will be fixed in the end grain of the noggins. Another sensitive part will be in the joint where the edge beams meet, due to the small edge distance.

Solution B has a lot of benefits, for example wires are strong and can easily be dimensioned for the loads that might be expected. However anchoring wires to the building structure in an effective way will be a challenge. Also running the wires trough the floor joists can be time consuming.

The evaluation of the solutions suggests solution C as being the most appropriate. With good possibilities to achieve a secondary load path and ability to get sufficient cohesion between floor elements. The possibility to fit installations inside the floor system is good since the plywood boards are installed on the building site.

6.4 Final solution

In the context of the analysis above solution C has promising potential and the key aspects of this system will be further described below.

To make sure that there is enough cohesion between the floor elements, joints between elements are designed according to Figure 6.4. Since the elements are mounted close together there is increased potential for continuity in the upper part of the floor structure.

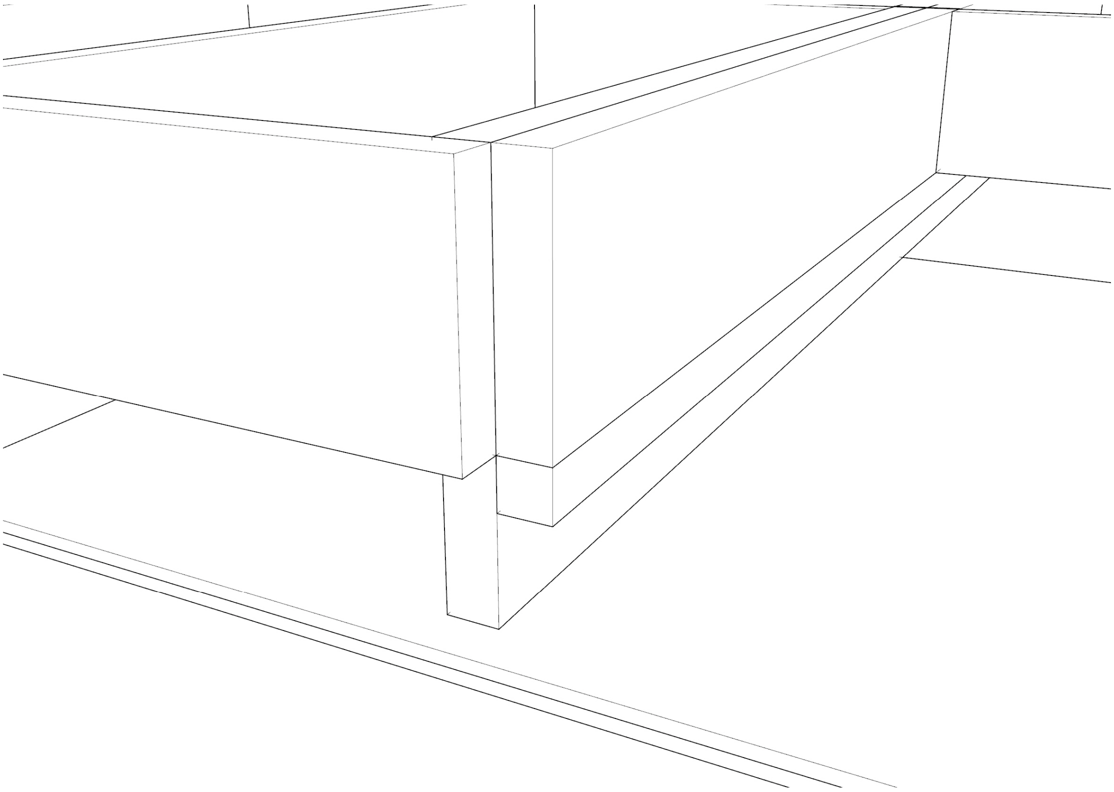


*Figure 6.4 Joint between floor elements. On the side of a floor element a 45*45 mm timber beam are mounted on the Kerto-S beam. The end floor joist a 45*220 mm timber beam on the oncoming floor element are placed on top of the 45*45 mm timber stud.*

Under the floor structure two layers of 12 mm plywood boards are installed, see Figure 6.5 at the building site, creating a horizontal tie which can contribute to secure a secondary load path.

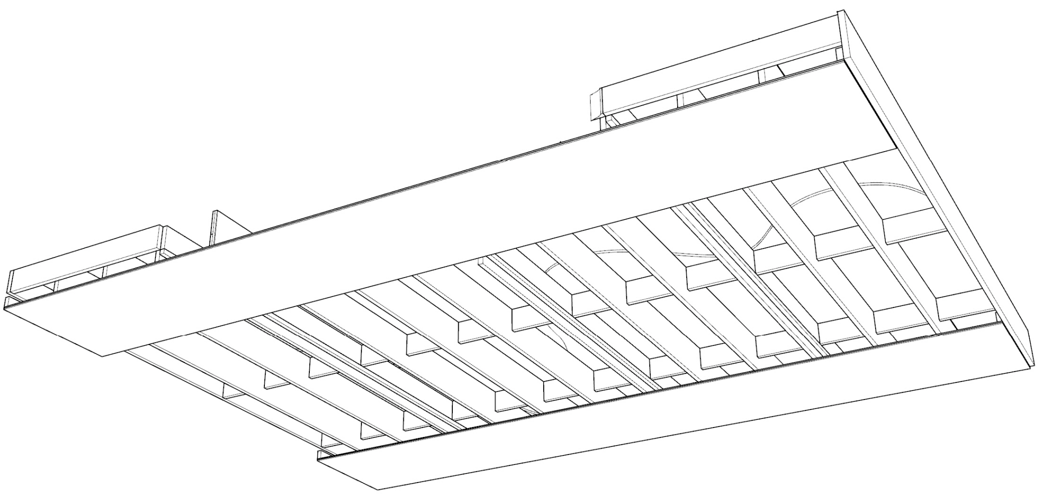


Figure 6.5 Plywood boards mounted under the floor construction.

7 FE-models of improved floor

To make a realistic model of this improved floor it is divided into two models. Where one model has continuous noggins and edge beams and the other is without noggins and edge beams. The data from these models is then compared and the average values are used designing connections later in this thesis. The materials used in the models are the same as for the existing floor but adding 2*12 mm plywood to the underneath the floor. Properties of plywood are listed in Table 7.1.

Table 7.1 Material properties of Swedish plywood that are used in in the FE-models.

Material	Mean density [kg/m^3]	E1 [GPa]	E2 [GPa]	Nu12 [-]	G12 [GPa]	G13 [GPa]	G23 [GPa]
Plywood S	460	7.2	4.8	0.5	0.5	0.5	0.15

7.1 Evaluation of the simulation

The model with edge beams and noggins has large tension stresses in these parts, see Figure 7.1. Data is extracted by creating paths at the worst place (at the middle of the span) from Abaqus into an Excel sheet and then the stresses are recalculated into a tension force and compression force. The result was a 45 kN tension force and a 10 kN compressive force for the edge beam. For noggin the result was a 31 kN tension force and a 10 kN compressive force, see Appendix B.

The problem with this model of the floor system is in this case the tension force, since the edge beams and noggins are not continuous over the span and withdrawal capacity in end grain are poor. So connecting these members together to withstand these enormous forces is almost impossible.

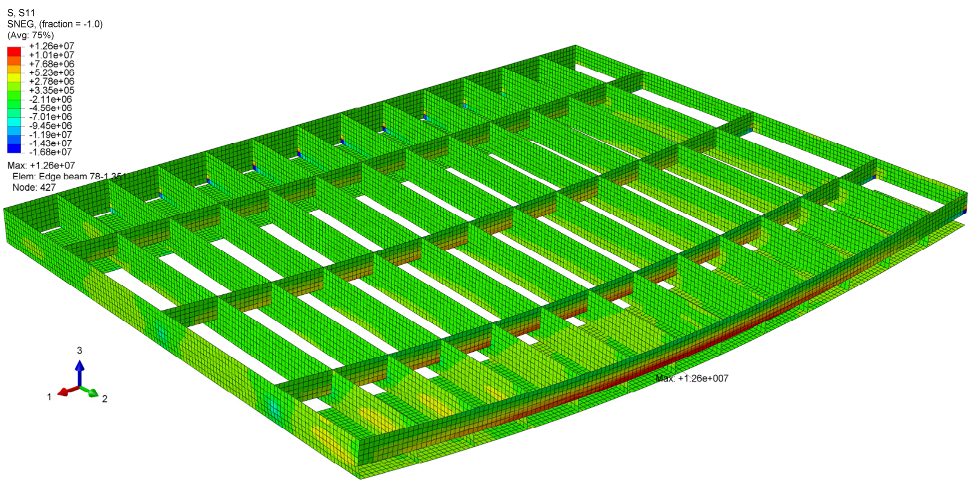


Figure 7.1 FE-model of the improved floor with edge beams and noggins but without top plate to make it easier to see all the parts of the model. Where stresses in 1-direction are shown by the colours of the legend. Maximum stress is 12.6 MPa.

With that in mind another model was made without edge beams and noggins, to see if the floor would manage the load without these parts, see Figure 7.2. The result showed that the floor structure has enough capacity to withstand the load. Some high stresses were created due to the boundary conditions at the edge of the Kerto-S beam, hence the high values in the legend in Figure 7.2.

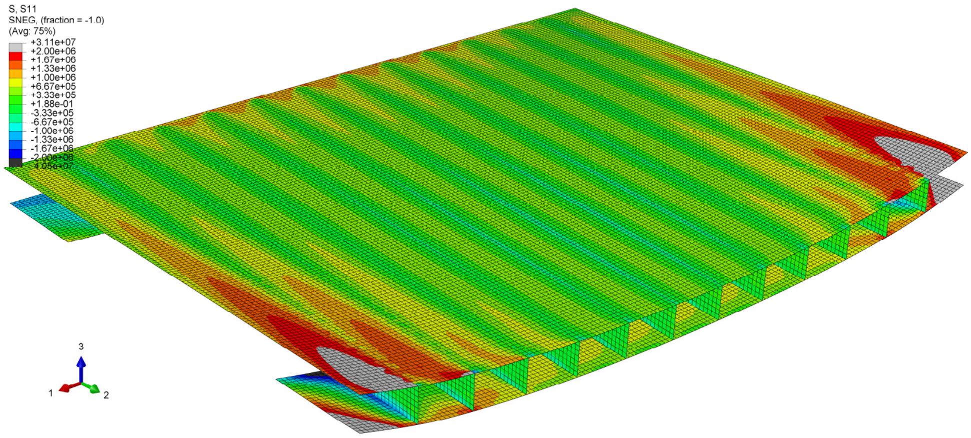


Figure 7.2 Model without edge beams and noggins, were stresses in 1-direction are shown by the colours of the legend. Maximum stress is set to 2 MPa and over that value the colure is light grey. Minimum value is set to -2 MPa and below that value the colour is dark grey. This is made to make a better visualisation of how the stress varies in the model.

7.2 Data for design of connections

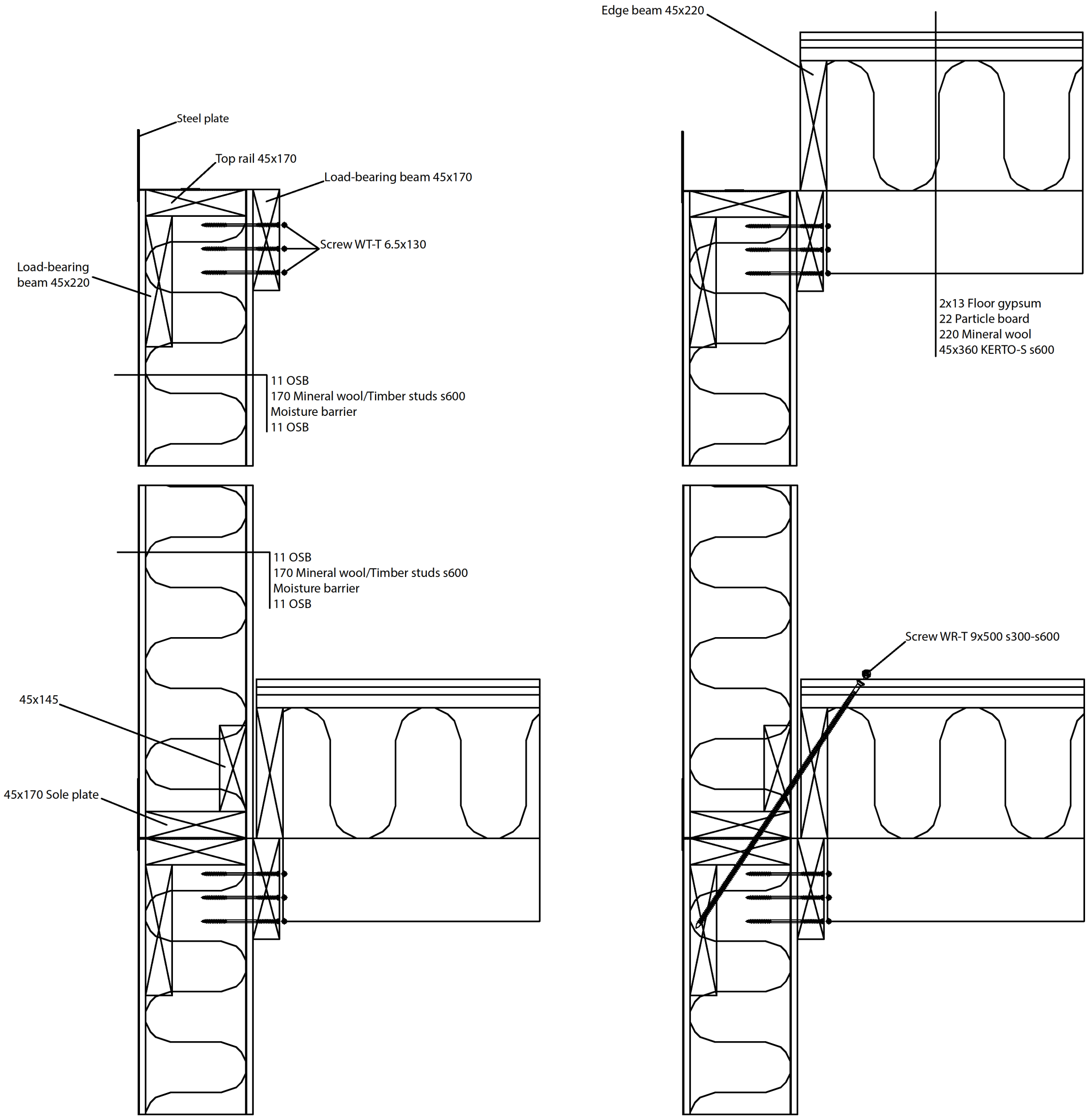
From the simulation of the improved floor, data is collected and used for designing the exterior horizontal tie in this case the plywood sheets. Data was also extracted for the design of the connection between the floor and exterior wall, since this is the connection that A-hus originally wanted to be further investigated. For the data see Appendix B.

8 Connection of floor elements to exterior wall

This chapter focuses on deepening the understanding of the existing solution for connection the floor element to the exterior wall. Structural capacity is evaluating and pros and cons of the connection is listed. To increase the knowledge and to get ideas for improvement there is going to be a short investigation about other types of solutions that exist today.

8.1 Existing solutions

In order to evaluate the connection between the floor element and the exterior wall it is necessary to understand the order of assembly of the building elements. First of all the exterior wall element are mounted. Secondly the floor element are lifted up and placed on the load-bearing beam with a distance of 8±4 mm from the edge of the OSB board. Thirdly the upper wall element is lowered on top of the bottom wall element. To connect the three elements a screw WR-T 9x500 is installed. Finally, the inner and outer cladding are applied to the elements, see Figure 8.1.



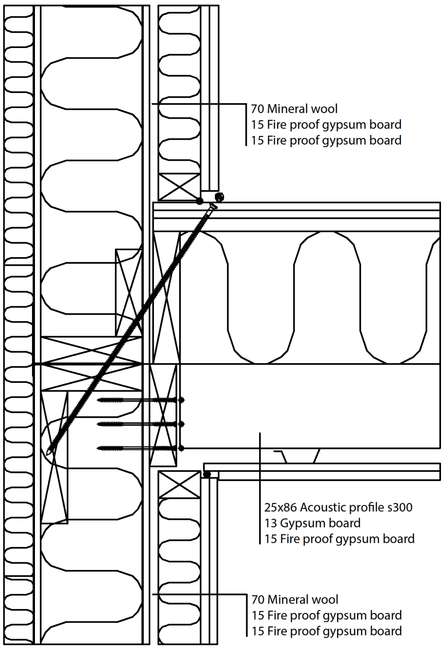


Figure 8.1 Assembly order for an exterior wall in a multi-story residential building from A-hus. With all material specifications and dimensions. Notice that there is a small gap between the wall and the floor in the figures and that gap is 8±4 mm due to the need of adjustment on the working site.

8.1.1 Structural analysis

Structural analysis is made based on Eurocode 5 section 8 and also based on the screw manufactures data sheet. When calculating the capacity of the connection some assumptions are made:

- The OSB board does not contribute to the structural capacities of the connection according to *Tragfähigkeit von Verbindungen mit stiftförmigen Verbindungsmitteln und Zwischenschichten,* (Blaß, H.J., Laskewitz, B. 2003). Another source states that minimum thickness for a structural member with non-predrilled holes is 45 mm for screws with a diameter of 9 mm, (Österreichisches Institut für Bautechnik 2012).
- The timber beam in the upper wall element has small edge distance, see Figure 8.2 part 4 and is therefore calculated with a reduced withdrawal and embedment capacity of 50%.[3]
- Characteristic yield moment in the screw is 19.2 Nm (Österreichisches Institut für Bautechnik 2012).
- Withdrawal capacity is calculated according to Eurocode 5 section 8 equation 8.40a since the screw is not produced according to EN 14592.[3]

The results are listed in Table 8.1. When designing a joint like this, it is important that failure modes a, b and c, see Figure 8.3 are avoided since these modes give a brittle failure. For all calculation steps, see Appendix C.

[3]Mail correspondence with Nils Horn designer at SFS intec 2016-02-25

Table 8.1 Result from hand-calculations of the connection. Johannsen's failure modes can be seen in Figure 8.3.

Connection between	Type of force	Failure mode	Capacity one screw [kN]
Wall-wall	Shear parallel to grain F_{inward}	Johansen f	4.6
Wall-wall	Shear perpendicular to grain $F_{\text{horizontal}}$	Johansen f	3.9
Floor-wall	Shear capacity parallel to grain F_{inward}	Johansen d	3.6
Floor-wall	Withdraw capacity $F_{\text{horizontal}}$	Failure in upper wall element	3.7

The smallest value of F_{inward} 3.6 kN and $F_{\text{horizontal}}$ 3.7 kN should be used when calculate the number of screws, see Figure 8.2 and Table 8.1.

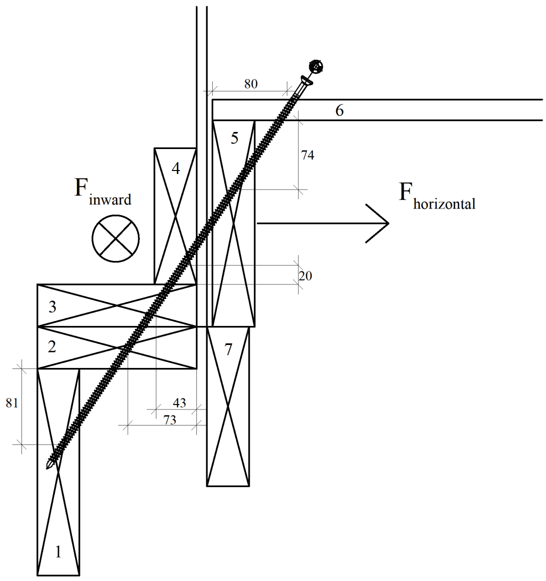


Figure 8.2 Forces that are calculated for the connection. The distance from edge to centre of the screw is also measured.

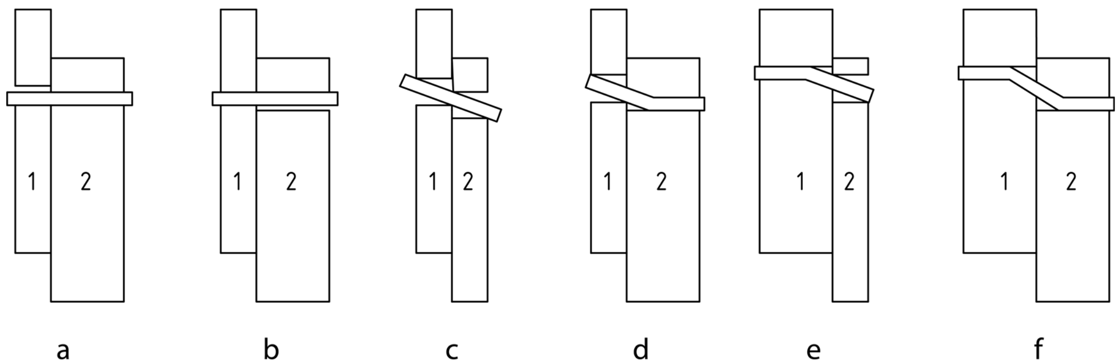


Figure 8.3 Failure modes for single shear members of timber parts and boards.

For loaded edge should edge distance be 4d according to Eurocode 5. In this case with a 9 mm screw the required edge distance is 36mm. The edge distance of timber part 4 is only 20 mm which makes this part uncertain when it comes to structural capacity, see Figure 8.2.

8.1.2 Pros and cons

From the calculation and interviews with the concerned parties; structural engineers and assembly teams, pros and cons are established when assessing the connection to get an overview of eventual areas of improvement.

Pros of the connection are:

- Few connectors needed, one connector is used both for connecting floor to wall and upper wall element to lower wall element.
- Standard timber parts and dimensions are used.

Cons of the connection:

- Need for a low gear drill-machine.[4]
- Short edge distance of the timber beam in the upper wall element, part 4 in Figure 8.2.
- Takes long time to drive the screw.[4]

8.2 Criteria for evaluation

Since A-hus is a timber house manufacturer there is a wish for using wood products as much as possible. Reduction in the number of fasteners that need to be applied for making the connection reach its structural capacity is also wanted, since this governs the effectiveness of assembly in their building system. It is preferable to make the building blocks as simple as possible to make the production process effective.[5]

Shrinkage of timber products used in the building blocks should be minimized since this might lead to unwanted consequences for the building. For example cracks in plastered facades which can lead to water damage. Another example could be cracks in interior layers, this might cause rupture in the waterproofing layer in moisture producing areas leading to moisture problems. A third example is cracks causing cosmetic problems to the interior walls. Shrinkage can also lead to unwanted inclination in buildings, resulting in floors that are unleveled and walls inclining. This inclination of the building has to be considered and foreseen in the structural analysis of the structure.[5]

From these requests key sentences are established for evaluation of other possible solutions to the connection. The key sentences are:

- Simplicity in design.
- Mostly wood products.
- Usage of standard timber dimension.
- Avoiding glulam and LVL as much as possible.

[4] Phone interview with Micael Neldemo CEO at Micael Neldemo Byggare 2016-02-09
[5] Meeting with Linus Abrahamsson designer at A-hus 2016-02-02

- Standard connectors (screws, bolts and nails).
- Small amount of connectors.
- Avoidance of timber in transverse direction.

8.3 Other solutions that exist

This part of the chapter aims to present how timber floors are connected to the exterior wall in both general ways as well as more specific solutions from different manufactures. There will also be an evaluation with regard to key sentences.

8.3.1 General solutions

In *System in Timber Engineering* written by Josef Kolb there is some figures of different ways to connect timber floor structures to the exterior wall. Figure 8.4 is presented below and is made with the book as a reference (Kolb, J. 2008).

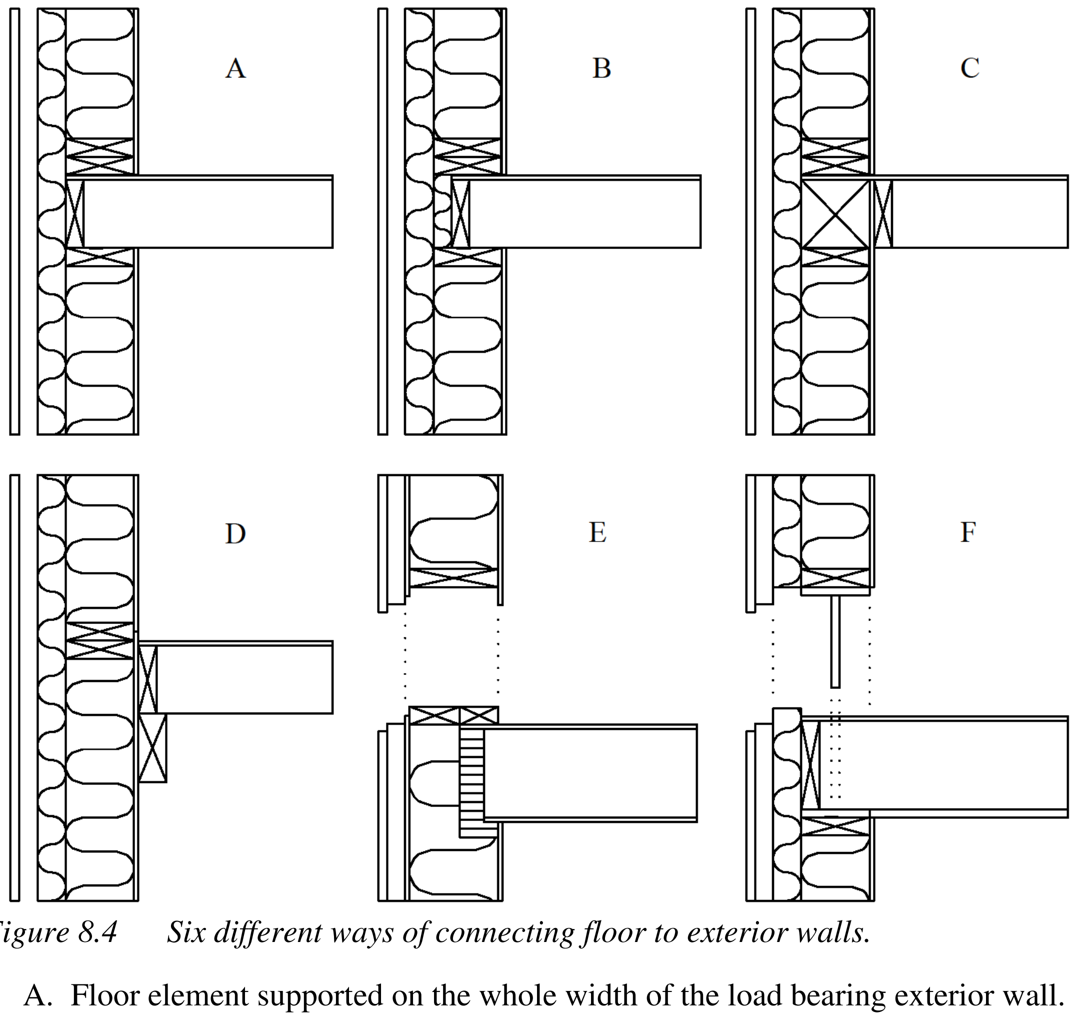


Figure 8.4 Six different ways of connecting floor to exterior walls.

A. Floor element supported on the whole width of the load bearing exterior wall.
B. Floor element partly supported on the load bearing exterior wall element.
C. The floor's upper panel stretches over the whole part of the load bearing exterior wall element, while the rest of the floor ends at the wall's inner panel.
D. The floor element is supported on an extra timber beam that is attached to the exterior wall.

E. The floor element is placed on a glulam beam that have been cut in a way so that the floor fits. The glulam beam is also integrated in the wall element.
F. Connection with steel components. The floor rests on the lower wall while the load from the structure above is transferred via a steel pin, resulting in that the structure is largely independent of deformations due to transverse floor joists (Kolb, J. 2008).

Base on key sentences A-C have too much timber in transvers direction and is not suitable for 4-8 story buildings. C have also used glulam which should be avoided. D have a solution that is basically the same as the existing solution. E have glulam and also needs to be cut in horizontal direction which is time consuming at the factory. F has steel components and is therefore not appropriate solution.

8.3.2 Specific solutions

There are many ways of connecting timber floors to the exterior wall, in this chapter some detailed ways are presented that could be an inspiration when improving A-hus connection.

For instance the company Martinsons has been building residential timber houses for a long time and they have the same type of problem as A-hus. But their solution is different in many ways, since a different building system is used. As structural members they use CLT in both walls and floors. In the floor system the CLT panel acts as a top flange while web and bottom flange is made out of glulam beams (Johansson, H. 2011). A screw WFD 10x180 is used to connect the floor to the exterior wall and it is driven from the CLT floor panel vertically into the center of the CLT wall element, see Figure 8.5.[6]

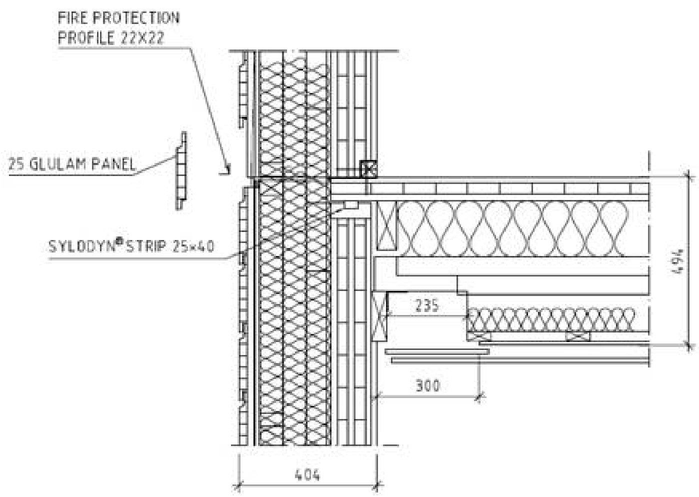


Figure 8.5 Figure showing how Martinsons connect their timber floors to the exterior wall.[6]

More inspiration comes from APA (American Plywood Association) – The Engineered Wood Associations homepage. APA is a nonprofit trade association

[6] Mail correspondence with Bas Boellaard, calculator and structural designer at Martinsons.

which focus on helping the industry to create structural wood products with exceptional strength, versatility and reliability, (APA 2016). Figure 8.6 is downloaded from this website and shows a connection with a hanger in steel and nails used for connecting the building members.

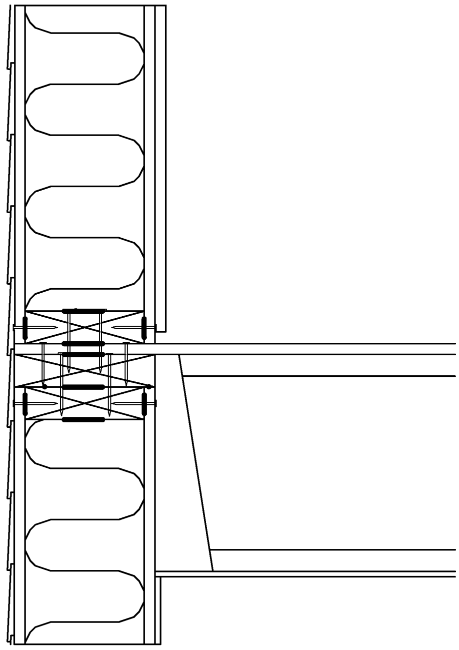


Figure 8.6 Connection where a top flange metal floor joist hanger is used to connect the floor to the exterior wall.

All connections in this chapter are neglected due to the key-sentences, but they still serve as inspiration for the design of the improved connection in the next chapter.

8.4 Improved connection

The improved connection is rather similar to the original connection, which is good from a manufacturing point of view. All of the improvements that are made aim to make the connection stronger by using more timber parts at appropriate places. No consideration is taken to economics, such as comparing what a screw costs compared to installing more timber parts into the connection. All improvements are listed below, Figure 8.7.

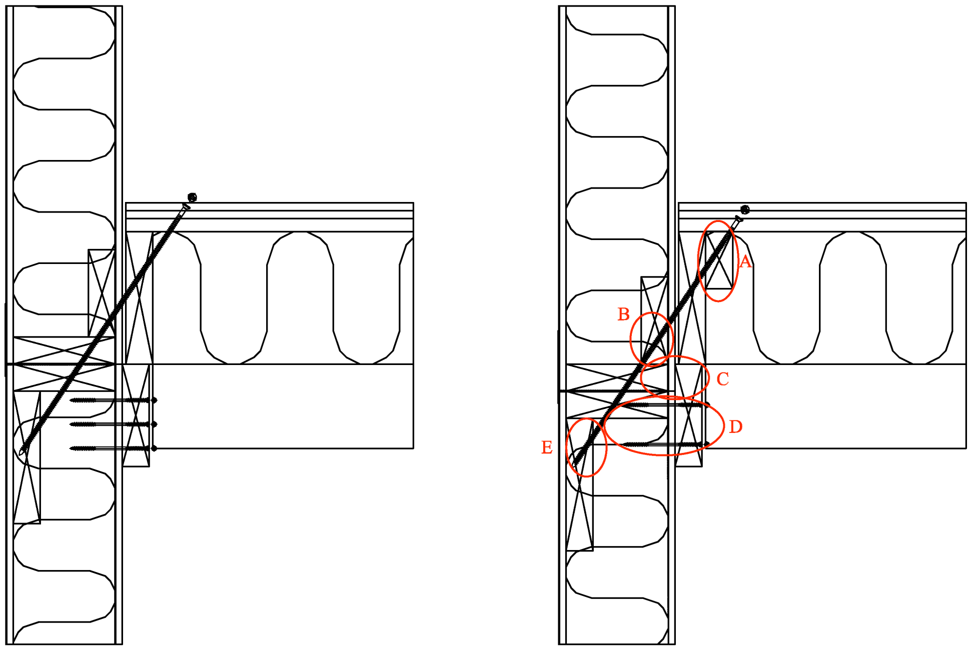


Figure 8.7 Comparison of old (to the left) and new connection (to the right) where improvements are highlighted.

A. Adding a noggin 45 x 95 mm between Kerto-S beams will increase the withdraw capacity of the screw in the floor element.
B. Edge distance is increased from 20 mm to 33 mm, which is almost the required loaded edge distance of 4d (36mm) according to Eurocode 5.
C. Since the load-bearing beam is moved up by 45 mm, the upper wall element is prevented from moving in the horizontal direction to the right. This will add extra strength to the wall-wall perpendicular shear connection in Table 9.2.
D. The screws that join the support beam to the wall are moved and are now attached to the top rail of the lower wall element. This is an improvement due to the risk of splitting the wall studs by the middle since it was previously installed close to the edge. This placement of support beam enables an increase in number of screws in the horizontal direction.
E. The risk of reduced withdrawal capacity due to missing this timber beam by buckling of the screw is less likely to occur. Since the distance between timber beam and top rail is shorter in the improved connection.

8.4.1 Structural analysis

Calculations are made basically with the same assumptions as for the original connection. But timber part B is now calculated with its full structural capacity, the whole calculations can be seen in Appendix D. The result is listed in the Table 8.2.

Table 8.2 Result from hand-calculations of the improved connection. Johannsen's failure modes can be seen in Figure 8.3.

Connection between	Type of force	Failure mode	Capacity one screw [kN]
Wall-wall	Shear parallel to grain F_{inward}	Johansen f	4.7
Wall-wall	Shear perpendicular to grain+ Withdraw of support beam $F_{\text{horizontal}}$	Johansen f+ Withdraw failure in screws	3.9+ The screws in the support beam
Floor-wall	Shear capacity parallel to grain F_{inward}	Johansen f	4.8
Floor-wall	Withdraw capacity $F_{\text{horizontal}}$	Failure in floor element	9.1

The smallest value of F_{inward} 4.7 kN and $F_{\text{horizontal}}$ 9.1 kN should be used when calculating the number of screws, see Figure 8.2 and Table 8.2. When comparing capacities for the old and the new connection it is obvious that there have been an improvement. F_{inward} has increased from 3.6 kN to 4.7 kN and $F_{\text{horizontal}}$ 3.7 kN to 9.1 kN there is a possibility to avoid the upper wall element to move by increasing the number of screws in the support beam.

Since F_{inward} is not designed in the FE-models, only $F_{\text{horizontal}}$ is relevant for the calculation on number of screws needed.

8.4.2 Number of screws

Now when the capacity of the screw is known it is possible to calculate how many screws needed in order to anchor the floor system to the exterior wall. The calculations can be found in Appendix B, and the required spacing of the screw is 400 mm at a distance of 1200 mm from the edge of the floor and between these edges distances the spacing needs to be 600 mm. The capacity and subsequent spacing are displayed in Figure 8.8.

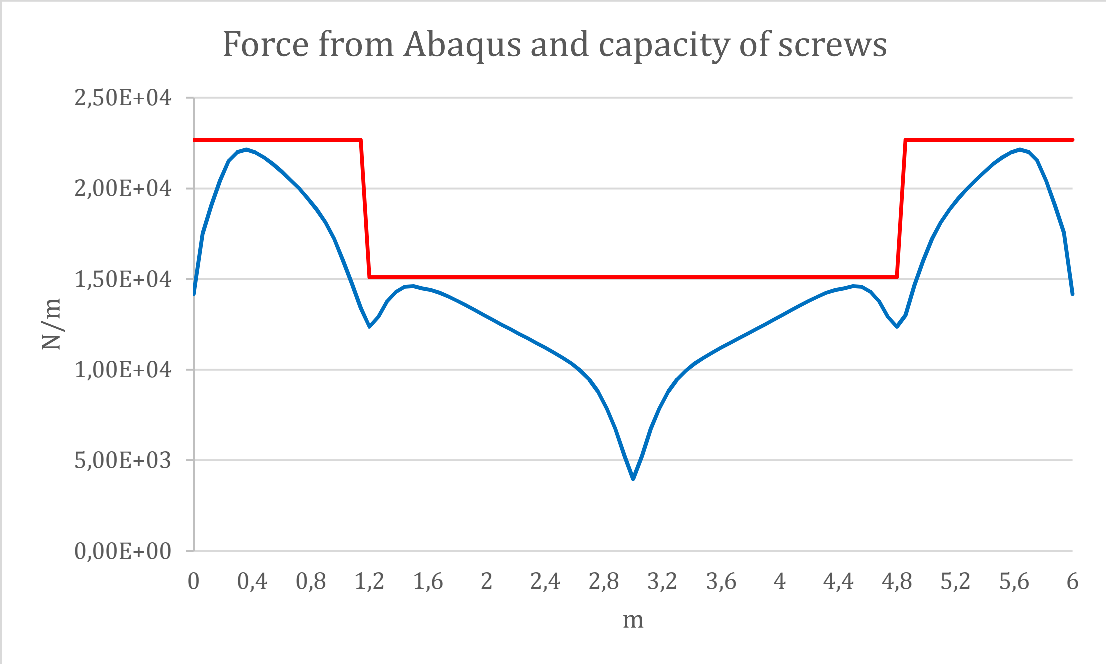


Figure 8.8 *Diagram over how the horizontal tension force varies along the short side of the floor, blue line. Values are collected from FE-models in Chapter 7. The graph is mirrored to represent the force distribution if the wall on the other side of the floor is lost in an accident. The red line is the capacity of the screws when placed at a spacing of 400 mm and 600 mm.*

9 Designing ties between floor elements

In Section 3.2 two alternative ways to ensure that a building has enough capacity to with stand a progressive collapse were presented. First method is to calculate forces in the horizontal ties based on equations and standard values. Second method is to use a simulation of the floor structure, to see what effects a removal of the load carrying wall would have.

To increase understanding of the consequence of choosing one of the two different methods, a comparison based on the five story apartment building described in Section 4.1.3 with the self-weight of the floor $g_{\mathrm{k}} = 1.0\ kN/m$ is outlined below.

9.1 Hand-calculations

In the first method Forces for designing horizontal ties in the floor structure is according to equation (1) that the horizontal internal ties $T_{\mathrm{l}} = 40\ \mathrm{kN/m}$ and equation (2) for horizontal exterior tie is $T_{\mathrm{p}} = 40\ \mathrm{kN/m}$. However these values are based on concrete structures[7] and could possibly be scaled down according equation (4). The load when the assumed value for concrete self-weight is 4.0 kN/m^2 (Martinsons 2006) is:

$$T_{\mathrm{p}} = T_{\mathrm{l}} = 40 * \frac{1.0+0.5*2.0}{4.0+0.5*2.0} = 16\ \mathrm{kN/m} \qquad (4)$$

This would be the design values to use for design of the horizontal ties if this method was chosen.

9.2 FE-modelling

From simulations of the floor system's behaviour when removing the main support it is obvious that range of forces in the ties vary substantially, see Appendix B. For example by using the two models in Chapter 7, a value for the force in the horizontal tie F_{tie} is calculated to 12.5 kN/m where plywood boards meet the exterior wall. In the centre of the span the force in the horizontal tie F_{tie} is 18.75 kN/m.

The variation in the force affecting the connections of the floor structure in the joint between floor and the exterior wall, were the variation of the force $F_{\mathrm{horisontal}}$ in this case governs the number of screws needed in the connection, see Section 8.4.2.

Summation of the forces from the horizontal tie F_{tie} and from the joint between floor and exterior wall $F_{\mathrm{horisontal}}$ that act in the joint where the horizontal tie meets the wall, see Figure 9.1 give the value of the total force acting in the region of the horizontal tie equal to, $F_{\mathrm{total}} = 12.5 + 18.73 = 33.7\ \mathrm{kN/m}$.

[7] Phone interview with Bo Westerberg 2016-03-15

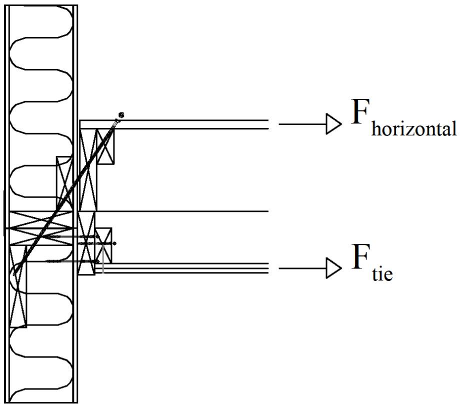


Figure 9.1 Forces acting in the region of the exterior horizontal tie.

9.3 Design of external horizontal tie

In the design of the exterior horizontal tie, results from the FE simulation are used. As mentioned in Section 7.2 there is a need to design the horizontal tie as a continuous unit. Therefore the connections between plywood layers and the connection to the exterior wall are designed, see Figure 9.2. The governing load for connecting floor elements together are diaphragm action and is not designed in this thesis, see joint D in Figure 9.2.

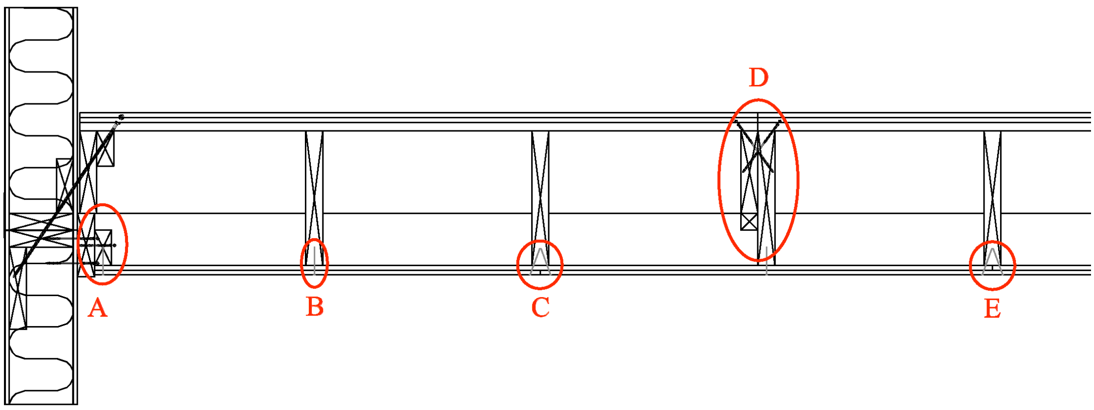


Figure 9.2 Connections to be further investigated are highlighted.

In the design of the horizontal tie a tension force of 22.5 kN is used, see Appendix B. The capacities of screws and nails are calculated in Appendix E. These two give following design of connectors in the horizontal tie:

A. Connections between plywood and timber noggin 45 x 95 mm are made with round nails 2.8 x 75 mm, placed at a spacing of 30 mm. The timber noggins are held in place by screwing 10 WT-T 6.5x90 screws into the support beam.
B. To attach plywood boards to Kerto-S beams the same nails are used as in A, but different spacing. The required spacing is now 80 mm. This connection is also used to transfer tension force between the upper and lower plywood board.

C. Joints between the lower plywood boards are connected with nails used in A, nailed at an angle to increase edge distance. Spacing needed is 80mm.
D. The connection of floor elements are made with screws WT-T 6.5x160 driven at an angle (36^0) from both sides.
E. Same as C. It is important that the upper and lower boards are not joined on the same Kerto-S beam.

10 Discussion

In this chapter a summary discussion of the findings in the literature study and results from analysis are discussed by the authors.

10.1 Literature study

The findings in the literature study clearly indicate the importance of continuity between building members. So that in the event of an accident in the primary zone, severe damage to the secondary zone could be mitigated and personal injuries could be avoided. Even though almost the whole literature study is based on one book that treats progressive collapse in concrete structures, all the different phenomenon are the same for timber structures.

Eurocode 1 provides two different ways of designing a building so that in the event of an accident, the damage to the secondary zone is limited and personal injuries thereby minimized. First is to apply a force in the direction of the interior- and exterior horizontal ties. The suggested force is rather high since it is based on concrete structures that has a higher self-weight. A possible reasonable approach is to scale down the design forces in the ties with a factor that is based on a comparison of self-weight between the different building materials, as in Section 3.3 and 9.1.

Secondly Eurocode 1 states that to design a floor structure with enough robustness, a wall section between two load-bearing walls should be removed from the load model. The floor structure should have an alternative load path, in this case via interior and exterior horizontal ties in floor structure. This scenario can lead to an unreasonable large part of the wall being removed, causing large forces in the ties. No considerations are taken to the weaker zones in the wall, such as larger windows and doors and joints between wall elements. If these were taken into account a smaller span might be developed and that would lead to a smaller force in the ties.

But from the mobile crane accident in Stavanger, it is clear that a large part of the building can be damaged and that the damage zones stretch above the second floor. This also stated in Section 3.2: "The recommended size of a local rupture is the smallest of 15 % of the floor area or 100 m^2 in each two adjacent floors, see Figure 3.1." This indicates that a large part of the building can be damaged and this possibility needs to be considered when designing buildings.

10.2 Case study

In the investigation of A-hus' building system it was questionable if the floor structure had enough structural properties so that it could be regarded as a continuous unit, once it is installed in the building. To get a better understanding of how the floor structure behaves if an accident were to occur the second alternative in Eurocode 1 is chosen for the design process. Even though one could argue that the floor structure could still hang in the wall above the assumed lost wall, since the screw penetrates both these walls, see Figure 8.1. It is the authors' opinion, based on the discussion in Section 10.1, that it is reasonable to assume that some damage is inflicted to the upper wall and that it loses its possibility to carry the floor.

When modelling the floor structure in Abaqus some simplifications had to be made due to the complex nature of modelling connections in a satisfying way without evaluating the stiffness in each individual joint. Further on it is obvious that the weak zones in the floor structure will govern the failure. In the model the particle board is continuous, but in reality there are joints that will influence in an unpredictable way. Also all the joints in the model are modelled so they would not fail. This implies that the modelled floor is stronger than in reality and the failure in analysis will lead to failure in reality. Further on, walls supporting the floor are assumed to be prevented from moving, but some deformation in these parts are likely to happen. To minimize these deformations it is vital that the wall, above the supposed lost wall, is designed to act as a high beam and does not fall down onto the floor below. Based on the authors' assumptions the floor will fail due to high stresses in the particle board, see Figure 5.3. Therefor improvements need to be made to the floor structure.

The main thing to improve within the existing floor system is to strengthen the horizontal exterior tie. This should be done 1200 mm from edge according to Eurocode 1, see Section 3.2 and this measurement corresponds neatly, with the dimensions of standard plywood boards.

To make the analysis of the improved floor system, two simplified models were made due to the complexity of the structure. The first model includes all structural components of the floor system and since the joints are treated as fixed they attract a lot of stresses. This will lead to an overestimation of load carrying capacity and it will be difficult to design these connections for the needed withdrawal capacity in end grain of the edge beams and noggins.

Two alternatives were discussed after this conclusion: One was to put in "springs" in all of these connections with the capacities of the screws used in the connection, recalculated into a stiffness for the "springs". Secondly, to make a model where these parts were removed, to underestimate the load bearing capacity of the floor system. The most reasonable according to the authors is to choose the second alternative and, merging the data from these two models, to get an approximation that should be near the reality.

From the result of the second model it is clear that there will be large tensile stress in the plywood boards, but well below the limit of the capacity for the plywood boards. However, in the model the plywood board are seen as a continuous unit and it can be hard to achieve this due to the tensile stress produced in this model. But since this model is an underestimation, the data from the model that overestimates the floor could be used and an average stress could be calculated. This is used in the design of whole horizontal tie.

The original aim of this thesis was to investigate and evaluate the connection of the floor system to exterior walls and to investigate what improvements could be made. However this task seams in the retrospect to have been taken a little too much of the authors' time resources, time that could have been spent on deeper analysis elsewhere in the thesis. After the analysis some small changes were suggested by the authors to improve the structural capacity, which led to a decreased number of screws required. This could lead to time and money savings in a building project.

There are two ways to get the design force for the horizontal tie, one is by hand-calculations and the other by doing FE-models. The hand-calculations are based on concrete structures and seams to overestimate the forces for light-weight buildings. When comparing the result from the numerical models and the scaled down hand-calculations the resulting forces vary between 12.5-18.75 kN/m and 16 kN/m. The coherence of these two results implies that it works to scale down the hand-calculations and that the values from FE-models are reasonable. By using these values to design the horizontal tie, the floor structure should be able to prevent a progressive collapse.

11 Conclusion

To prevent progressive collapse in the event of an accident, it is vital to have continuity in all of the building members. It is the authors' hope that this thesis will lead to safer timber buildings and to show that the actions needed to prevent a progressive collapse can be done with minor extra cost and effort.

When designing floor structures it is important to have continuous external horizontal ties. If the calculations are made based on hand-calculations or FE analysis it will likely not make a difference in the design of the horizontal tie. It is more important that the designer looks at this problem and realizes the importance of it. For example when looking at Eurocode 1's recommendation, a horizontal tie should be placed in a zone 1200 mm in from the edge of the floor. This is an example of best practise for engineers.

When looking at A-hus' floor system it is clear that the designer has not taken this into account. Thereby the authors present a solution that consist of two layers of plywood boards 12 mm mounted on the underside of the floor see Figure 11.1.

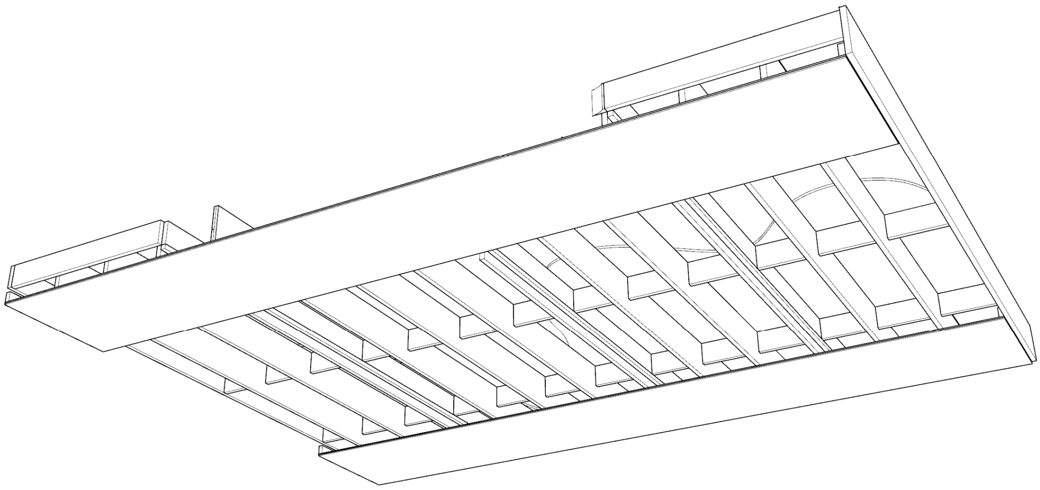


Figure 11.1 Two double layered plywood boards ensuring the load-carrying ability in the event of an accident to the main support. Where some of the particle boards and other parts are removed to make a better view of the floors components.

Further on improvements are made to the connection of the floor to the exterior wall elements by increasing the edge distances and the amount of wood that is penetrated by the screw, see Section 8.4. It is the authors' opinion that the screw is the best solution for joining the three elements together.

11.1Further studies

To ensure that A-hus' building system is capable of preventing a progressive collapse in the event of an accident, further studies on the wall construction are preferred. A model of the wall as a high-beam stretching over the lost wall in this master thesis should be enough. Important when investigating wall structures is the natural weakness in them, for example joints between wall elements, openings in the structure such as doors and large windows. These inhomogeneous in the wall could create weaknesses that might govern the ability to act as a continuous high beam.

Even though not outlined in the Eurocode 1 recommendations for design of prevention of a progressive collapse, a model of the building system with a lost corner as in the accident in Stavanger would be of interest since this might cause tilting of the structure which might affect its total stability.

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13 Appendix

Contents

A Hand-calculation of deflection

	Part	h [m]	b [m]	A [m^2]	E_{mean} [Pa]
Particle board	1	2,20E-02	6,00E-01	1,32E-02	1,80E+09
Kerto-S	2	3,60E-01	4,50E-02	1,62E-02	1,38E+10
		3,82E-01			

EA	z [m]	EAz	EA(z_{na}-z)	EA$(z_{na}-z)^2$	$Ebh^3/12$
2,38E+07	1,10E-02	2,61E+05	4,10E+06	7,08E+05	9,58E+02
2,24E+08	2,02E-01	4,52E+07	4,10E+06	7,53E+04	2,41E+06
2,47E+08		4,54E+07			

z_{na}= 1,84E-01

∑EI
7,09E+05
2,49E+06
3,20E+06

Material constants

$G_{mean.Kerto-S}$= 6,00E+08 Pa

Distributed load over one section

q= 1,200E+03 N/m

Span

l= 6 m

Deflection bend	6,33E-03	(5*q*l^4)/(384*∑EI)
Deflection shear	6,67E-04	1,2*(q*l^2)/(8*$G_{mean.Kerto}$*$A_{Kerto)}$
Total deflection	7,00E-03	

B Data from Abaqus

The data that is used in the thesis are collected by creating paths at places that is of interest. These places are:

- Connection of noggin and edge beam
- Tension stress at the short side of the floor
- Tension stress in the plywood

Connection of noggin and edge beam

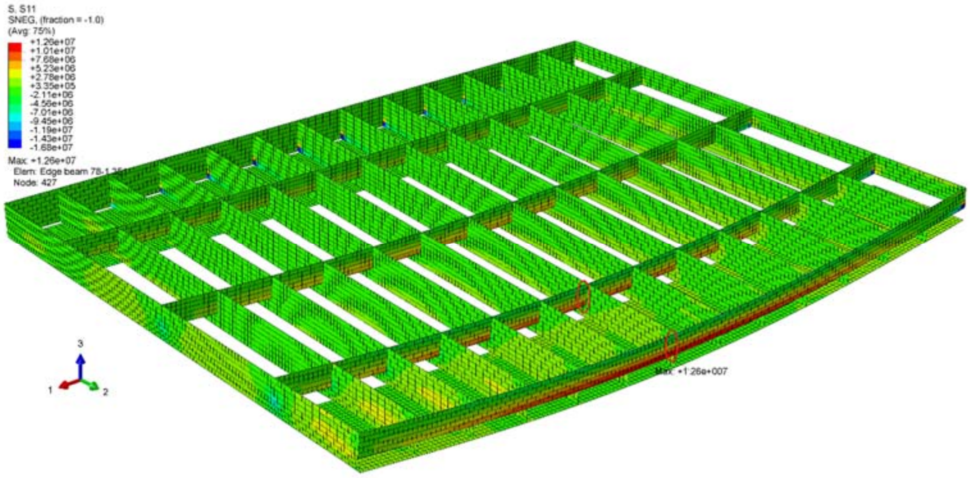


Figure B.1 FEM-model of the improve floor with noggin where the place of data are highlighted in red.

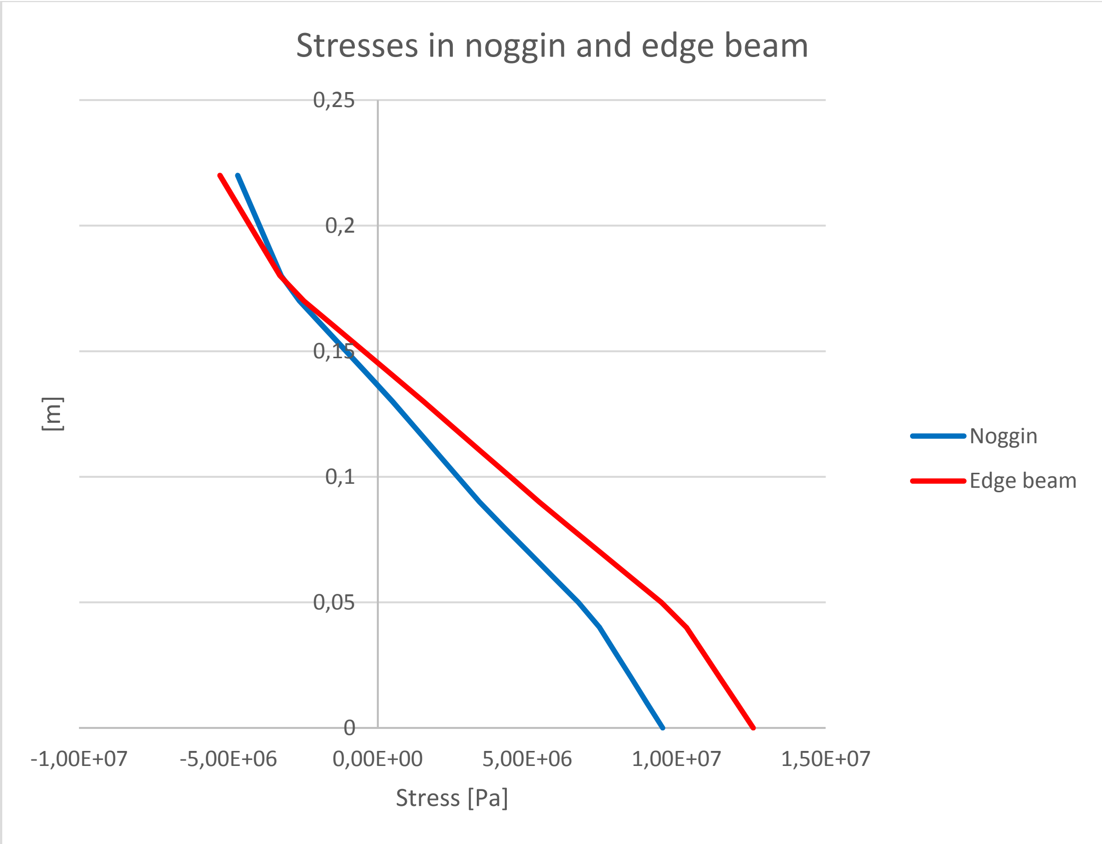


Figure B.2 Diagram over how the stress varies in edge beam and noggin.

Force in noggin and edge beam are calculated by taking the area below the graphs and the result are 45 kN and 31 kN tension force and 10 kN and 10 kN compressive force for edge beam and noggin.

Tension stress at the short side of the floor

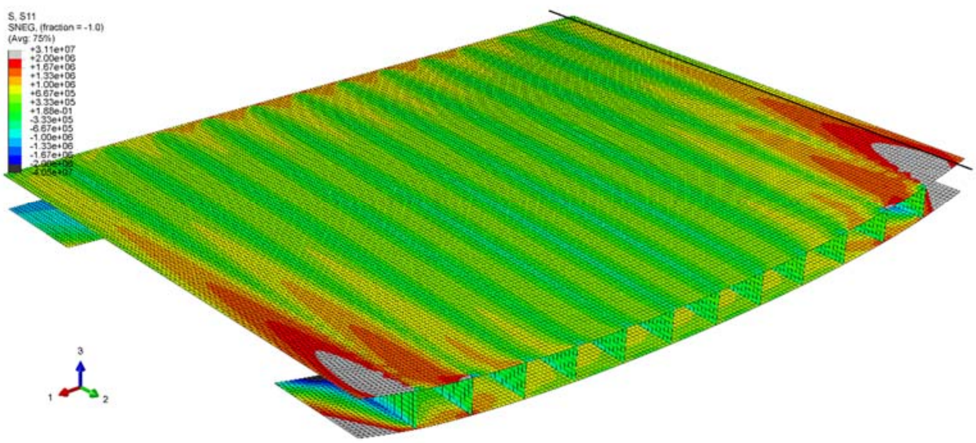


Figure B.3 Illustration of where data are collected from in the Abaqus models.

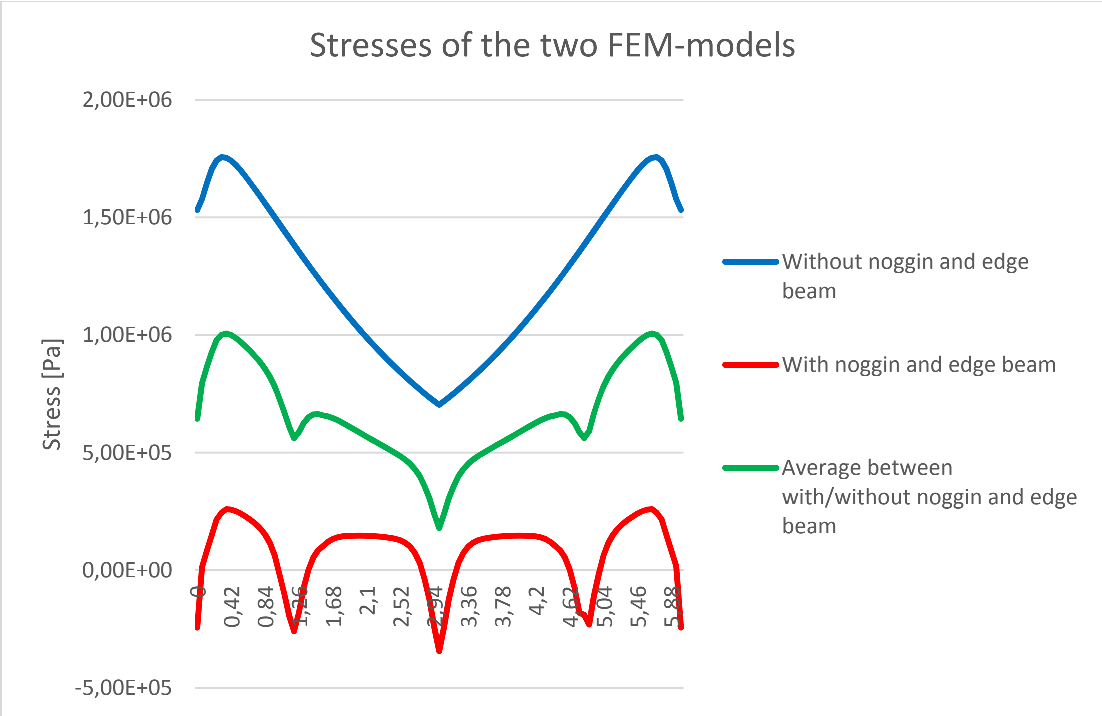


Figure B.4 Graph showing the difference between the two models and the average between them. The graphs are mirrored to symbolize if the wall on the other side is taken away.

This stress is then calculated into a force per meter by multiply it by the thickness of the particle board (22 mm).

Tension force in the plywood

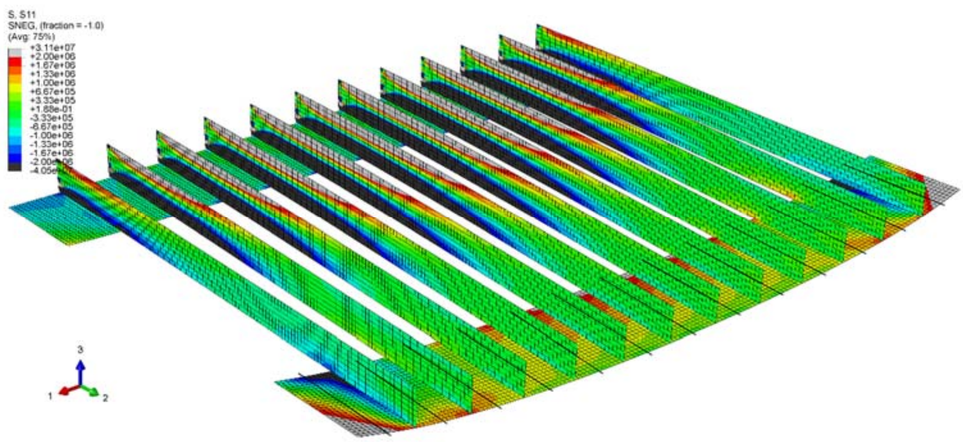


Figure B.5 Illustration where paths where drawn when collecting data.

The average stress along the paths are calculated and multiplied by thickness and width (24 mm and 1200 mm). The result is shown in Figure B.6. There were some differences between the stress 0 m - 3.9 m and 3.9 m – 7.8 m but the higher of those two were taken to make it easier to design.

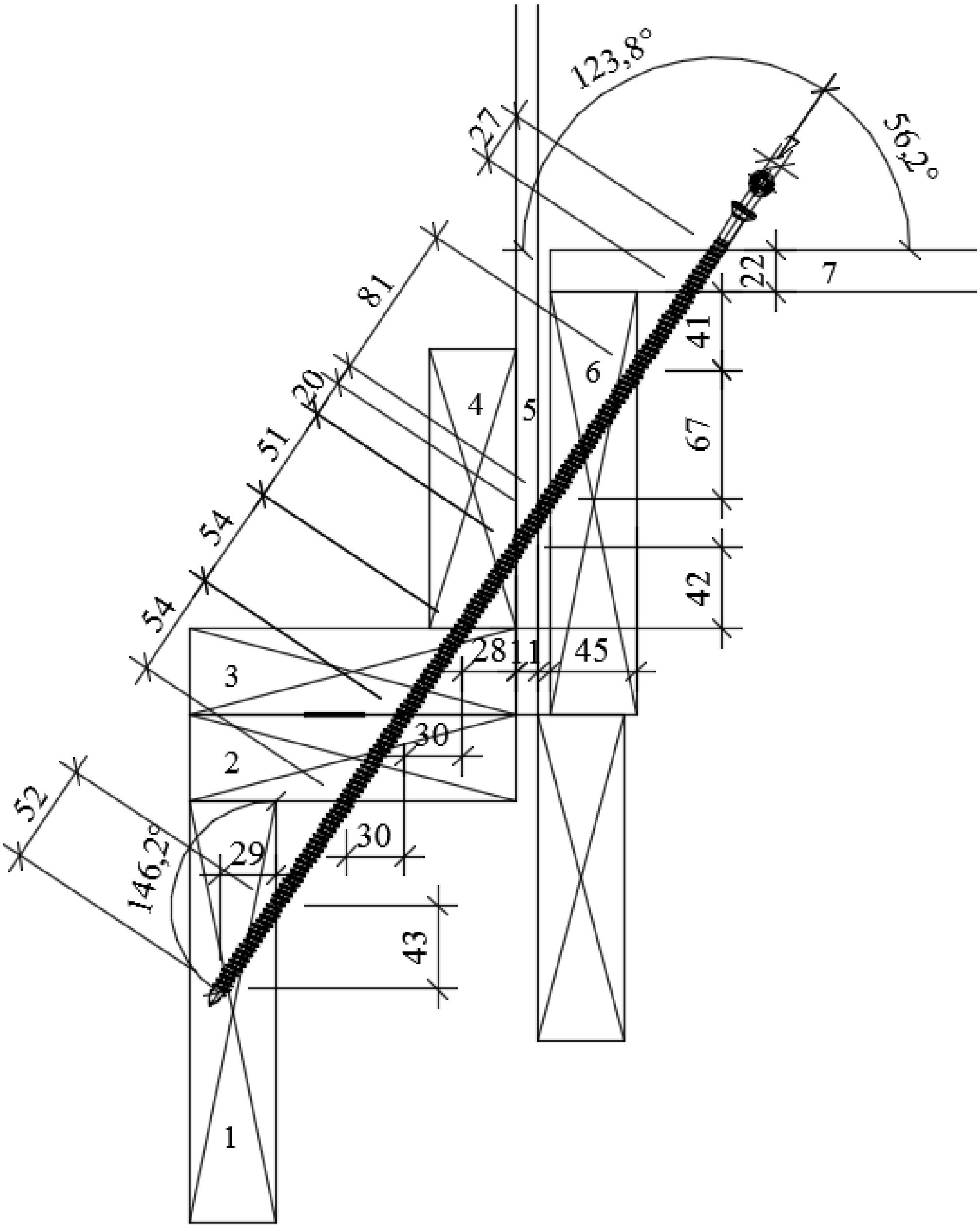
Figure B.6 Graph showing the difference between the two models and the average between them.

The maximum value of the average graph is 22.5 kN and that value is used for the design nails and screws that connects the plywood boards to the rest of the structure. The tension force for the average graph near the edge is 15.0 kN.

Tension force horizontal tie

By taking the average of the first values in the average graph form 0 – 1200 mm in Figure B.4 multiplied by the thickness of the particle board a value of 18.73 kN/m is calculated. Then divide the value 15.0 kN by 1200 mm and get 18.75 kN/m. These two are then added together to 33.7 kN/m. This value is the total tension force at the edge of the floor.

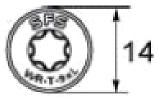
C Structural capacity of original connection

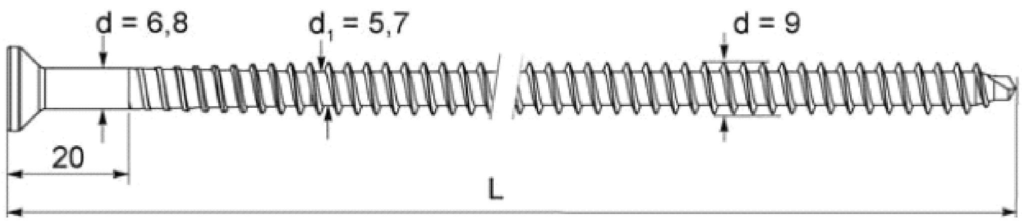


Material properties screw:

WR-T-9,0 x L

Material: special carbon steel





$d_{screw} := 5.7mm$	Threaded root diameter
$d_{thread} := 9mm$	Threaded diameter
$l_{screw} := 500mm$	Length of screw
$d_{head.screw} := 14mm$	Diameter of the head
$M_{y.Rk} := 19.2N \cdot m$	Value from manufacture according to ETA-12/0062 table A2.1. Since the screw is <u>not</u> produces according to EN14592 fastener values from EC5 does not work.
$f_{tens.k} := 35.9kN$	Characteristic tensile strength from manufacture.

<u>Material properties timber and particle board</u>

$t_1 := 52mm$	Penetrations depth the timber stud, see Figure above.
$t_2 := 54mm$	
$t_3 := 54mm$	
$t_4 := 50\% \cdot 51mm$	Assuming that 50% of the screws are splitting timber beam 4 because of short edge distances.
$t_5 := 20mm$	Do not calculate OSB as a structural member.
$t_6 := 81mm$	
$t_7 := 27mm$	
$t_{particle.board} := 22mm$	Thickness of particle board

$$\rho_{timber} := 350 \frac{kg}{m^3}$$

Characteristic density of timber C24

$$\rho_{paricle.board} := 550 \frac{kg}{m^3}$$

Characteristic density of particle board 24mm P5

$$f_{h.timber.k} := 0.082 \cdot \left(1 - 0.01 \cdot \frac{d_{thread}}{mm}\right) \cdot \frac{\rho_{timber}}{\frac{kg}{m^3}} \cdot \frac{N}{mm^2} = 26.117 \cdot MPa$$

EC5 Eq 8.32 embedment strength parallel to grain

$$k_{90} := 1.35 + 0.015 \cdot \frac{d_{thread}}{mm} = 1.485$$

Soft wood Eq.8.33

$$f_{h.90.k} := \frac{f_{h.timber.k}}{k_{90} \cdot \sin(90deg)^2 + \cos(90deg)^2} = 17.587 \cdot MPa$$

Eq.8.31 embedment strength perpendicular to grain.

$$f_{h.particle.board.k} := 50 \cdot \left(\frac{d_{thread}}{mm}\right)^{-0.6} \cdot \left(\frac{t_{particle.board}}{mm}\right)^{0.2} \cdot \frac{N}{mm^2} = 24.826 \cdot MPa$$

EC5 Eq 8.37

Withdrawal strength

$$F_{ax.1.Rk} := 1 \cdot 13.5 \left(\frac{\rho_{timber}}{350 \frac{kg}{m^3}}\right)^{0.8} \cdot \frac{t_1}{mm} \frac{d_{thread}}{mm} \cdot N = 6.318 \times 10^3 N$$

Eq 8.40a

$$F_{ax.2.Rk} := 1 \cdot 13.5 \left(\frac{\rho_{timber}}{350 \frac{kg}{m^3}}\right)^{0.8} \cdot \frac{t_2}{mm} \frac{d_{thread}}{mm} \cdot N = 6.561 \times 10^3 N$$

$$F_{ax.3.Rk} := 1 \cdot 13.5 \left(\frac{\rho_{timber}}{350 \frac{kg}{m^3}}\right)^{0.8} \cdot \frac{t_3}{mm} \frac{d_{thread}}{mm} \cdot N = 6.561 \times 10^3 N$$

$$F_{ax.4.Rk} := 1 \cdot 13.5 \left(\frac{\rho_{timber}}{350 \frac{kg}{m^3}}\right)^{0.8} \cdot \frac{t_4}{mm} \frac{d_{thread}}{mm} \cdot N = 3.098 \times 10^3 N$$

$$F_{ax.5.Rk} := 0 = 0$$

No withdraw capacity in OSB

$$F_{ax.6.Rk} := 1 \cdot 13.5 \left(\frac{\rho_{timber}}{350 \frac{kg}{m^3}} \right)^{0.8} \cdot \frac{t_6}{mm} \cdot \frac{d_{thread}}{mm} \cdot N = 9.841 \times 10^3 N$$

$$F_{ax.7.Rk} := 1 \cdot 13.5 \left(\frac{\rho_{paricle.board}}{350 \frac{kg}{m^3}} \right)^{0.8} \cdot \frac{t_7}{mm} \cdot \frac{d_{thread}}{mm} \cdot N = 4.71 \times 10^3 N$$

Connection between wall elements

Shear capacity parallel to grain

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

EC5 Eq 8.6 Timber to timber connections for fasteners in single shear

$$F_{v.Rk.a} := f_{h.timber.k} \cdot t_2 \cdot d_{thread} = 1.269 \times 10^4 N$$

$$F_{v.Rk.b} := f_{h.timber.k} \cdot t_3 \cdot d_{thread} = 1.269 \times 10^4 N$$

$$Rope_{effect.1.2} := \frac{F_{ax.1.Rk} + F_{ax.2.Rk}}{4} = 3.22 \times 10^3 N$$ Rope effect from part 1 and 2

$$Johansen_c := \frac{f_{h.timber.k} \cdot t_2 \cdot d_{thread}}{1 + \beta} \cdot \left[\sqrt{\beta + 2 \cdot \beta^2 \cdot \left[1 + \frac{t_3}{t_2} + \left(\frac{t_3}{t_2} \right)^2 \right] + \beta^3 \cdot \left(\frac{t_3}{t_2} \right)^2} - \beta \cdot \left(1 + \frac{t_3}{t_2} \right) \right]$$

$$Johansen_c = 5.258 \times 10^3 N$$

$$F_{v.Rk.c} := \begin{array}{|l} 2 Johansen_c \ \text{ if } \ Rope_{effect.1.2} > Johansen_c \\ Johansen_c + Rope_{effect.1.2} \ \text{ otherwise} \end{array} = 8.477 \times 10^3 N$$

The rope effect should be limited according to 8.2.2(2) for screws to max 100% of the Johansen yield part.

$$Johansen_d := 1.05 \cdot \frac{f_{h.timber.k} \cdot t_2 \cdot d_{thread}}{2 + \beta} \cdot \left[\sqrt{2 \cdot \beta \cdot (1 + \beta) + \frac{4 \cdot \beta \cdot (2 + \beta) \cdot M_{y.Rk}}{f_{h.timber.k} \cdot d_{thread} \cdot t_2^2}} - \beta \right]$$

$$F_{v.Rk.d} := \begin{array}{|l} 2 Johansen_d \ \text{ if } \ Rope_{effect.1.2} > Johansen_d \\ Johansen_d + Rope_{effect.1.2} \ \text{ otherwise} \end{array} = 8.028 \times 10^3 N$$

$$Johansen_e := 1.05 \cdot \frac{f_{h.timber.k} \cdot t_3 \cdot d_{thread}}{2 + \beta} \cdot \left[\sqrt{2 \cdot \beta^2 \cdot (1 + \beta) + \frac{4 \cdot \beta \cdot (2 + \beta) \cdot M_{y.Rk}}{f_{h.timber.k} \cdot d_{thread} \cdot t_3^{\,2}}} - \beta \right]$$

$$F_{v.Rk.e} := \begin{vmatrix} 2Johansen_e \ \text{ if } \ Rope_{effect.1.2} > Johansen_e \\ Johansen_e + Rope_{effect.1.2} \ \text{ otherwise} \end{vmatrix} = 8.028 \times 10^3\,N$$

$$Johansen_f := 1.15 \cdot \sqrt{\frac{2 \cdot \beta}{1 + \beta}} \cdot \sqrt{2M_{y.Rk} \cdot f_{h.timber.k} \cdot d_{thread}} = 3.455 \times 10^3\,N$$

$$F_{v.Rk.f} := \begin{vmatrix} 2Johansen_f \ \text{ if } \ Rope_{effect.1.2} > Johansen_f \\ Johansen_f + Rope_{effect.1.2} \ \text{ otherwise} \end{vmatrix} = 6.675 \times 10^3\,N$$

$$F_{v.Rk} := \min\left(F_{v.Rk.a}, F_{v.Rk.b}, F_{v.Rk.c}, F_{v.Rk.d}, F_{v.Rk.e}, F_{v.Rk.f}\right) = 6.675 \cdot kN$$

$k_{mod.timber} := 0.9$ SC 2 Short term action

$\gamma_{M.timber} := 1.3$

$$F_{v.Rd.wall.wall.0} := \frac{k_{mod.timber} \cdot F_{v.Rk}}{\gamma_{M.timber}} = 4.621 \cdot kN$$

Shear capacity perpendicular to grain

$$\beta := \frac{f_{h.90.k}}{f_{h.90.k}} = 1$$

EC5 Eq 8.6 Timber to timber connections for fasteners in single shear

$$F_{v.Rk.a.90.wall} := f_{h.90.k} \cdot t_2 \cdot d_{thread} = 8.547 \times 10^3\,N$$

$$F_{v.Rk.b.90.wall} := f_{h.90.k} \cdot t_3 \cdot d_{thread} = 8.547 \times 10^3\,N$$

$$Rope_{effect.1.2} = 3.22 \times 10^3\,N$$

$$Johansen_{c.90.wall} := \frac{f_{h.90.k} \cdot t_2 \cdot d_{thread}}{1 + \beta} \cdot \left[\sqrt{\beta + 2 \cdot \beta^2 \cdot \left[1 + \frac{t_3}{t_2} + \left(\frac{t_3}{t_2}\right)^2 \right] + \beta^3 \cdot \left(\frac{t_3}{t_2}\right)^2} - \beta \cdot \left(1 + \frac{t_3}{t_2}\right) \right]$$

$$F_{v.Rk.c.90.wall} := \begin{cases} 2Johansen_{c.90.wall} & \text{if } Rope_{effect.1.2} > Johansen_{c.90.wall} \\ Johansen_{c.90.wall} + Rope_{effect.1.2} & \text{otherwise} \end{cases} = 6.76 \times 10^3 \, N$$

$$F_{v.Rk.c.90.wall} = 6.76 \times 10^3 \, N$$

$$Johansen_{d.90.wall} := 1.05 \cdot \frac{f_{h.90.k} \cdot t_2 \cdot d_{thread}}{2 + \beta} \cdot \left[\sqrt{2 \cdot \beta \cdot (1 + \beta) + \frac{4 \cdot \beta \cdot (2 + \beta) \cdot M_{y.Rk}}{f_{h.90.k} \cdot d_{thread} \cdot t_2^2}} - \beta \right]$$

$$F_{v.Rk.d.90.wall} := \begin{cases} 2Johansen_{d.90.wall} & \text{if } Rope_{effect.1.2} > Johansen_{d.90.wall} \\ Johansen_{d.90.wall} + Rope_{effect.1.2} & \text{otherwise} \end{cases} = 6.574 \times 10^3 \, N$$

$$Johansen_{e.90.wall} := 1.05 \cdot \frac{f_{h.90.k} \cdot t_3 \cdot d_{thread}}{2 + \beta} \cdot \left[\sqrt{2 \cdot \beta^2 \cdot (1 + \beta) + \frac{4 \cdot \beta \cdot (2 + \beta) \cdot M_{y.Rk}}{f_{h.90.k} \cdot d_{thread} \cdot t_3^2}} - \beta \right]$$

$$F_{v.Rk.e.90.wall} := \begin{cases} 2Johansen_{e.90.wall} & \text{if } Rope_{effect.1.2} > Johansen_{e.90.wall} \\ Johansen_{e.90.wall} + Rope_{effect.1.2} & \text{otherwise} \end{cases} = 6.574 \times 10^3 \, N$$

$$Johansen_{f.90.wall} := 1.15 \cdot \sqrt{\frac{2 \cdot \beta}{1 + \beta}} \cdot \sqrt{2M_{y.Rk} \cdot f_{h.90.k} \cdot d_{thread}} = 2.835 \times 10^3 \, N$$

$$F_{v.Rk.f.90.wall} := \begin{cases} 2Johansen_{f.90.wall} & \text{if } Rope_{effect.1.2} > Johansen_{f.90.wall} \\ Johansen_{f.90.wall} + Rope_{effect.1.2} & \text{otherwise} \end{cases} = 5.67 \times 10^3 \, N$$

$$F_{v.Rk.90.wall.1} := \min\left(F_{v.Rk.a.90.wall}, F_{v.Rk.b.90.wall}, F_{v.Rk.c.90.wall}\right) = 6.76 \cdot kN$$

$$F_{v.Rk.90.wall.2} := \min\left(F_{v.Rk.d.90.wall}, F_{v.Rk.e.90.wall}, F_{v.Rk.f.90.wall}\right) = 5.67 \times 10^3 \, N$$

$$F_{v.Rk.90.wall} := \min\left(F_{v.Rk.90.wall.1}, F_{v.Rk.90.wall.2}\right) = 5.67 \times 10^3 \, N$$

$$F_{v.Rd.90.wall} := \frac{k_{mod.timber} \cdot F_{v.Rk.90.wall}}{\gamma_{M.timber}} = 3.926 \cdot kN$$

Connection between floor and wall elements

Shear capacity parallel to grain

$$\beta_{5.6} := \frac{f_{h.timber.k}}{f_{h.timber.k}} = 1$$

EC5 Eq 8.6 Timber to timber connections for fasteners in single shear

$$F_{v.Rk.a} := f_{h.timber.k} \cdot t_4 \cdot d_{thread} = 5.994 \times 10^3\,N$$

$$F_{v.Rk.b} := f_{h.timber.k} \cdot t_6 \cdot d_{thread} = 1.904 \times 10^4\,N$$

$$Rope_{effect.6.7} := \frac{F_{ax.6.Rk} + F_{ax.7.Rk}}{4} = 3.638 \times 10^3\,N$$

$$Rope_{effect.3.4} := \frac{F_{ax.3.Rk} + F_{ax.4.Rk}}{4} = 2.415 \cdot kN$$

$$Rope_{effect} := \min\left(Rope_{effect.6.7}, Rope_{effect.3.4}\right) = 2.415 \times 10^3\,N$$

$$Johansen_c := \frac{f_{h.timber.k} \cdot t_4 \cdot d_{thread}}{1 + \beta_{5.6}} \cdot \left[\sqrt{\beta_{5.6} + 2 \cdot \beta_{5.6}^2 \cdot \left[1 + \frac{t_6}{t_4} + \left(\frac{t_6}{t_4}\right)^2\right] + \beta_{5.6}^3 \cdot \left(\frac{t_6}{t_4}\right)^2} \ldots \\ + -1\beta_{5.6} \cdot \left(1 + \frac{t_6}{t_4}\right)\right]$$

$$F_{v.Rk.c} := \begin{array}{|l} 2Johansen_c \text{ if } Rope_{effect} > Johansen_c \\ Johansen_c + Rope_{effect} \text{ otherwise} \end{array} = 8.763 \times 10^3\,N$$

$$Johansen_d := 1.05 \cdot \frac{f_{h.timber.k} \cdot t_4 \cdot d_{thread}}{2 + \beta_{5.6}} \cdot \left[\sqrt{2 \cdot \beta_{5.6} \cdot \left(1 + \beta_{5.6}\right) + \frac{4 \cdot \beta_{5.6} \cdot \left(2 + \beta_{5.6}\right) \cdot M_{y.Rk}}{f_{h.timber.k} \cdot d_{thread} \cdot t_4^2}} - \beta_{5.6}\right]$$

$$F_{v.Rk.d} := \begin{array}{|l} 2Johansen_d \text{ if } Rope_{effect} > Johansen_d \\ Johansen_d + Rope_{effect} \text{ otherwise} \end{array} = 5.24 \times 10^3\,N$$

$$F_{v.Rk.d} = 5.24 \times 10^3\,N$$

$$Johansen_e := 1.05 \cdot \frac{f_{h.timber.k} \cdot t_6 \cdot d_{thread}}{2 + \beta_{5.6}} \cdot \left[\sqrt{2 \cdot \beta_{5.6}^2 \cdot \left(1 + \beta_{5.6}\right) + \frac{4 \cdot \beta_{5.6} \cdot \left(2 + \beta_{5.6}\right) \cdot M_{y.Rk}}{f_{h.timber.k} \cdot d_{thread} \cdot t_6^2}} - \beta_{5.6}\right]$$

$$F_{v.Rk.e} := \begin{array}{|l} 2Johansen_e \text{ if } Rope_{effect} > Johansen_e \\ Johansen_e + Rope_{effect} \text{ otherwise} \end{array} = 9.325 \times 10^3\,N$$

$$F_{v.Rk.e} = 9.325 \times 10^3\,N$$

$$Johansen_f := 1.15 \cdot \sqrt{\frac{2 \cdot \beta_{5.6}}{1 + \beta_{5.6}}} \cdot \sqrt{2M_{y.Rk} \cdot f_{h.timber.k} \cdot d_{thread}} = 3.455 \times 10^3 \, N$$

$$F_{v.Rk.f} := \begin{vmatrix} 2Johansen_f \ \text{ if } \ Rope_{effect} > Johansen_f \\ Johansen_f + Rope_{effect} \ \text{ otherwise} \end{vmatrix} = 5.87 \times 10^3 \, N$$

$$F_{v.Rk} := \min(F_{v.Rk.a}, F_{v.Rk.b}, F_{v.Rk.c}, F_{v.Rk.d}, F_{v.Rk.e}, F_{v.Rk.f}) = 5.24 \cdot kN$$

$k_{mod.timber} := 0.9$ SC 2 Short term action

$\gamma_{M.timber} := 1.3$

$$F_{v.Rd.wall.floor} := \frac{k_{mod.timber} \cdot F_{v.Rk}}{\gamma_{M.timber}} = 3.628 \cdot kN$$

Shear capacity perpendicular to grain

Is taken by the timber beam that the floor is supported on

Axial capacity of the connection

$F_{ax.wall} := F_{ax.3.Rk} + F_{ax.4.Rk} + F_{ax.5.Rk} = 9.659 \times 10^3 \, N$ Withdraw capacity of screw upper wall element

$F_{ax.floor} := F_{ax.6.Rk} + F_{ax.7.Rk} = 1.455 \times 10^4 \, N$ Withdraw capacity of screw floor element

$$F_{horizontal.ax.Rd} := k_{mod.timber} \cdot \frac{\min(F_{ax.wall}, F_{ax.floor})}{\gamma_{M.timber}} = 6.687 \cdot kN$$

$$F_{horizontal.ax.Rd} < f_{tens.k} = 1$$

$$F_{horizontal.Ed} := \cos(56.2deg) \cdot F_{horizontal.ax.Rd} = 3.72 \cdot kN$$

Summary

Wall to wall connection

$F_{v.Rd.wall.wall.0} = 4.621 \cdot kN$ Shear capacity parallel to grain

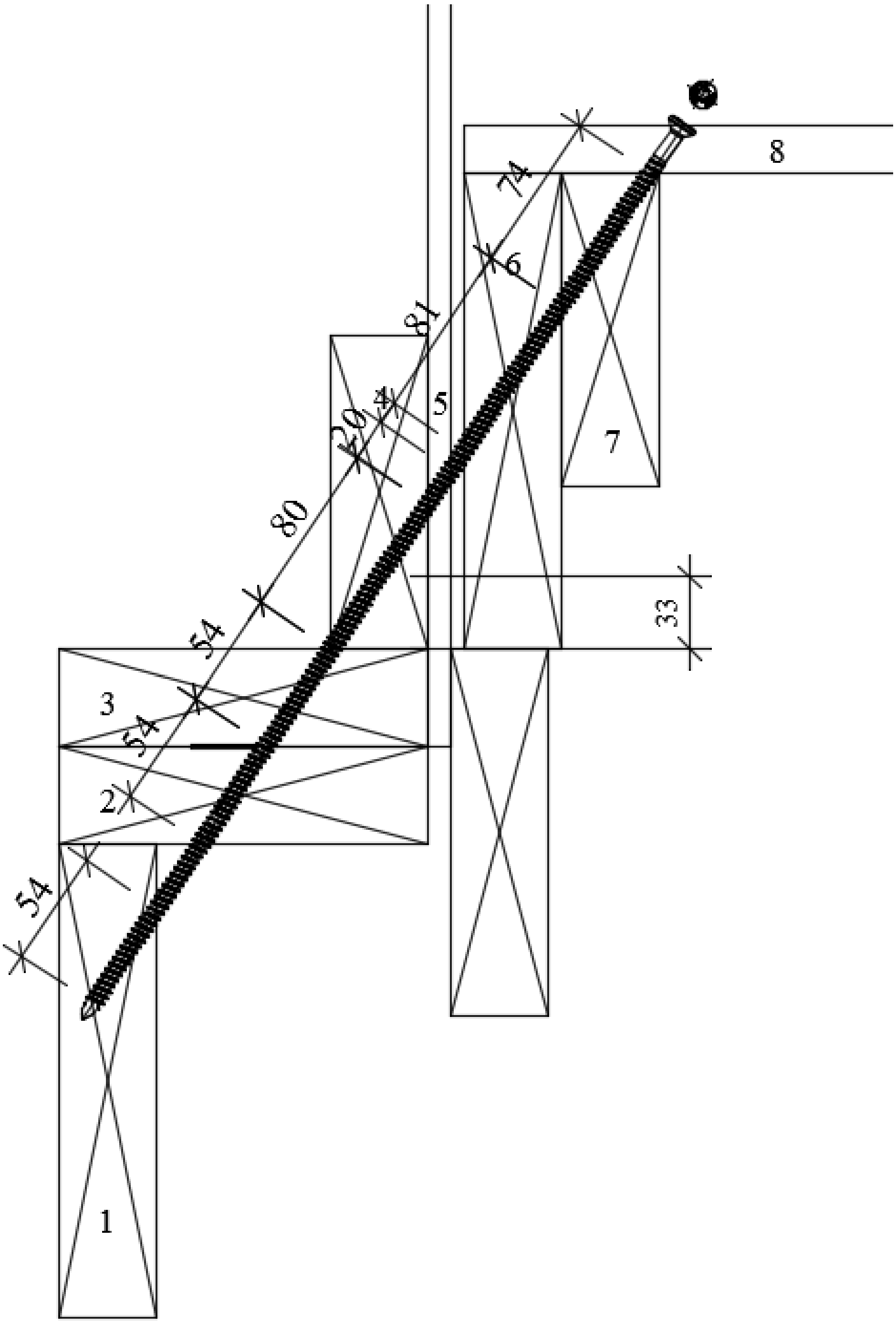
$F_{v.Rd.90.wall} = 3.926 \cdot kN$ Shear capacity perpendicular to grain

Wall to floor connection

$F_{v.Rd.wall.floor} = 3.628 \cdot kN$ Shear capacity parallel to grain

$F_{horizontal.Ed} = 3.72 \cdot kN$ Horizontal capacity

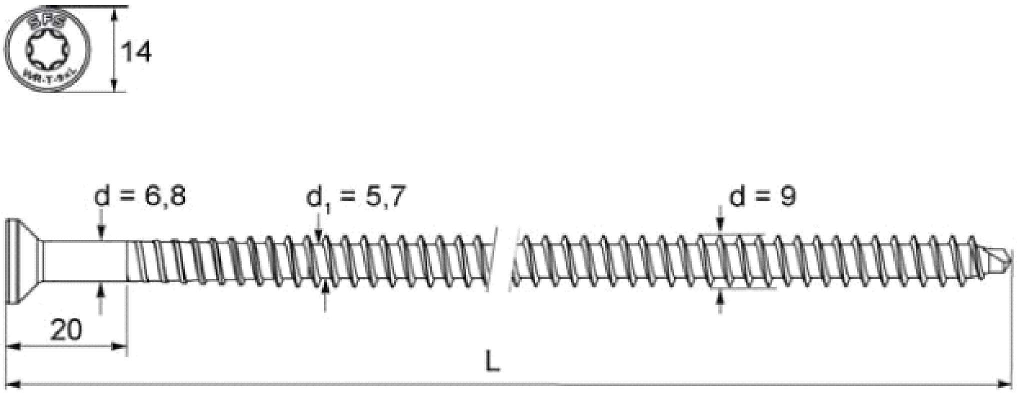
D Structural capacity of improved connection



Material properties screw:

WR-T-9,0 x L

Material: special carbon steel



$d_{screw} := 5.7mm$	Threaded root diameter
$d_{thread} := 9mm$	Threaded diameter
$l_{screw} := 500mm$	Length of screw
$d_{head.screw} := 14mm$	Diameter of the head
$M_{y.Rk} := 19.2N \cdot m$	Value from manufacture according to ETA-12/0062 table A2.1. Since the screw is <u>not</u> produces according to EN14592 fastener values from EC5 does not work.
$f_{tens.k} := 35.9kN$	Characteristic tensile strength from manufacture.

Material properties timber and chip board

$t_1 := 54mm$ Penetrations depth the timber stud, see figure above.

$t_2 := 54mm$

$t_3 := 54mm$

$t_4 := 80mm$

$t_5 := 20mm$

$t_6 := 81mm$

$t_7 := 74mm$

$t_8 := 27\text{mm}$

$t_{particle.board} := 22\text{mm}$ Thickness of particle board

$\rho_{timber} := 350\dfrac{\text{kg}}{\text{m}^3}$ Characteristic density of timber C24

$\rho_{particle.board} := 550\dfrac{\text{kg}}{\text{m}^3}$ Characteristic density of particle board

$$f_{h.timber.k} := 0.082\cdot\left(1 - 0.01\cdot\frac{d_{thread}}{\text{mm}}\right)\cdot\frac{\rho_{timber}}{\frac{\text{kg}}{\text{m}^3}}\cdot\frac{\text{N}}{\text{mm}^2} = 26.117\cdot\text{MPa}$$

EC5 Eq 8.32 embedment strength parallel to grain

$$k_{90} := 1.35 + 0.015\cdot\frac{d_{thread}}{\text{mm}} = 1.485$$

Soft wood Eq.8.33

$$f_{h.90.k} := \frac{f_{h.timber.k}}{k_{90}\cdot\sin(90\text{deg})^2 + \cos(90\text{deg})^2} = 17.587\cdot\text{MPa}$$

Eq.8.31 embedment strength perpendicular to grain.

$$f_{h.chip.board.k} := 50\cdot\left(\frac{d_{thread}}{\text{mm}}\right)^{-0.6}\cdot\left(\frac{t_{particle.board}}{\text{mm}}\right)^{0.2}\cdot\frac{\text{N}}{\text{mm}^2} = 24.826\cdot\text{MPa}$$

EC5 Eq 8.37

Withdrawal strength

$$F_{ax.1.Rk} := 1\cdot 13.5\left(\frac{\rho_{timber}}{350\frac{\text{kg}}{\text{m}^3}}\right)^{0.8}\cdot\frac{t_1}{\text{mm}}\,\frac{d_{thread}}{\text{mm}}\cdot\text{N} = 6.561\times 10^3\,\text{N}$$

Eq 8.40a

$$F_{ax.2.Rk} := 1\cdot 13.5\left(\frac{\rho_{timber}}{350\frac{\text{kg}}{\text{m}^3}}\right)^{0.8}\cdot\frac{t_2}{\text{mm}}\,\frac{d_{thread}}{\text{mm}}\cdot\text{N} = 6.561\times 10^3\,\text{N}$$

$$F_{ax.3.Rk} := 1\cdot 13.5\left(\frac{\rho_{timber}}{350\frac{\text{kg}}{\text{m}^3}}\right)^{0.8}\cdot\frac{t_3}{\text{mm}}\,\frac{d_{thread}}{\text{mm}}\cdot\text{N} = 6.561\times 10^3\,\text{N}$$

$$F_{ax.4.Rk} := 1 \cdot 13.5 \left(\frac{\rho_{timber}}{350 \frac{kg}{m^3}} \right)^{0.8} \cdot \frac{t_4}{mm} \cdot \frac{d_{thread}}{mm} \cdot N = 9.72 \times 10^3 \, N$$

$F_{ax.5.Rk} := 0$ No withdraw capacity in OSB

$$F_{ax.6.Rk} := 1 \cdot 13.5 \left(\frac{\rho_{timber}}{350 \frac{kg}{m^3}} \right)^{0.8} \cdot \frac{t_6}{mm} \cdot \frac{d_{thread}}{mm} \cdot N = 9.841 \times 10^3 \, N$$

$$F_{ax.7.Rk} := 1 \cdot 13.5 \left(\frac{\rho_{timber}}{350 \frac{kg}{m^3}} \right)^{0.8} \cdot \frac{t_7}{mm} \cdot \frac{d_{thread}}{mm} \cdot N = 8.991 \times 10^3 \, N$$

$$F_{ax.8.Rk} := 1 \cdot 13.5 \left(\frac{\rho_{particle.board}}{350 \frac{kg}{m^3}} \right)^{0.8} \cdot \frac{t_8}{mm} \cdot \frac{d_{thread}}{mm} \cdot N = 4.71 \times 10^3 \, N$$

Connection between wall elements

Shear capacity parallel to grain

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

EC5 Eq 8.6 Timber to timber connections for fasteners in single shear

$$F_{v.Rk.a} := f_{h.timber.k} \cdot t_2 \cdot d_{thread} = 1.269 \times 10^4 \, N$$

$$F_{v.Rk.b} := f_{h.timber.k} \cdot t_3 \cdot d_{thread} = 1.269 \times 10^4 \, N$$

$Rope_{effect.1.2} := \dfrac{F_{ax.1.Rk} + F_{ax.2.Rk}}{4} = 3.28 \times 10^3 \, N$ Rope effect from part 1 and 2

$$Johansen_c := \frac{f_{h.timber.k} \cdot t_2 \cdot d_{thread}}{1 + \beta} \cdot \left[\sqrt{\beta + 2 \cdot \beta^2 \cdot \left[1 + \frac{t_3}{t_2} + \left(\frac{t_3}{t_2} \right)^2 \right] + \beta^3 \cdot \left(\frac{t_3}{t_2} \right)^2} - \beta \cdot \left(1 + \frac{t_3}{t_2} \right) \right]$$

$$Johansen_c = 5.258 \times 10^3 \, N$$

$$F_{v.Rk.c} := \begin{cases} 2Johansen_c & \text{if } Rope_{effect.1.2} > Johansen_c \\ Johansen_c + Rope_{effect.1.2} & \text{otherwise} \end{cases} = 8.538 \times 10^3\,N$$

The rope effect should be limited according to 8.2.2(2) for screws to max 100% of the Johansen yield part.

$$Johansen_d := 1.05 \cdot \frac{f_{h.timber.k} \cdot t_2 \cdot d_{thread}}{2+\beta} \cdot \left[\sqrt{2 \cdot \beta \cdot (1+\beta) + \frac{4 \cdot \beta \cdot (2+\beta) \cdot M_{y.Rk}}{f_{h.timber.k} \cdot d_{thread} \cdot t_2^{\,2}}} - \beta\right]$$

$$F_{v.Rk.d} := \begin{cases} 2Johansen_d & \text{if } Rope_{effect.1.2} > Johansen_d \\ Johansen_d + Rope_{effect.1.2} & \text{otherwise} \end{cases} = 8.089 \times 10^3\,N$$

$$Johansen_e := 1.05 \cdot \frac{f_{h.timber.k} \cdot t_3 \cdot d_{thread}}{2+\beta} \cdot \left[\sqrt{2 \cdot \beta^2 \cdot (1+\beta) + \frac{4 \cdot \beta \cdot (2+\beta) \cdot M_{y.Rk}}{f_{h.timber.k} \cdot d_{thread} \cdot t_3^{\,2}}} - \beta\right]$$

$$F_{v.Rk.e} := \begin{cases} 2Johansen_e & \text{if } Rope_{effect.1.2} > Johansen_e \\ Johansen_e + Rope_{effect.1.2} & \text{otherwise} \end{cases} = 8.089 \times 10^3\,N$$

$$Johansen_f := 1.15 \cdot \sqrt{\frac{2 \cdot \beta}{1+\beta}} \cdot \sqrt{2M_{y.Rk} \cdot f_{h.timber.k} \cdot d_{thread}} = 3.455 \times 10^3\,N$$

$$F_{v.Rk.f} := \begin{cases} 2Johansen_f & \text{if } Rope_{effect.1.2} > Johansen_f \\ Johansen_f + Rope_{effect.1.2} & \text{otherwise} \end{cases} = 6.735 \times 10^3\,N$$

$$F_{v.Rk} := \min\left(F_{v.Rk.a}, F_{v.Rk.b}, F_{v.Rk.c}, F_{v.Rk.d}, F_{v.Rk.e}, F_{v.Rk.f}\right) = 6.735 \cdot kN$$

$k_{mod.timber} := 0.9$ SC 2 Short term action

$\gamma_{M.timber} := 1.3$

$$F_{v.Rd.wall.wall.0} := \frac{k_{mod.timber} \cdot F_{v.Rk}}{\gamma_{M.timber}} = 4.663 \cdot kN$$

Shear capacity perpendicular to grain

$$\beta := \frac{f_{h.90.k}}{f_{h.90.k}} = 1$$

EC5 Eq 8.6 Timber to timber connections for fasteners in single shear

$$F_{v.Rk.a.90.wall} := f_{h.90.k} \cdot t_2 \cdot d_{thread} = 8.547 \times 10^3 \, N$$

$$F_{v.Rk.b.90.wall} := f_{h.90.k} \cdot t_3 \cdot d_{thread} = 8.547 \times 10^3 \, N$$

$$Rope_{effect.1.2} = 3.28 \times 10^3 \, N$$

$$Johansen_{c.90.wall} := \frac{f_{h.90.k} \cdot t_2 \cdot d_{thread}}{1+\beta} \cdot \left[\sqrt{\beta + 2\cdot\beta^2 \cdot \left[1 + \frac{t_3}{t_2} + \left(\frac{t_3}{t_2}\right)^2 \right] + \beta^3 \cdot \left(\frac{t_3}{t_2}\right)^2} - \beta \cdot \left(1 + \frac{t_3}{t_2}\right) \right]$$

$$F_{v.Rk.c.90.wall} := \begin{cases} 2Johansen_{c.90.wall} & \text{if } Rope_{effect.1.2} > Johansen_{c.90.wall} \\ Johansen_{c.90.wall} + Rope_{effect.1.2} & \text{otherwise} \end{cases} = 6.821 \times 10^3 \, N$$

$$F_{v.Rk.c.90.wall} = 6.821 \times 10^3 \, N$$

$$Johansen_{d.90.wall} := 1.05 \cdot \frac{f_{h.90.k} \cdot t_2 \cdot d_{thread}}{2+\beta} \cdot \left[\sqrt{2\cdot\beta\cdot(1+\beta) + \frac{4\cdot\beta\cdot(2+\beta)\cdot M_{y.Rk}}{f_{h.90.k}\cdot d_{thread}\cdot t_2^2}} - \beta \right]$$

$$F_{v.Rk.d.90.wall} := \begin{cases} 2Johansen_{d.90.wall} & \text{if } Rope_{effect.1.2} > Johansen_{d.90.wall} \\ Johansen_{d.90.wall} + Rope_{effect.1.2} & \text{otherwise} \end{cases} = 6.634 \times 10^3 \, N$$

$$Johansen_{e.90.wall} := 1.05 \cdot \frac{f_{h.90.k} \cdot t_3 \cdot d_{thread}}{2+\beta} \cdot \left[\sqrt{2\cdot\beta^2\cdot(1+\beta) + \frac{4\cdot\beta\cdot(2+\beta)\cdot M_{y.Rk}}{f_{h.90.k}\cdot d_{thread}\cdot t_3^2}} - \beta \right]$$

$$F_{v.Rk.e.90.wall} := \begin{cases} 2Johansen_{e.90.wall} & \text{if } Rope_{effect.1.2} > Johansen_{e.90.wall} \\ Johansen_{e.90.wall} + Rope_{effect.1.2} & \text{otherwise} \end{cases} = 6.634 \times 10^3 \, N$$

$$Johansen_{f.90.wall} := 1.15 \cdot \sqrt{\frac{2\cdot\beta}{1+\beta}} \cdot \sqrt{2M_{y.Rk}\cdot f_{h.90.k}\cdot d_{thread}} = 2.835 \times 10^3 \, N$$

$$F_{v.Rk.f.90.wall} := \begin{cases} 2Johansen_{f.90.wall} & \text{if } Rope_{effect.1.2} > Johansen_{f.90.wall} \\ Johansen_{f.90.wall} + Rope_{effect.1.2} & \text{otherwise} \end{cases} = 5.67 \times 10^3 \, N$$

$$F_{v.Rk.90.wall.1} := \min\left(F_{v.Rk.a.90.wall}, F_{v.Rk.b.90.wall}, F_{v.Rk.c.90.wall}\right) = 6.821 \cdot kN$$

$$F_{v.Rk.90.wall.2} := \min\left(F_{v.Rk.d.90.wall}, F_{v.Rk.e.90.wall}, F_{v.Rk.f.90.wall}\right) = 5.67 \times 10^{3} N$$

$$F_{v.Rk.90.wall} := \min\left(F_{v.Rk.90.wall.1}, F_{v.Rk.90.wall.2}\right) = 5.67 \times 10^{3} N$$

$$F_{v.Rd.90.wall} := \frac{k_{mod.timber} \cdot F_{v.Rk.90.wall}}{\gamma_{M.timber}} = 3.926 \cdot kN$$

Connection between floor and wall elements

Shear capacity parallel to grain

$$\beta_{5.6} := \frac{f_{h.timber.k}}{f_{h.timber.k}} = 1$$

EC5 Eq 8.6 Timber to timber connections for fasteners in single shear

$$F_{v.Rk.a} := f_{h.timber.k} \cdot t_4 \cdot d_{thread} = 1.88 \times 10^{4} N$$

$$F_{v.Rk.b} := f_{h.timber.k} \cdot t_6 \cdot d_{thread} = 1.904 \times 10^{4} N$$

$$Rope_{effect.6.7.8} := \frac{F_{ax.6.Rk} + F_{ax.7.Rk} + F_{ax.8.Rk}}{4} = 5.886 \times 10^{3} N$$

$$Rope_{effect.3.4} := \frac{F_{ax.3.Rk} + F_{ax.4.Rk}}{4} = 4.07 \cdot kN$$

$$Rope_{effect} := \min\left(Rope_{effect.6.7.8}, Rope_{effect.3.4}\right) = 4.07 \times 10^{3} N$$

$$Johansen_c := \frac{f_{h.timber.k} \cdot t_4 \cdot d_{thread}}{1 + \beta_{5.6}} \cdot \left[\sqrt{\beta_{5.6} + 2 \cdot \beta_{5.6}^{2} \cdot \left[1 + \frac{t_6}{t_4} + \left(\frac{t_6}{t_4}\right)^2\right] + \beta_{5.6}^{3} \cdot \left(\frac{t_6}{t_4}\right)^2} \ldots \atop + -1 \beta_{5.6} \cdot \left(1 + \frac{t_6}{t_4}\right)\right]$$

$$F_{v.Rk.c} := \begin{cases} 2 Johansen_c & \text{if } Rope_{effect} > Johansen_c \\ Johansen_c + Rope_{effect} & \text{otherwise} \end{cases} = 1.191 \times 10^{4} N$$

$$Johansen_d := 1.05 \cdot \frac{f_{h.timber.k} \cdot t_4 \cdot d_{thread}}{2 + \beta_{5.6}} \cdot \left[\sqrt{2 \cdot \beta_{5.6} \cdot \left(1 + \beta_{5.6}\right) + \frac{4 \cdot \beta_{5.6} \cdot \left(2 + \beta_{5.6}\right) \cdot M_{y.Rk}}{f_{h.timber.k} \cdot d_{thread} \cdot t_4^{2}}} - \beta_{5.6}\right]$$

$$F_{v.Rk.d} := \begin{cases} 2Johansen_d & \text{if } Rope_{effect} > Johansen_d \\ Johansen_d + Rope_{effect} & \text{otherwise} \end{cases} = 1.09 \times 10^4 N$$

$$F_{v.Rk.d} = 1.09 \times 10^4 N$$

$$Johansen_e := 1.05 \cdot \frac{f_{h.timber.k} \cdot t_6 \cdot d_{thread}}{2 + \beta_{5.6}} \cdot \left[\sqrt{2 \cdot \beta_{5.6}^2 \cdot (1 + \beta_{5.6}) + \frac{4 \cdot \beta_{5.6} \cdot (2 + \beta_{5.6}) \cdot M_{y.Rk}}{f_{h.timber.k} \cdot d_{thread} \cdot t_6^2}} - \beta_{5.6} \right]$$

$$F_{v.Rk.e} := \begin{cases} 2Johansen_e & \text{if } Rope_{effect} > Johansen_e \\ Johansen_e + Rope_{effect} & \text{otherwise} \end{cases} = 1.098 \times 10^4 N$$

$$F_{v.Rk.e} = 1.098 \times 10^4 N$$

$$Johansen_f := 1.15 \cdot \sqrt{\frac{2 \cdot \beta_{5.6}}{1 + \beta_{5.6}}} \cdot \sqrt{2M_{y.Rk} \cdot f_{h.timber.k} \cdot d_{thread}} = 3.455 \times 10^3 N$$

$$F_{v.Rk.f} := \begin{cases} 2Johansen_f & \text{if } Rope_{effect} > Johansen_f \\ Johansen_f + Rope_{effect} & \text{otherwise} \end{cases} = 6.91 \times 10^3 N$$

$$F_{v.Rk} := \min(F_{v.Rk.a}, F_{v.Rk.b}, F_{v.Rk.c}, F_{v.Rk.d}, F_{v.Rk.e}, F_{v.Rk.f}) = 6.91 \cdot kN$$

$k_{mod.timber} := 0.9$ SC 2 Short term action

$\gamma_{M.timber} := 1.3$

$$F_{v.Rd.wall.floor} := \frac{k_{mod.timber} \cdot F_{v.Rk}}{\gamma_{M.timber}} = 4.784 \cdot kN$$

Shear capacity perpendicular to grain

Is taken by the timber beam that the floor is supported on.

Axial capacity of the connection

$F_{ax.wall} := F_{ax.3.Rk} + F_{ax.4.Rk} \cdot 1100 + F_{ax.5.Rk} = 1.07 \times 10^7 N$ Withdraw capacity of screw upper wall element

$F_{ax.floor} := F_{ax.6.Rk} + F_{ax.7.Rk} + F_{ax.8.Rk} = 2.354 \times 10^4 N$ Withdraw capacity of screw floor element

$$F_{horizontal.ax.Rd} := k_{mod.timber} \cdot \frac{\min(F_{ax.wall}, F_{ax.floor})}{\gamma_{M.timber}} = 16.298 \cdot kN$$

$F_{horizontal.ax.Rd} < f_{tens.k} = 1$

$F_{horizontal.Ed} := \cos(56.2deg) \cdot F_{horizontal.ax.Rd} = 9.067 \cdot kN$

Summary

Wall to wall connection

$F_{v.Rd.wall.wall.0} = 4.663 \cdot kN$	Shear capacity parallel to grain
$F_{v.Rd.90.wall} = 3.926 \cdot kN$	Shear capacity perpendicular to grain

Wall to floor connection

$F_{v.Rd.wall.floor} = 4.784 \cdot kN$	Shear capacity parallel to grain
$F_{horizontal.Ed} = 9.067 \cdot kN$	Horizontal capacity

E Design of other connections

Plywood to edge beam

Round nails:

$d_{nail} := 2.8mm$ — Diameter nail

$l_{nail} := 75mm$ — Length of nail

$d_{head.nail} := 2 \cdot d_{nail} = 5.6 \cdot mm$ — Diameter of the head

$f_{u.k} := 600MPa$

$f_{y.k} := 355MPa$

$o_1 := mm^{0.4}$ — Unit correction for Mathcad

$M_{y.Rk} := 0.3 \cdot f_{u.k} \cdot o_1 \cdot d_{nail}^{2.6} = 2.617 \cdot kN \cdot mm$ — EC5 Eq 8.14 round nails

Plywood S 2x12mm

$t_1 := 24mm$

$k_{mod.plywood} := 0.9$ — Short term load service class 1 table 3.1 EC 5

$\gamma_{M.plywood} := 1.2$ — Table 2.3 EC 5

$\rho_{plywood} := 410 \frac{kg}{m^3}$ — Characteristic density of plywood

$o := \frac{m^3}{kg} \cdot MPa \cdot mm^{0.3}$ — Unit correction for Mathcad

$f_{h.1.k} := 0.11 \cdot \rho_{plywood} \cdot o \cdot d_{nail}^{-0.3} = 33.115 \cdot MPa$ — EC5 Eq 8.20

Timber beam C24

$t_2 := l_{nail} - t_1 = 51 \cdot mm$

$k_{mod.timber} := 0.9$ — Short term load service class 1 table 3.1 EC 5

$\gamma_{M.timber} := 1.3$ — Table 2.3 EC 5

$\rho_{timber} := 350 \frac{kg}{m^3}$ — Characteristic density of timber C24

$f_{h.2.k} := 0.082 \cdot \rho_{timber} \cdot o \cdot d_{nail}^{-0.3} = 21.073 \cdot MPa$

Calculation of one nail

$$\beta := \frac{f_{h.2.k}}{f_{h.1.k}} = 0.636$$ EC5 Eq 8.8

$$F_{ax.Rk} := 0$$ Unknown value assumed 0 according to 8.2.2(2)

$$d := d_{nail} = 2.8 \cdot mm$$

EC5 Eq 8.6 Timber to timber connections for fasteners in single shear

$$F_{v.Rk.a} := f_{h.1.k} \cdot t_1 \cdot d = 2.225 \times 10^3 \, N$$

$$F_{v.Rk.b} := f_{h.2.k} \cdot t_2 \cdot d = 3.009 \times 10^3 \, N$$

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

$$F_{v.Rk.d} := 1.05 \cdot \frac{f_{h.1.k} \cdot t_1 \cdot d}{2 + \beta} \cdot \left[\sqrt{2 \cdot \beta \cdot (1 + \beta) + \frac{4 \cdot \beta \cdot (2 + \beta) \cdot M_{y.Rk}}{f_{h.1.k} \cdot d \cdot t_1^{\,2}}} - \beta \right] + \frac{F_{ax.Rk}}{4} = 812.338 \, N$$

$$F_{v.Rk.e} := 1.05 \cdot \frac{f_{h.1.k} \cdot t_2 \cdot d}{2 + \beta} \cdot \left[\sqrt{2 \cdot \beta^2 \cdot (1 + \beta) + \frac{4 \cdot \beta \cdot (2 + \beta) \cdot M_{y.Rk}}{f_{h.1.k} \cdot d \cdot t_2^{\,2}}} - \beta \right] + \frac{F_{ax.Rk}}{4} = 1.028 \times 10^3 \, N$$

$$F_{v.Rk.f} := 1.15 \cdot \sqrt{\frac{2 \cdot \beta}{1 + \beta}} \cdot \sqrt{2 M_{y.Rk} \cdot f_{h.1.k} \cdot d} + \frac{F_{ax.Rk}}{4} = 706.604 \, N$$

$$F_{v.Rk} := \min\left(F_{v.Rk.a}, F_{v.Rk.b}, F_{v.Rk.c}, F_{v.Rk.d}, F_{v.Rk.e}, F_{v.Rk.f}\right) = 706.604 \cdot N$$

$$F_{v.Rd} := F_{v.Rk} \cdot \frac{\sqrt{k_{mod.timber} \cdot k_{mod.plywood}}}{\max\left(\gamma_{M.timber}, \gamma_{M.plywood}\right)} = 489.187 \, N$$

$$R_d := 1.2 \cdot F_{v.Rd} = 587.025 \, N$$ EC5 Eq 9.2.4.2(5)

Edge distances

$$a_{3.t.plywood} := (3 + 4 \cdot \sin(90deg)) \cdot d = 19.6 \cdot mm$$ Loaded edge for plywood

$$a_{4.t.timber} := (5 + 2 \sin(90)) \cdot d = 19.006 \cdot mm$$ Loaded edge for timber

These are ok since there below 45/2 mm

Shortest distances between nails

$a_{1.plywood} := 0.85\cdot(5 + 5\cos(0deg))d = 23.8\cdot mm$

$a_{2.plywood} := 0.85\cdot(5d) = 11.9\cdot mm$

$a_{1.timber} := (5 + 5\cos(0deg))d = 28\cdot mm$

$s_{nail} := \max(a_{1.plywood}, a_{2.plywood}, a_{1.timber}) = 28\cdot mm$

$Force := 22.5kN$ Value from FEM-models

$n_{nails} := \mathrm{round}\left(\frac{Force}{R_d}\right) = 38$ Number of nails needed

$s_{nail} := \frac{1200mm}{n_{nails} + 1} = 30.769\cdot mm$ Spacing is rounded down to 30mm to make it easy on building site.

Timber nogging to wall element

Just withdraw capacity

SFS WT-T 6.5x90

L_{ef} is 40mm

$$R_{ax.k.WT.6.5} := 1\cdot 12.9\cdot\left(\frac{\rho_{timber}}{350\frac{kg}{m^3}}\right)^{0.8}\cdot\frac{N}{mm^2}\cdot 40mm\cdot 6.5mm = 3.354\cdot kN$$

$$R_{ax.d.WT.6.5} := \frac{k_{mod.timber}\cdot R_{ax.k.WT.6.5}}{\gamma_{M.timber}} = 2.322\cdot kN$$

$n_{WT.6.5} := \frac{Force}{R_{ax.d.WT.6.5}} = 9.69$ Gives 10 screws

Plywood to plywood

$t_1 := 12mm$

$f_{h.1.k} := 0.11\cdot\rho_{plywood}\cdot o\cdot d_{nail}^{-0.3} = 33.115\cdot MPa$ EC5 Eq 8.20

$t_2 := 12mm$

$f_{h.2.k} := 0.11\cdot\rho_{plywood}\cdot o\cdot d_{nail}^{-0.3}$

Calculation of one nail

$\beta := \frac{f_{h.2.k}}{f_{h.1.k}} = 1$ EC5 Eq 8.8

$F_{ax.Rk} := 0$

Unknown value assumed 0 according to 8.2.2(2)

$d := d_{nail} = 2.8 \cdot mm$

EC5 Eq 8.6 Timber to timber connections for fasteners in single shear

$$F_{v.Rk.a} := f_{h.1.k} \cdot t_1 \cdot d = 1.113 \times 10^3 \, N$$

$$F_{v.Rk.b} := f_{h.2.k} \cdot t_2 \cdot d = 1.113 \times 10^3 \, N$$

$$F_{v.Rk.c} := \frac{f_{h.1.k} \cdot t_1 \cdot d}{1 + \beta} \cdot \left[\sqrt{\beta + 2 \cdot \beta^2 \cdot \left[1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1} \right)^2 \right] + \beta^3 \cdot \left(\frac{t_2}{t_1} \right)^2} - \beta \cdot \left(1 + \frac{t_2}{t_1} \right) \right] \ldots = 460.885 \, N$$
$$+ \frac{F_{ax.Rk}}{4}$$

$$F_{v.Rk.d} := 1.05 \cdot \frac{f_{h.1.k} \cdot t_1 \cdot d}{2 + \beta} \cdot \left[\sqrt{2 \cdot \beta \cdot (1 + \beta) + \frac{4 \cdot \beta \cdot (2 + \beta) \cdot M_{y.Rk}}{f_{h.1.k} \cdot d \cdot t_1^{\,2}}} - \beta \right] + \frac{F_{ax.Rk}}{4} = 592.099 \, N$$

$$F_{v.Rk.e} := 1.05 \cdot \frac{f_{h.1.k} \cdot t_2 \cdot d}{2 + \beta} \cdot \left[\sqrt{2 \cdot \beta^2 \cdot (1 + \beta) + \frac{4 \cdot \beta \cdot (2 + \beta) \cdot M_{y.Rk}}{f_{h.1.k} \cdot d \cdot t_2^{\,2}}} - \beta \right] + \frac{F_{ax.Rk}}{4} = 592.099 \, N$$

$$F_{v.Rk.f} := 1.15 \cdot \sqrt{\frac{2 \cdot \beta}{1 + \beta}} \cdot \sqrt{2 M_{y.Rk} \cdot f_{h.1.k} \cdot d} + \frac{F_{ax.Rk}}{4} = 801.213 \, N$$

$$F_{v.Rk} := \min\left(F_{v.Rk.a}, F_{v.Rk.b}, F_{v.Rk.c}, F_{v.Rk.d}, F_{v.Rk.e}, F_{v.Rk.f}\right) = 460.885 \cdot N$$

$$F_{v.Rd} := F_{v.Rk} \cdot \frac{k_{mod.plywood}}{\gamma_{M.plywood}} = 345.664 \, N$$

$R_{d.plywood.plywood} := 1.2 \cdot F_{v.Rd} = 414.797 \, N$

EC5 Eq 9.2.4.2(5)

$$n_{nails} := \mathrm{round}\left(\frac{\frac{Force}{2 \cdot 2}}{R_{d.plywood.plywood}} \right) = 14$$

Number of nails needed. The force is divided by 2*2 because of 2 plywood boards and 2 Kerto-S beams

$$n_{nails} := \frac{1200mm}{n_{nails} + 1} = 80 \cdot mm$$

Spacing is 80mm.