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Stabilisation of higher wooden houses in volume building technology

Stabilisering av högre trähus inom volymbyggnadsteknik

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Foreword

This master thesis has been carried out during spring 2020 as a cooperation between the Division of Structural Engineering at Lund's Faculty of Engineering, LTH, and Derome Husproduktion AB. The thesis comprises 30 hp and accomplishes five years of studies for the Master of Science in Civil Engineering.

Several people have supported me during the writing of this thesis and whom I want to thank. First of all, I would like to thank my supervisor Eva Frühwald Hansson at Division of Structural Engineering, who guided me through this project giving great support and answers to all my questions. I would also like to thank my supervisor at Derome, Joakim Seldert, who with his deep insight in the building technology of wooden houses, been very supporting. I would also like to thank the rest of the team at Derome that I have been in contact with, for all the help and for the nice tour of your factory.

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Ebba Gipperth

Abstract

The construction of residential buildings with timber as load-bearing system has according to Statistics Sweden increased with 85% since 2011, which is the latest low point of the collected data. Statistics Sweden also shows how residential buildings consisting of 4-8 storeys are, in Sweden, the most built type of house today. Of these statistics, 4-8-storey residential buildings made with timber as load-bearing system predicts to be more requested in the near future.

Together with Derome Husproduktion AB, an investigation about how to design higher timber buildings with volume building technology has been performed. The bracing system for buildings made of modules (also called volumes) are shear walls, which are all the outer walls of every module, and the investigation has focused on how the horizontal wind load will be carried by the shear walls, and further down to the foundation.

Volume building technology means that volumes are produced at a factory where the modules are in the greatest extent completed with tiles, floor, kitchen fitments, bathroom etc. When the assembling of the modules is completed, the modules are transported by a truck to the building site and a crane can place the modules in the right spot to create a complete building. When the modules are in place, a facade and a roof are attached to the outside of the modules. Today, Derome Husproduktion AB builds houses with volume building technology with a maximum height of four storeys, but there is a desire to build both six and eight storeys as well. An investigation has been done to ascertain the amount of load a building of both six and eight storeys can be exposed to and how the capacity of a four-storey building stands against these loads. An already designed building is investigated to get an example of how the modules can be placed in a building. The investigated building consists of two structures made of modules with a corridor in between. The stabilising shear walls are the enclosing walls of every module. All the necessary data and dimensions of the building been provided by Derome.

The wind load has been designed according to Eurocode and the capacity of the shear walls has been designed with two methods, Method A from Eurocode and an elastic method from Carling et al (1992). Method A has been used to a greater extent in the analysis since it is the method that Eurocode recommends, and it is the method that Derome uses today. Two of the walls have been analysed with the elastic method to be able to observe the differences between the two methods.

According to both methods, all the walls in a six-storey building with the same floor plan and wall set-up as the investigated four-storey building, will hold for the horizontal loads it is exposed for. However, some of the transverse walls will fail to meet the requirements given by the Eurocode due to a lack of shear capacity when the height of the building is increased to eight storeys.

The capacity of the shear walls can be increased through several parameters. For example, the location of the openings in the walls, a well-chosen location increases the capacity of the wall. The number of short segments should be kept low since the capacity of short segments has to be reduced according to Method A. Other ways to increase the capacity is for example to use staples with greater capacity or to make sure that forces can be transmitted through all walls in the building, especially the transverse walls. This can be done by applying beams in the corridor between the structures which are designed to carry the loads from one structure to the other.

In addition to which the walls need to resist the horizontal loads created by the wind load, the risk of tilting and the need for anchoring of modules are also analysed for the building. Both for a six- and eight storey building there is a risk for tilting of the structures and connections between the structures or anchoring of the structure to the ground is necessary. It is the modules closest to the gables that will be affected the most of tension loads created on the building, and a design of how much the connectors between these modules and the rest of the buildings will have to resist, has been done. The connections between the top modules and the modules below are also analysed to secure that the up-lifting wind load is not greater than the self-weight of the top modules.

As a summery, it is considered possible to raise the height of the building to six storeys with the same floor plan and wall set-up but with a adjustment to reduce the risk for tilting while greater efforts are needed, for example through relocation of openings, a different wall set-up and connections with a greater capacity between some modules to make it possible to raise the building to eight storeys.

Sammanfattning

Byggnationen av bostadshus med trästomme har enligt Statistiska centralbyrån ökat med 85% efter den senast lägsta uppmätningen 2011. Statistiska centralbyrån menar också att det är bostadshus bestående av 4-8 våningar som byggs mest i Sverige idag. Av denna statistik förutspås det bli en ökning av efterfrågan på bostadshus med 4-8 våningar med trästomme inom den närmsta framtiden.

Tillsammans med Derome Husproduktion AB har en undersökning om dimensioneringen av högre trähus byggda av volymelement genomförts. Det stabiliserande systemet är i detta fall de modulavskiljande väggarna och undersökningen har fokuserat på hur den horisontella lasten i form av vindlast ska tas upp av skjuvväggarna i byggnaden för att i sin tur kunna ta ner lasten till grunden.

Att bygga ett hus av volymelement betyder att moduler (även kallade volymer) produceras på fabrik där de i största mån görs helt färdiga med kakel, golv, köksinredning, badrum etc. När monteringen i fabrik är klar fraktas volymerna på lastbil till byggarbetsplatsen där en lyftkran kan placera volymerna på plats för att därigenom bli till ett färdigt hus. När volymerna är på plats kläs dessa in med en fasad och ett förtillverkat tak lyfts på plats. Idag bygger Derome Husproduktion AB volymhus av trä med en maximal höjd på fyra våningar, men har en önskan om att bygga både sex och åtta våningar. En undersökning har därför genomförts för att ta reda på hur mycket last en byggnad på både sex och åtta våningar utsätts för och hur kapaciteten i dagens fyravåningshus står sig mot dessa laster. Ett redan dimensionerat hus har studerats för att få ett exempel på hur modulerna kan placeras i ett hus. Referensbyggnaden består av två längor med moduler med en gemensam korridor emellan. De stabiliserande skjuvväggarna är de omslutande väggarna av varje modul. All nödvändig information och dimensioner av referensbyggnaden har tillhandahållits av Derome.

Vindlasten har beräknats enligt Eurokod och kapaciteten av skjuvväggarna har beräknats enligt två metoder, Metod A från Eurokod och en elastisk metod från Carling et al (1992). Metod A har använts i större grad i analysen eftersom det är den metod som Eurokod rekommenderar att använda samt att det är den metod som Derome använder idag. Två av väggarna har analyserats med den elastiska metoden för att kunna observera skillnader de två metoderna emellan.

Enligt båda metoderna ska väggarna i ett sexvåningshus, med samma planlösning och vägguppbyggnad som det undersökta fyravåningshuset, klara den horisontella last de utsätts för. Däremot kommer en del av de transversella väggarna i huset inte möta kraven givna i Eurokod på grund av brist på tvärkraftskapacitet, när höjden ökas till ett åttavåningshus.

Skjuvkapaciteten kan ökas med en rad olika parametrar. Exempelvis kan en väl vald placering av öppningarna i väggarna ge högre kapacitet. Kapaciteten av korta väggsegment måste reduceras enligt Metod A och därför ska mängden korta segment reduceras i största mån för att få en högre kapacitet i väggarna. Andra sätt att öka kapacitetn är till exempel att använda klamrar med högre kapacitet eller att se till att krafter kan överföras mellan alla väggar i huset, alltså skapa samverkan mellan väggarna i den tvärgående riktnignen av huset. Detta kan göras genom att applicera balkar i korridoren mellan modulerna som är dimesnionerade för att klara att föra över lasterna som bildas.

Utöver det att väggarna måste kunna motstå den horisontella last som uppstår, undersöks även risken för stjälpning av byggnaden och behovet av förankring av moduler. Både för ett sex- och åttavåningshus finns det en risk för att längorna av moduler kommer att välta och antingen behövs bättre anslutningar mellan längorna av moduler eller en förankring ner i grunden för att motstå stjälpning. Det är modulerna längst ut i kanterna på huset som kommer påverkas mest av de sugande krafter som uppstår på huset och en dimensionering av hur stor kapacitet kopplingarna, från resten av huset till dessa moduler måste ha, har därför genomförts. Även kopplingarna mellan de översta och näst översta modulerna har dimensionerats för att säkerställa att de lyftande vindlasterna på taket är lägre än egentyngden av modulerna.

Sammanfattningsvis bedöms det som möjligt att öka byggnadshöjden till sex våningar med bibehållen planlösning och vägguppbyggnad men med en åtgärd för att minska risken för stjälpnig medan det krävs större ansträngningar, till exempel genom flytt av öppningar, annan vägguppbyggnad och anslutningar med högre kapacitet mellan vissa moduler för att en ökning av byggnadshöjden till åtta våningar ska vara möjlig.

Contents

Chapter 1

Introduction

1.1 Background

Timber as a building material, especially for higher buildings, is getting more popular and the buildings with 4-8 storeys are the most requested and built type of buildings today (SCB, 2020). Buildings of this height have normally been constructed with timber as a frame system over the years and this approach is still considered as a good choice. The highest wooden building today is 85.4 meters and located in Brumunddal, Norway. The building, called *Mjøstårnet*, has 18 storeys and is built with cross-laminated timber (Alpman, 2019). CLT consists of glued boards or planks layered alternately at right-angles, giving good strength properties. Together with a high degree of prefabrication, it gives a good fit for the structural systems of higher buildings (Svenskt trä, 2017). Even if the knowledge about construction of higher wooden buildings has increased, there is a need to further understand how to construct high buildings in wood, particularly those mostly built.

In order to more precisely understand the need of knowledge about stabilisation of higher wooden buildings in volume building technology, a contact with Derome Husproduktion AB was made. Derome Husproduktion AB has a concept for building named "Adder-aPluss", where they build modules which can be combined in different ways. The modules are industrially made and thereafter transported to the building site where they can be assembled as blocks. The maximum height of "AdderaPluss" today is four floors. Derome wants to raise their standard height of these buildings with volume building technology to six or eight storeys. Higher building will however result in more horizontal load acting on the buildings, leading to higher laods on the shear walls. There are three well-known ways to stabilise a building: truss, frame and shear walls. When building with prefabricated plane or volume elements, shear walls is a common way to stabilise the building horizontally. The design of these shear walls for higher buildings will be analysed in this report.

1.2 Purpose

The purpose of this master thesis is to analyse the resistance of the shear walls of a four-storey building made of timber volume elements and to investigate how to design the shear walls for the same type of buildings with six or eight storeys.

1.3 Limitations

This thesis is only focusing on the horizontal and vertical load-bearing capacity of the walls of the modules and not on any other parameter essential for the design of the modules, such as fire, moisture, or acoustics. The study is limited to Derome's existing design of volume elements, "AdderaPluss". An already designed building will be the reference object for this thesis.

The wind load is the only analysed action in this report since this is the major load, and a detailed description and design of this load can be found in appendix B. Loads from sway imperfections, eccentric loads and earthquakes also categorise as a horizontal loads but no consideration will be taken in regards to these loads in the report. The internal wind load for the building is also neglected throughout the report. The results in this report can only be used during an early design stage to determine approximate results for the investigated parts of the building.

1.4 Research questions

1. How much load are the existing construction elements (the walls) capable of carrying today and how does this capacity meet the needs for higher buildings?
2. How can the elements be designed to resist the higher loads acting on a higher building?
3. Where and why are there a need for connections between the modules to withstand higher wind loads?

1.5 Outline of thesis

A literature review has been done to investigate the development of the construction of buildings with volume elements through time, how and why it is built with wooden elements today and how to design the stabilisation system with shear walls for higher wooden buildings (chapter 2). In chapter 3, different types of horizontal stabilisation systems will be presented with most focus on shear walls. Two methods will also be presented on how to calculate the capacity of the shear walls. These first chapters are the result of the literature review and further on in the report, the knowledge is used to investigate a reference building, the reference building will be presented in chapter 1.6.

In chapter 4, a description on how to calculate the wind loads and and the capacities of the walls for the two different methods will be presented. In chapter 4, the design to obtain the risk for tilting of the building and the need for anchoring of the modules will also be presented. The results obtained in the study are presented and discussed in chapter 5. The conclusions of the study can be found in chapter 6.

More detailed descriptions of the relevant data used in the report are found in the appendix. That includes more insight in the investigated building (appendix A), calculations of wind load (appendix B), an analysis of the risk for up-lifting forces on the roof structure (appendix C), the design of the capacities of the walls (appendix D and E) and an analysis of the risk of tilting and the need for anchoring of the modules (appendix F).

The structure of this thesis is designed to present the reader with all necessary data, formulas and explanations in the report and, if it is of interest, a more detailed description of the different parts can be found in the appendix. Therefore, some of the equations and the explanations are found both in the report and in the appendix.

1.6 Short introduction of investigated building

The building of interest for this project is an already designed project from Derome, but with a few changes. In reality this is a 1+4 storey structure where the first floor is made of concrete and the floors above are made of timber. For this project, the first floor is neglected, and a four-storey timber building will be investigated. The report also contains comparisons with higher buildings. A comparison will be done to a six-storey and an eight-storey building. Except for the height, the three different buildings are assumed to have the same design. In figure 1.1, the floor plan is presented. As shown in figure 1.1, the investigated building for this report consists of two blocks of modules, the blocks will be called structures further on in the report and when discussing the structures together it will be referred to as the building. The walls named 1-6 and A-Y in figure 1.1 act as shear walls and the horizontal loads from the wind are carried by these walls. The wind load acting on the building is accumulated from the top down to the ground. Therefore, the shear walls on the first floor will have to carry all the wind load acting on the building and will be the most exposed walls in the building. A more detailed description about the loads, and how the building will carry the loads is found in chapter 3.

A summary of the building is listed below:

- The length of the building is 57.58 m.
- The width of the building is 19.83 m.
- The different heights of buildings:
  - 4-storey building: 17.7 m.
  - 6-storey building: 23.7 m.
  - 8-storey building: 29.7 m.
- The height of the roof structure is, at maximum, 5.7 m.
- The height of every module is 3 m.
- The building is composed of two structures named structure 1 (the modules between wall 1-3) and structure 2 (the modules between wall 4-6).
- There are two set-ups of wall structures which are used the most in the building; the outer walls are made with OSB and the inner walls are made with fibre gypsum, see figure 1.2. The walls made of OSB consist of two panels, one on each side of the insulation and the walls made of fibre gypsum consist of two panels on the same side of the insulation. The panels of both materials are fastened with staples onto the timber frame.
- Only the named walls (1-6 and A-Y) in figure 1.1 resist horizontal loads, in other words, are shear walls.

A more detailed description of the building is found in appendix A and a greater illustration of the floor plan is found in figure A.15.

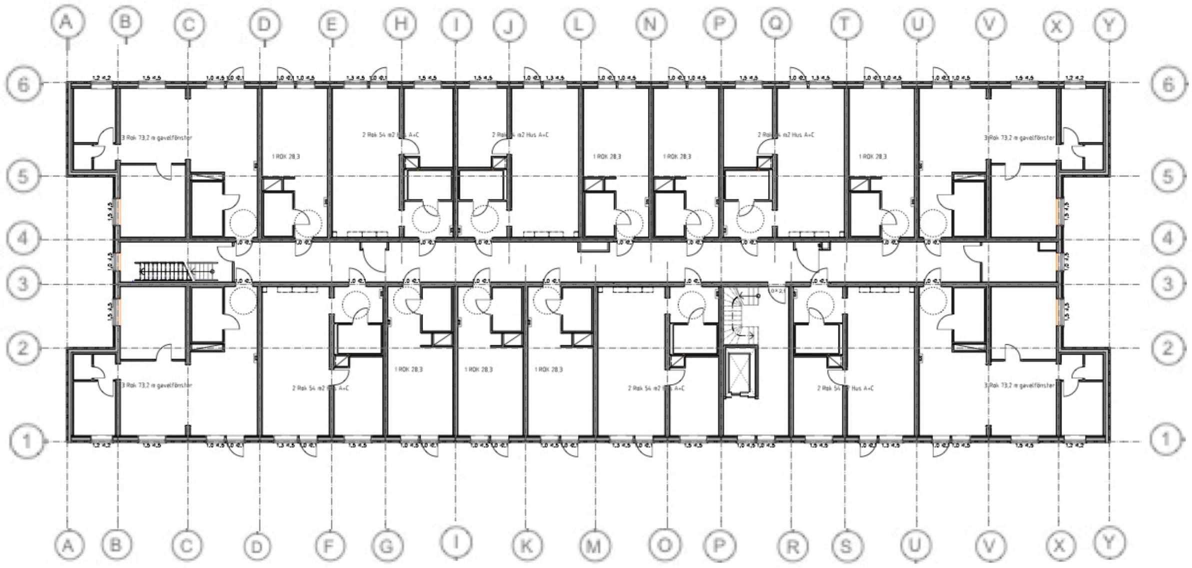
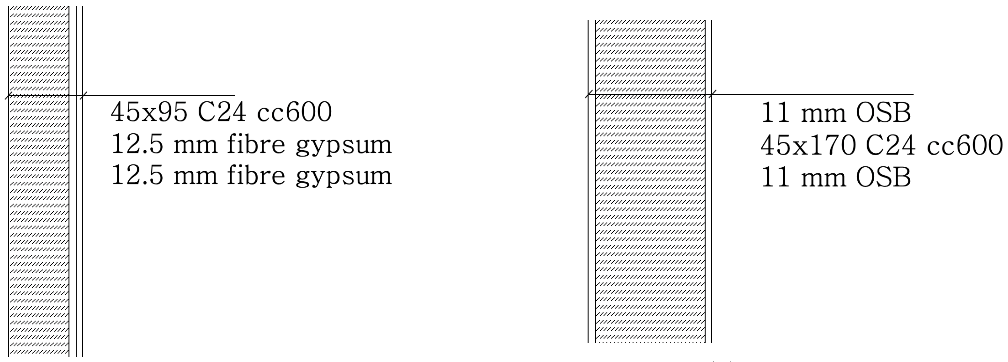


Figure 1.1: Floor plan of the investigated building.



(a) Module separating walls. *(b) Outer walls.*

Figure 1.2: Wall set-up for the two different walls.

Chapter 2

Prefabrication and Timber volume elements

2.1 History

In Sweden, the first prefabricated building, as it is known today, was built in the early years of the 1820s. Architect Fredrik Blom had an idea about movable houses and his solution was prefabrication. He built two houses for the Swedish king Karl XIV Johan, a military exercise area and the castle of Rosendal (Lidelöw et al., 2015). His idea was observed outside the Swedish borders and sold to Russia, Germany, France and even the United States of America (Bengtsson, 1990). By the end of the 19th century, the company *Fogelfors bruk* was the first company to present a series of residential houses where the building plans and descriptions were standardised (Lidelöw et al., 2015). This was the beginning of industrial prefabricated elements.

The popularity to build with prefabricated elements has shifted over the years. According to Boverket (2006), the industrial building era in Sweden can be divided into three phases. The first phase occurred after the Second World War in correlation to housing shortage. In this time Sweden started with standardised module-building and became a leading country in this matter. The decrease in loan limit which was introduced in 1955 encouraged the development of building methods with reduced costs (Boverket, 2006).

Prefabricated volume elements have been used in Sweden since the 1950s (Boverket, 2006). The municipal corporation *Bostadsbolaget* in Gothenburg was, according to Boverket (2006), the only company at this time that used modules for apartment buildings. *Bostadsbolaget* built with the "hybrid"-method, a method also known today, where a combination of plane and volume elements were used. Otherwise it was plane elements, made of concrete, that dominated the market at this time. *Bostadsbolaget* saw the potential in improved working environment and the ability for labour to work in older age, that came with a high degree of prefabrication. The first phase continued up to and including the Million program, in the 1960s and 1970s (Boverket, 2006).

The second phase began in the late 1970s and continued through 1980s. The new phase was introduced due to a greater demand for variations of building, user customisation and environmental awareness. In the 1970s new favourable tax laws for building single-family houses were introduced and the manufacturing of these houses was increased, many of them were constructed with timber. In the 1980s the share of houses built with prefabricated methods was 85% (Boverket, 2006).

The third phase started in the 1990s and is still going on today. This phase is controlled by the flexibility over the lifetime of a building, information technology and international cooperation. In the middle of the 1990s, Sweden again faced housing shortage and in combination with new requirements on fire-safety for apartment buildings, the focus shifted from the quality of the material of the construction, to the time the building could resist fire without collapse. With the previous restriction it was almost impossible two build higher than two floors with timber as a construction material, but with the new requirements it provides room for timber as a construction material and systems with prefabricated plane and volume elements were further developed (Boverket, 2006). Since then, timber as a construction material has been more popular in Sweden and guidelines about how the development for timber was supposed to take place were written. In 2004 the Swedish Parliament stated *"in 10-15 years, timber as a construction material shall be a natural alternative in all kinds of building – and eventually also in Europe"* (Platen & Nord, 2004).

2.2 Different degrees of prefabrication of multi-storey buildings with timber as a load-bearing system

The usage of timber as a construction material has a lot of advantages. As shown in table 2.1 below, timber is an appropriate choice for all types of frame systems and all types of manufacturing methods (Gustafsson et al., 2013).

Table 2.1: Different materials for framework system after Gustafsson et al. (2013).

Stabilising system material	**System**					
	Load-bearing system			Manufacturing method		
	Column-beam	Studs	Shear walls	Building at site	Prefabricated plane element	Prefabricated volume element
Steel	+	+	-	+	+	+
Concrete	+	-	+	+	+	0
Timber	+	+	+	+	+	+

Where (+) is suitable, (0) is possible and (-) is not suitable.

This also shows that timber for industrial construction is a suitable choice.

The degree of prefabrication of a multi-storey building with timber as a frame system can vary. According to Gustafsson et al. (2013), there are three different main categories that buildings of this type can be divided into:

1. Column-beam-framework system with timber floors
2. Load-bearing walls with timber floors/plane elements
3. Modules/Volume elements

The categories are numerated after the lowest degree of prefabrication, i.e. the Column-beam-framework system has the lowest degree of prefabrication. According to Lidelöw et al. (2015), they are also numerated by flexibility, where Column-beam-framework system is the most flexible type of construction. In other words, with more prefabrication less flexibility will follow.

Column-beam-frame systems can both be categorised as a closed or open system. During the 1960s and 1970s, the concept of open or closed systems were introduced. A closed system is a system where one company designs and produces the module, which cannot be combined with elements of different companies. An open system is suitable for minor producers that can construct smaller parts of a system and then combine with elements from other producers (Boverket, 2006). The technical information in an open system has to be available for everyone involved (Boverket, 2006). In reality, this is hard to achieve according to Lidelöw et al. (2015), who believes that manufacturers produce their solutions for special connectors between their elements, and therefore create a closed system even though the idea from the beginning was an open system.

The greatest advantages with a Column-beam-frame system, is its possibility to provide flexibility in design. Open floor plans are possible since the span can be wide, and the areas can be divided into smaller parts with simple (non load-bearing) walls. These walls can easily be moved if the purpose of the building changes. Since it is only the columns that are the vertical load-bearing parts of this system, more space can be included in the room layout (Gustafsson et al., 2013; Lidelöw et al., 2015). This system is suitable for offices and industrial buildings, which often require big open spaces. For residential buildings, there is always a need for walls to separate the apartments and the rooms, therefore, it is beneficial to use the walls as the load-bearing system (Gustafsson et al., 2013; Lidelöw et al., 2015).

Load-bearing walls with timber floors have a higher degree of prefabrication than the earlier mentioned column-beam frame system. It is the most common way to build residential buildings today (Lidelöw et al., 2015). There are two types of methods to build plane elements, either with wooden studs or with solid wood/cross-laminated timber. This type of system gives a high range of possibilities for the design and it is easier to control the quality for every element since the production is repeated. The limitation for this system is the transportation to the building site. The building with plane elements can either be an open or a closed system depending on the client's preferences (Gustafsson et al., 2013).

The third system is to build with volumes (also called modules). The volumes are constructed at a factory and then transported to the working site as boxes. The limitation for the dimensions of the modules are also here the transportation. Due to the weight of the modules, they are rarely constructed by concrete (Lidelöw et al., 2015) as shown in table 2.1. Timber volume elements are further described in section 2.3 below.

2.3 Timber volume elements

To assure the cost efficiency it has to be decided, already from the beginning, whether the building is going to be constructed with timber volume elements (TVE) or not. This is because the design is not that flexible and certain floor layouts and technical solutions do not fit for a TVE building (Svenskt trä, 2015).

The dimensions of the volumes are controlled by the limitations of what is allowed on

Swedish public roads since the volumes have to be transported from the factory to the building site. The width of the volumes is the critical parameter, it cannot be wider than 4.15 meters without being transported with a police escort (Gustafsson et al., 2013). The height of the modules is dependent of the type of transport. Depending on the type of truck or train there are different limitations, but the maximum height is approximately 3 meters. The length of the module is dependent on the transport but also the manufacturer. Typical maximum length is 8-13 meters (Svenskt trä, 2015).

The construction of the modules in the factory is done on an assembly line. The walls, floor and roof are constructed as plane elements separately at first. The thickness of the layers and the contents of the elements vary for the different manufactures, but a typical thickness of a wall is 300 mm according to Gustafsson et al. (2013). Depending on what type of element is produced it stops at different workstations. An example for a wall segment; the wooden studs are first connected to form a timber frame and then some type of sheet is attached. All necessary holes in the sheet are done by a machine that is linked to the CAD-drawings. Next, the wall is filled with insulation and other required layers are attached. The walls are speckled and painted and if it is a kitchen wall, all the kitchen equipment is connected and thereafter sent to the assembly of the whole volume (Persson, 2020).

When building TVE in the factory, the stage of completion should be as high as possible to reduce the time needed at the working site to assemble the volumes (Gustafsson et al., 2013). The installations for electricity, HVAC (heating, ventilation and air condition) and computer are normally done in the factory (Gustafsson et al., 2013). The bathrooms are often fabricated as small volumes outside the assembly line and then placed inside the modules. This is because there is no time for tiling and the following drying time during the assembling of the modules (Persson, 2020).

The stabilisation system in TVE is carried out by the walls and the timber floor. All the bearing elements need to work together to resist the horizontal loads and transfer them to the base of the foundation. The walls and floors are often used as diaphragm elements (Gustafsson et al., 2013), which is further described in chapter 3.

2.4 Advantages and disadvantages with timber and volume element building

The advantages of building modules in factories are many, and especially with timber as material. More produced buildings with timber as a building material can lower the climate effect (Gustafsson et al., 2013). From an environmental point of view, the following advantages have been discovered when choosing timber as a construction material instead of concrete or steel.

- A factory-made wooden residential building emits 40% less carbon dioxide than a comparable concrete building during the lifetime of the building, according to Brege et al. (2017).
- Timber from a sustainable forestry is a renewable material (Gustafsson et al., 2013).
- The energy used for the production of wooden products is relatively low and, in most cases, it is renewable energy (Gustafsson et al., 2013).

- The timber framework binds carbon dioxide during its lifetime and contains a large bio-energy asset in case of demolition (Gustafsson et al., 2013).

Timber as material is light which makes the transport easier and more effective compared to a module made of steel or concrete (Gustafsson et al., 2013). However, the transporting on Swedish public roads have certain restrictions and the modules have to be limited to these restrictions.

The work environment is improved inside a factory compared to the building site and it is easier to build with resource efficiency due to the assembly line in the factory (Gustafsson et al., 2013). Factory production also results in shorter share of assembly duration to take place at the building site and larger share in the factory, in a controlled in-door environment, which is beneficial e.g. for moisture risk during assembly.

If a short construction time is important TVE is a suitable way to build. Depending on the degree of prefabrication, the assembly on the working site can be as short as a few days (Gustafsson et al., 2013). When using standardised construction the uncertainties can be reduced which leads to improved cost efficiency and the set construction time is easier to achieve (Gustafsson et al., 2013).

As a result of the short construction time and efficiency in the factory, the flexibility concerning the layout of the apartments and the building, choice of materials, technical solutions and changes in the design after the start of the production have their limitations (Gustafsson et al., 2013).

For all construction materials, the requirements about acoustics, fire safety, moisture and tightness must be fulfilled. For timber there are today technical solutions for all of these parameters. The moisture demand is the most critical for TVE. When the modules are transported and assembled at the building site the modules can be exposed to moisture and rain and the solution to this is a good transport routine and weather protection at the building site (Gustafsson et al., 2013).

Chapter 3

Horizontal Stabilisation

A building needs the stability of the structural system to resist the acting horizontal loads. These forces are in particular wind load, sway imperfection and the result of eccentric loads (Isaksson et al., 2017), although wind is the only action considered in the present thesis. The contents in this chapter will be on how to horizontally stabilise a building. Firstly, static equilibrium, including tilting and sliding and secondly the bracing system in the construction.

3.1 Exterior phenomena

When designing the stabilisation system for horizontal loads acting on an entire building two exterior phenomena need to be accounted for, sliding and tilting of the building (Svenskt trä, 2017). Since the investigated building is carried out with volume building technology, except for analysing the risk of tilting, the risk of sliding of individual modules needs to be evaluated as well. If necessary, the anchoring force between the modules is designed.

3.1.1 Tilting of building

Tilting of a building can be a problem if the horizontal wind loads cannot be balanced by the self-weight of the building. According to Svenskt trä (2017), this can be checked by comparing the resultant of the vertical ground reaction and the core boundary of the building. The core boundary is approximately located one-sixth of the width of the building, from the centre line, see figure 3.1. If the vertical reaction is in the range of the core boundary, the building is safe from overturn (Svenskt trä, 2017). If it is not, one of the following two solutions has to be implemented: increase the self-weight of the building or anchor the ground concrete slab to the underlying ground (Svenskt trä, 2017). The self-weight has to be reduced by a factor of 0.9 since it acts as favourable load and is designed according to equation 6.10 ”Design values of actions (EQU)” in SS-EN 1990 (2004), since it is a control of static equilibrium.

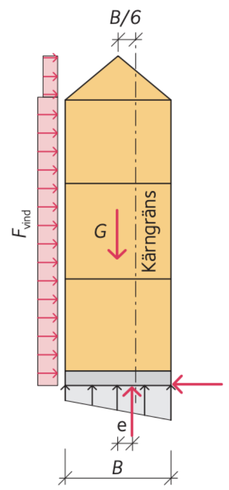


Figure 3.1: Control of overturn with the core boundary plotted. The illustration is from the CLT handbook, Svenskt trä (2015) and used with permission. (Kärngräns = Core boundary).

3.1.2 Anchoring of outermost modules

Derome has a strategy for analysing how much the modules on the gables need to be secured against the tension loads, suction on the gables, created by the wind load acting on the long sides of the building, see figure. 3.2. The tension loads created on the gables will affect the outermost modules as illustrated in figure 3.3. Anchorage should prevent sliding of single modules as well as tilting of a single vertical stack of modules.

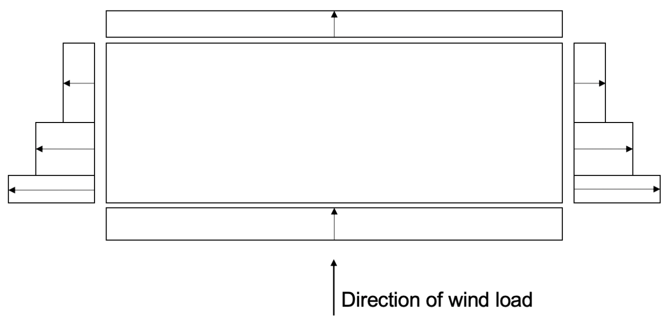


Figure 3.2: Illustration of how the wind load, acting on one of the long sides, is affecting the building. View from a above.

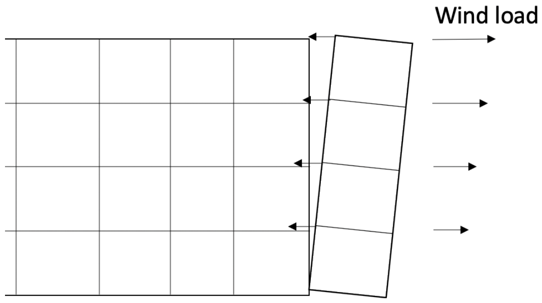


Figure 3.3: Illustration of the outermost modules. View from the long side of the building.

The modules are fastened together in the horizontal direction with tension ties (F1-F4.1 in figure 3.4). These tension ties, in addition to the self-weight (G_d) of the modules, have to carry the moment from the wind load (Q), see figure 3.4. It is assumed that every tension tie will carry the same load. This is a simplification which are used by Derome and the assumption is, therefore, implemented for this report as well. The resistance of these tension ties can be designed through a moment equilibrium. The acting wind load is the tension load created on the gables when the wind load acts on the long sides of the building. It generates a tension force distributed along the gables, and the tension ties between the outermost modules and the adjacent modules have to be designed to resist this load. The self-weight is a favourable load and is multiplied with a factor 1.0, according to EC1-1-4 (2008) equation 6.10b, since this problem is not focusing on the entire building, unlike the control for tilting. A detailed design of the required resistance of the tension ties is presented in appendix F.

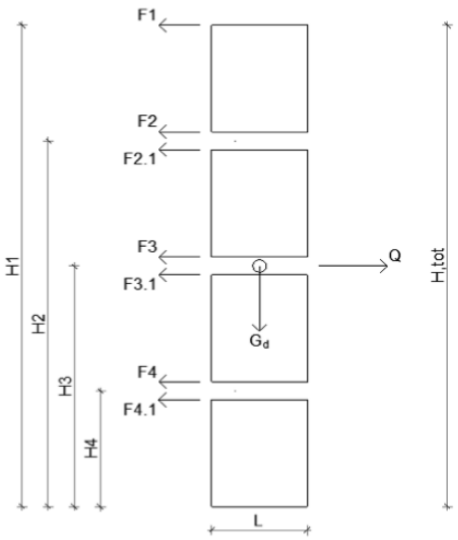


Figure 3.4: Evaluation of tilting according to Derome.

3.1.3 Anchoring of top modules against up-lifting

There is a risk for the top modules to be uplifted by the horizontal wind load when it is acting on the long sides of the building, see figure 3.5.

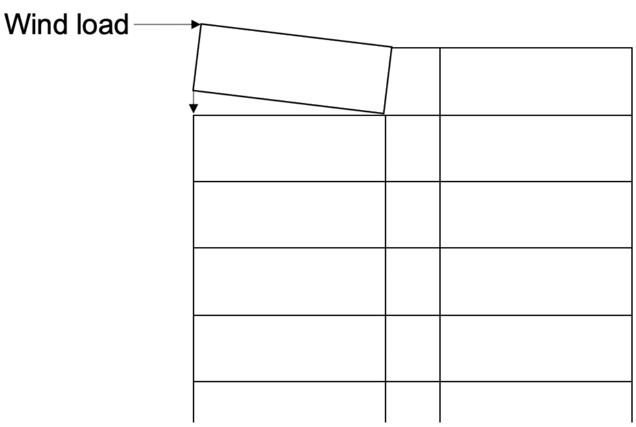


Figure 3.5: Illustration of top modules. View from the short side of the building.

In figure 3.6, one top module is illustrated, where G is the self-weight, Q is the wind load and F is the anchoring force. The anchoring is only necessary if the overturning moment from the wind load is greater than the counteracting moment from the self-weight. The design is further described in appendix F.

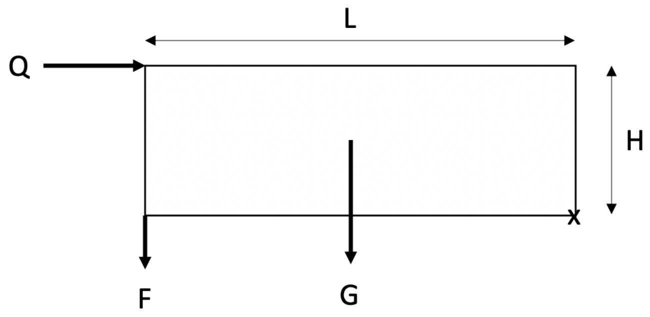


Figure 3.6: Evaluation of anchoring for top modules. View from the short side of the building.

3.1.4 Sliding

When preventing the building from sliding a control is needed making sure that the shear stress between the foundation slab and the underlying ground is less than the design stress of the ground material (Källsner & Girhammar, 2008). If this is not fulfilled, friction material can be placed under the foundation slab. Sliding will not be further investigated in this report.

3.2 Stabilising system

There are three well-known techniques to stabilise a building:

- Truss
- Framework
- Shear walls

A common way to stabilise a building with *trusses* is by using wind-braces. The braces can be located in the walls and the roof. The bracing, together with the beams and columns, will transfer the wind load to the foundation. This bracing is ordinarily made of steel due to its good tension properties (Isaksson et al., 2017).

To stabilise a building with *framework* means that the horizontal loads are resisted by a moment in the foundation or in the connection between elements in the system. For example, it can be done by a three-hinged frame/arch or a beam-column system, also called a two-hinged frame, with fixed connection in the ground (Isaksson et al., 2017).

The choice of the stabilising system generates different consequences for the building. Type of construction, the cost as well as technical solutions for the connections are parameters that change for the different systems (Isaksson et al., 2017). The usage of the building should, therefore, be closely considered before choice of stabilising system is done. It is also possible to combine the different stabilising systems, which is common in multi-storey buildings.

Since this thesis is focusing on stabilisation with shear walls a further investigation is made on this technique and two calculation methods (Method A in EC1-1-4 (2008) and an elastic method according to Carling et al (1992)) are presented for this system.

Shear walls

A shear wall or roof, also called diaphragm actions, is rigid in its own plane, but not perpendicular to the surface. To make sure that a system is stable, at least three walls, that will not intersect in one point in their elongation, and a roof diaphragm is needed. The structural systems, when building with shear walls, are often lightweight and made of steel or timber and combined with sheets made of timber or gypsum (Isaksson et al., 2017). Today the gypsum board is common in residential buildings due to its good acoustic and fire safety properties. A shear wall does usually contain several panels where every panel consists of a sheet fixed to a timber/steel frame. It can be sheets on one or both sides of the frame.

Wind pressure is distributed to the diaphragms through the exterior walls. The resulting force is then transferred horizontally to the shear walls where it is transferred vertically to the ground (Källsner & Girhammar, 2008). In figure 3.7 below, the case for a one-storey building is presented. The dotted lines represent the deformation and for this case the front studs in the sheathed timber frame walls are assumed to be fully anchored. In volume building technology, the modules can be assumed to carry the wind load acting on every module respectively, since no greater connection between the modules are usually implemented.

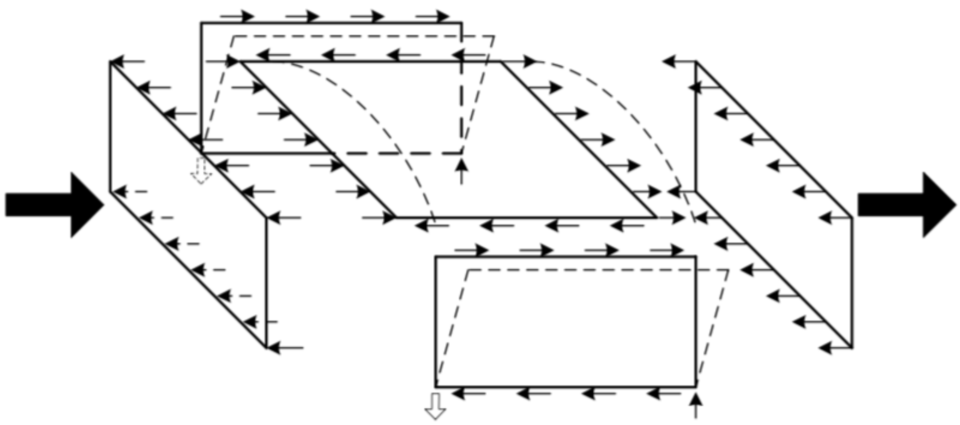


Figure 3.7: How the horizontal loads are carried by the walls and the floor structures to the ground (Källsner & Girhammar, 2008). Used with permission.

Figure 3.8 shows examples of how the floors can be horizontally connected to the walls to carry the load down to the foundation, either with a supported floor or with suspended floor.

For the case where the floor is supported (left) the connection between the floor and the wall above needs to be designed in shear for the load corresponding to the horizontal load acting on the upper wall. The connection for the other case, where the floor is suspended (right), needs to be designed for the horizontal load which the floor structure transfers to the underlying wall (Svenskt trä, 2017). For the investigated building the connection called supported floor is used.

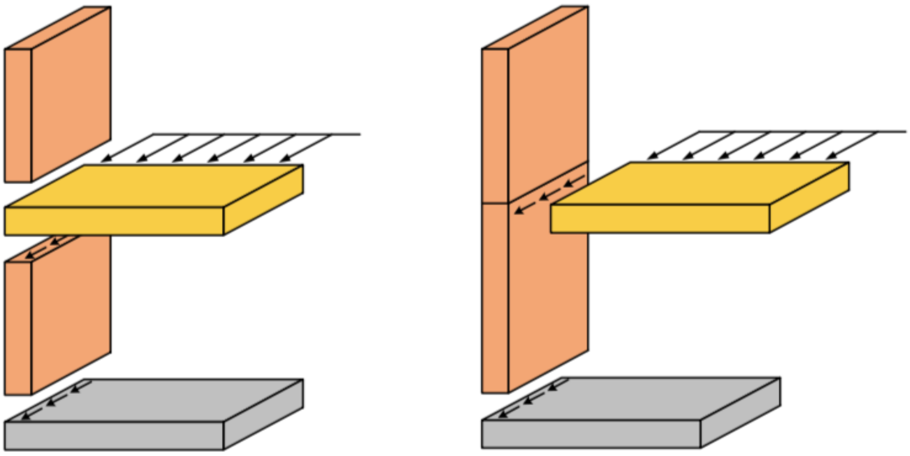


Figure 3.8: How the horizontal loads are carried from the floor to the walls with either supported floor (left) or suspended floor (right) (Källsner & Girhammar, 2008). Used with permission.

If the wind load is assumed to be constant over the height of the building, the floor structures are exposed to the same amount of load. An exception is the top and bottom floor structure that will carry less load because of the area of influence for these loads, see load H_3 and H_0 in figure 3.9. The walls, on the other hand, are accumulating the loads over the height of the building which will lead to higher loads further down in the building, see left building in figure 3.9 where the lowest wall will have to carry the loads H_1, H_2 and H_3 (Svenskt trä, 2017).

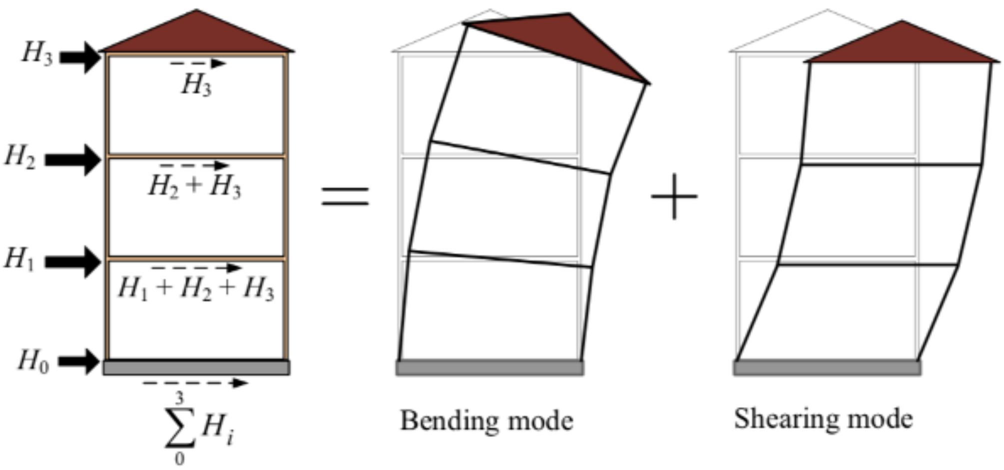


Figure 3.9: The distribution of the horizontal loads and bending and shear deformation for the floor levels of the building (Källsner & Girhammar, 2008). Used with permission.

It can also be seen in figure 3.9 that the walls need to resist both bending and shear forces. Depending on material and design, the distribution of the two modes are different. For wall panels of the sheathed timber frame wall type, which are weak in shear, the shearing mode is completely dominant. For rigid shear walls, on the other hand, the bending mode is dominant. This could for example be the case for walls made of concrete (Källsner & Girhammar, 2008).

Except for the shear force acting in the walls, there will also be a lifting force on one side of the wall and a pressure force on the other side, due to the wind load. This is shown in figure 3.7 where the white arrow is an anchoring force at the leading stud and the black arrow is a compressing force at the trailing stud (Källsner & Girhammar, 2008).

3.2.1 Elastic or plastic design method

There are two known calculation models to use for the design of shear walls, the elastic and plastic design method. In Carling et al (1992) following assumptions are listed describing what the *elastic method* is based upon.

- The timber framework consists of rigid elements that are connected with joints and anchored in the layer beneath.
- The sheets are completely stiff and prevented from buckling. The sheets are fastened in a way that they do not touch each other or the surrounding construction.
- The deformations are minor comparing to the height and width of the wall elements.
- The fasteners have linear elastic properties and are not dependent on the direction of the offset.

For this method, it is also assumed that the wall will be divided into wall fragments. All these fragments contain one sheet that is the same height as one storey and the timber framework associated with this board (Carling et al, 1992). The capacity of the wall for the elastic method is dependent on the capacity of the most loaded joint between the board and the timber framework (Källsner & Girhammar, 2008). As shown in figure 3.10 below, the horizontal force from the floor structure acts in the upper part of the wall and the maximum reaction forces are located in the corner of the panel.

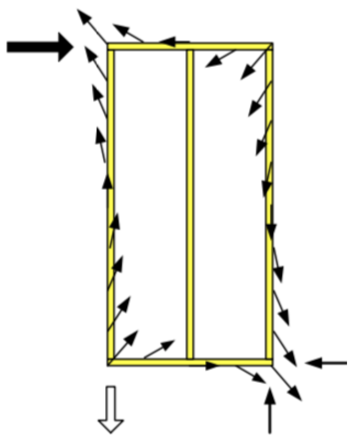


Figure 3.10: Force displacement for elastic calculation model (Källsner & Girhammar, 2008). Used with permission.

Källsner and Girhammar (2008) also implies that the method is based on the assumption that none of the timber elements are exposed to bending. According to Källsner and Girhammar (2008) the connection between the leading stud and the layer beneath can either be fastened or, if the self-weight of the building is high enough, the weight can resist the uplifting forces created by the horizontal loads.

Equation 3.1 design the capacity of the shear walls according to Carling et al (1992). The capacity of the shear wall, H_{Rd}, is dependent on the design shear capacity of the fasteners, F_{vd}.

$$H_{Rd} = \frac{F_{vd}}{h\sqrt{(\frac{x_{max}}{\sum x_i^2})^2 + (\frac{y_{max}}{\sum y_i^2})^2}} \tag{3.1}$$

Where $x_{max} = w/2$ and $y_{max} = h/2$ according to figure 3.11 below.

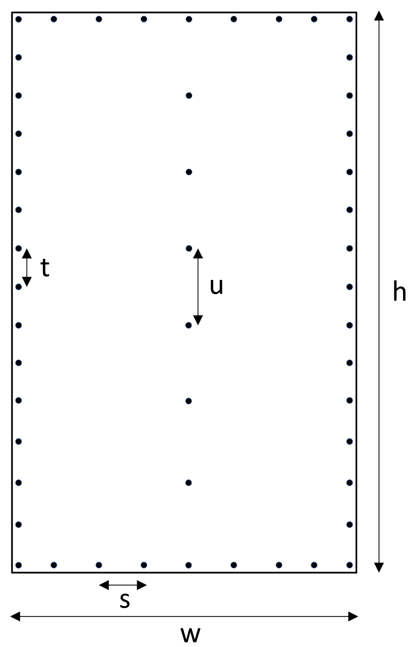


Figure 3.11: The dimensions of a panel.

Approximate expressions for $\sum x_i^2$ and $\sum y_i^2$ are taken from Carling et al (1992) and for a panel like the one in figure 3.11, the expressions are:

$$\sum x_i^2 = \frac{w^2}{12}(2n + 6m) \tag{3.2}$$

$$\sum y_i^2 = \frac{h^2}{12}(6n + 2m + p - 3) \tag{3.3}$$

Where:

$$n = \frac{w}{s} \tag{3.4}$$

$$m = \frac{h}{t} \tag{3.5}$$

$$p = \frac{h}{u} \tag{3.6}$$

For a panel without the stud in the middle, the expressions are:

$$\sum x_i^2 = \frac{w^2}{12}(2n + 6m) \tag{3.7}$$

$$\sum y_i^2 = \frac{h^2}{12}(6n + 2m) \tag{3.8}$$

The difference between the two methods is the flexibility for the designer in the choice of force flow in the construction *plastic method*. Similar to the elastic method, the force can be carried by the leading stud to the foundation, called full anchoring according to Källsner and Girhammar (2008). This implies that the framework is restrained from moving vertically relative to the ground. By anchoring the bottom rail, the joint between the sheet and the framework can carry the bending load to the underlying structure. This is called incomplete or partial anchoring, see figure 3.12 (Källsner & Girhammar, 2008).

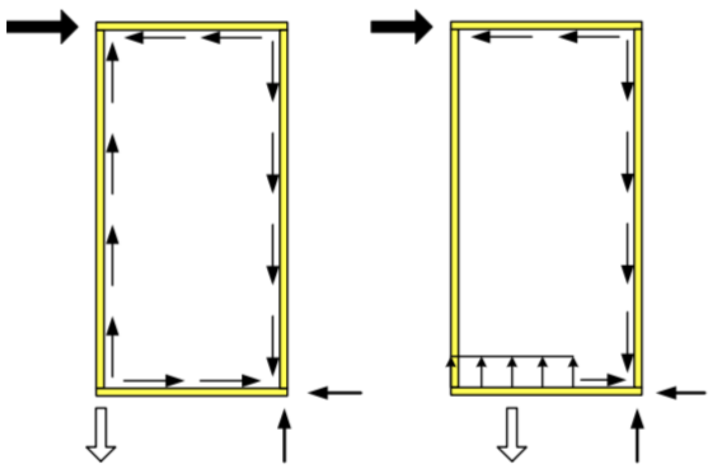


Figure 3.12: Force displacement for plastic calculation model. Complete anchoring of leading stud (left) and incomplete anchoring of leading stud but anchoring of bottom rail (right) (Källsner & Girhammar, 2008). Used with permission.

For incomplete anchoring in the plastic method, it is accepted for the vertical studs to displace relative the bottom rail, but the rail has to be fastened to the ground.

According to Källsner and Girhammar (2008), the plastic method should be used for design in the ultimate limit state but both Näslund (2012) and Bodén (2009) claim that the difference between the elastic and plastic method is not that big in the end. In the present study, Method A in EC1-1-4 (2008), which is a variant of the plastic method, will be compared to the elastic method according to Carling et al (1992).

3.2.2 Shear wall design according to Eurocode

In EC5-1-1 (2009) two methods for calculations of shear walls are presented, Method A and Method B. The difference between the two methods is the assumed boundary conditions. For Method A, the leading stud (the stud closest to the loaded side of the wall) is assumed to be fully anchored. A pure shear flow will occur in the wall and plasticity is reached in the fasteners along the perimeter at approximately the same time. For Method B, the bottom rail is assumed to only be partly anchored and the failure mode will be different (Vessby, 2011). Both methods are based on theory of plasticity but since Method A is recommended in Eurocode, and this is also the method that Derome has chosen to use, only this method will be presented and further discussed.

The design load-carrying capacity, $F_{v,Rd}$, for a wall made up of several panels loaded with a force, $F_{v,Ed}$, acting at the top of a cantilevered panel secured against uplift, is the sum of the capacity of the individual panels, see equation 3.9.

$$F_{v,Rd} = \sum F_{i,v,Rd} \tag{3.9}$$

One panel consists of a sheet fixed to a timber frame, either on one or both sides. The design load capacity for every panel can be calculated as:

$$F_{i,v,Rd} = \frac{F_{f,Rd} \cdot b_i \cdot c_i}{s} \tag{3.10}$$

Where:

$F_{f,Rd}$ is the lateral design capacity for a single fastener;
b_i is the wall panel width;
s is the spacing of fasteners.

and

$$c_i = \begin{cases} 1 & \text{for } b_i \geq b_0 \\ \frac{b_i}{b_0} & \text{for } b_i < b_0 \end{cases} \tag{3.11}$$

Where:

b_0 =h/2;
h is the height of the wall.

The lateral load carrying capacity for the fasteners along the edges of an individual panel can be increased with a factor of 1.2 because of the interaction between all the fasteners. This method is only applicable if the spacing between the fasteners are the same along the perimeter of every panel and if the width of the sheet is more than $h/4$.

The panels that are on top or below an opening, such as a window or a door, are not allowed to be taken into account for the load-carrying capacity, see figure 3.13 segment (2).

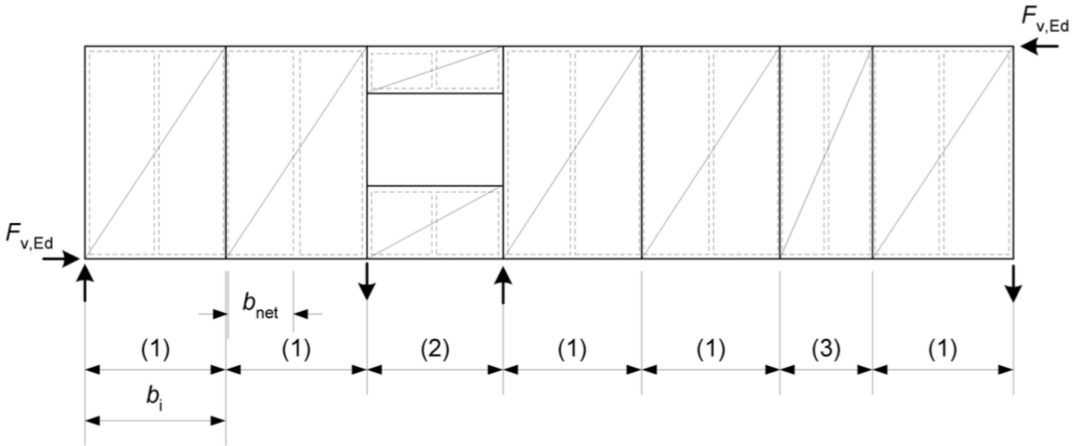


Figure 3.13: Wall segment consisting of several panels (EC5-1-1, 2009).

In figure 3.13 it is also shown a panel that is not wide enough to be part of the load-carrying capacity, segment (3). An exception to this, according to Derome, is the inner walls where they glue the fibre boards together in the interface of the two boards. For these walls, the individual panels are not of interest, only the total length of the walls. More about this in appendix A.

If the wall consists of sheets on both sides, sheets of the same type and with the same dimensions, the sum of both sides is taken as the capacity. To make sure that the centre stud in a panel can be considered as a support to the sheet the spacing of the fasteners in the centre stud cannot be greater than twice the spacing of the fasteners along the perimeter of the sheet.

Shear buckling of the panels can be disregarded if $\frac{b_{net}}{t} \leq 100$.

Where:

b_{net}	is the clear distance between studs;
t	is the thickness of a sheet.

Chapter 4

Method for design of stability in this study

The ratio between the horizontal loads and the resistance of the shear walls will be analysed. Firstly, a four-storey building is going to be investigated and thereafter an analysis of how higher buildings will change the ratio between load and capacity, will follow in the investigation. If the resistance of the higher buildings is insufficient some suggestions and reasoning about how to stabilise those buildings will be included.

The capacity of the walls will be calculated with two different methods to enable a comparison of the results from the methods. The first method is called Method A in Eurocode (EC5-1-1, 2009) and the second is an elastic design method that is described in Carling et al (1992). Derome is using Method A today and the differences in the outcomes of the methods are of interest. Since Method A is a type of plastic design method an elastic method was chosen as a comparison to be able to analyse the differences.

4.1 Wind load

The only horizontal load which is considered is wind load and it is calculated according to EC1-1-4 (2008) and EKS 11 (2019). The key values of the design are presented in the section below and a more detailed design is presented in appendix B. When calculating the wind load acting on the building, the location of the building is assumed to be the west coast of Sweden, with the reference velocity of $v_b = 25$ m/s and the terrain category number II.

The designed values for the wind pressure according to EC1-1-4 (2008) and EKS 11 (2019) are compared to values taken from table 2.7 in Isaksson et al. (2017). The detailed results of the comparison are found in appendix B, but it shows that there is no significant difference between the methods and the values obtained with EC1-1-4 (2008) and EKS 11 (2019) will further on be used.

The design wind load can thereafter be obtained and depending on what side of the building (the long sides or the gables) the wind load is acting; the walls act differently. If the wind load acts on the short side of the building, the walls in the longitudinal direction of the building will have to carry both the pressure and tension loads that will be created, as shown in figure 4.1. The accumulated total wind load per meter in the horizontal direction which the shear walls in the longitudinal direction of the building will have to carry are presented in table 4.1, where the bold values are the total load.

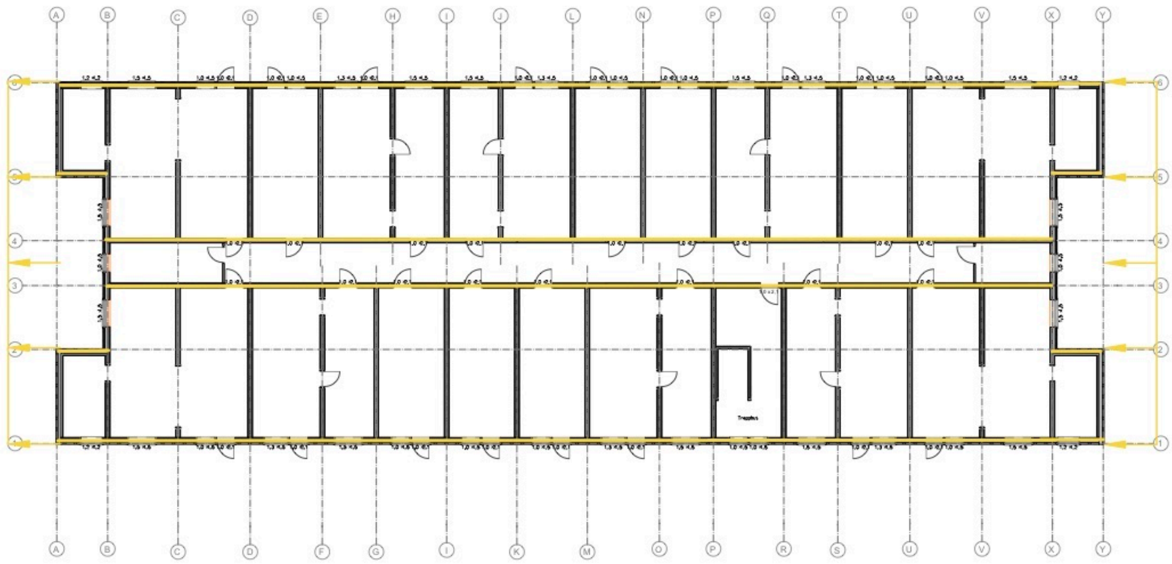
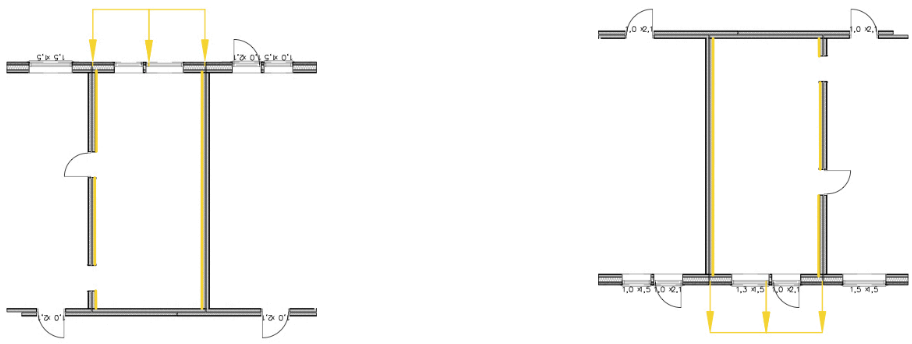


Figure 4.1: Stabilising walls if wind load acts on the short side of the building.

Table 4.1: Design wind loads when acting on the short side of the different buildings. Load per meter in the horizontal plane.

	$\mathbf{H}_{d,pressure}$**(kN/m)**	$\mathbf{H}_{d,tension}$**(kN/m)**	**Total load (kN/m)**
4-storey building	10.4	4.6	**15.0**
6-storey building	16.9	8.1	**25.0**
8-storey building	24.2	12.2	**36.4**

If the wind load acts on the long side of the building, the walls will only have to carry either the pressure or tension loads since the two combined structures do not work together and the wind load cannot be transferred from one structure to the other, see figure 4.2. This is because the two structures are two separate buildings but with the same facade and the connections between the structures are not designed to carry the vertical load between the structures. The wind load acting on the roof will for some parts be an up-lifting load and for some parts a pressure load, this load is acting perpendicular to the inclined roof. According to EC1-1-4 (2008), four different cases of the distribution of the wind load on the roof have to be tested to find the most adverse case. For the design of shear walls the decisive case is when there is a pressure load on the windward side of the roof and an up-lifting load on the roof of the structure on the leeward side. The horizontal vectors of these loads are added to the wind load acting on the outer walls. The vertical vectors are further analysed in section 4.1.1. The design of the wind load on the roof is in detail described in appendix B. The accumulated wind load, from the roof top to 1.5 meters above the ground are designed and presented in table 4.2 below. The wind load acting from the ground to 1.5 m above the ground are directly carried by the foundation and not further investigated.



(a) Wind on windward side, pressure load. *(b) Wind on leeward side, tension load.*

Figure 4.2: How the modules in the two structures carry the wind load when the wind load acts on one of the long sides of the building.

As shown in table 4.2, the pressure force of the wind load is always the most unfavourable load between the pressure and the tension loads. Because of the asymmetry of the roof, different design loads are found for the two long sides of the building. The values in bold are the design loads for the walls in the two structures respectively, for the three different heights of the building.

Table 4.2: Design wind loads for the walls in the transverse direction of the building when the wind load is acting on wall 1 and wall 6, respectively. Load per meter in the horizontal plane.

Storeys	Wind direction	Structure	$H_{d,pressure}$(kN/m)	$H_{d,tension}$(kN/m)
4	Wind at wall 1	1	**13.0**	7.8
	Wind at wall 6	2	**10.2**	11.6
6	Wind at wall 1	1	**20.6**	13.0
	Wind at wall 6	2	**17.2**	17.2
8	Wind at wall 1	1	**28.7**	18.5
	Wind at wall 6	2	**24.7**	23.0

4.1.1 Up-lifting loads on roof structure

An analysis to make sure the self-weight of the modules is greater than the up-lifting loads on the roof created by the wind load, is performed. As mentioned above, four cases of the distribution of the wind load on the roof have to be analysed to receive the most adverse case. For this analysis, the most adverse case is when both sides of the roof have an up-lifting load. The up-lifting load for when the wind load is acting on the long sides of the building is compared to the up-lifting load that is created when the wind load is acting on the gables. From the figures C.1-C.2 showing the values and locations for the pressure coefficients when the wind load is acting on the long side of the building, and figures B.4-B.5 showing the values and locations for the pressure coefficients when the wind load is acting on the gable, the conclusion that greater up-lifting forces will be distributed along the roof when the wind load is acting on the gables can be drawn. Therefore, only this case is further analysed. As shown in table C.1 and in figure C.2, the greatest up-lifting forces are located in the corner of the building closest to the gable where the wind load is acting. The lengths of this area (e/4 and e/2 in figure 4.3) reveals that only the three outermost modules will be affected by the greatest up-lifting load (see figure 4.3) and only these three modules will, consequently, be analysed.

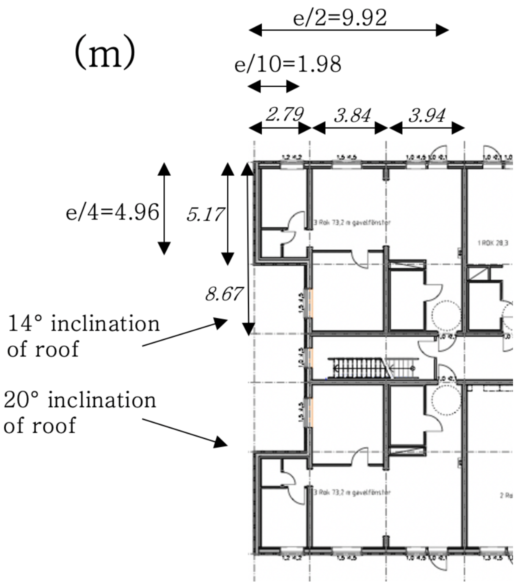


Figure 4.3: The three outermost modules which are analysed for uplifting forces.

The design of the up-lifting forces and the self-weights of the top modules are in more detail described in appendix C. In tables 4.3 and 4.4, the results for an eight-storey building are presented both for structure 1, where the inclination of the roof is 20°, and for structure 2 where the inclination is 14°. The fourth column in the tables present the differences between the forces. A positive number indicates that the self-weight is greater than the up-lifting load. In the fifth column, the utilisation of the self-weight is presented for an eight-storey building. A value lower than 1 indicates that there is no risk for up-lift of the top modules. As shown in the tables, the highest utilisation can be found in the module between wall A/B in both structure 1 and 2. If a module has a high degree of utilisation, this module is the first to become critical if there is a change in higher wind load, lower self-weight or an increase of the height of the module.

Table 4.3: Comparison of up-lifting force and self-weight for modules in structure 1 (roof inclination is 20°).

Structure 1 (20°)	**Up-lifting force (kN)**	**Self-weight (kN)**	**Difference (kN)**	**Ratio**
Module C/D	-45.87	110.30	64.43	0.42
Module B/C	-44.71	78.58	33.87	0.57
Module A/B	-27.92	33.51	5.59	0.83

Table 4.4: Comparison of up-lifting force and self-weight for modules in structure 2 (roof inclination is 14°).

Structure 2 (14°)	**Up-lifting force (kN)**	**Self-weight (kN)**	**Difference**	**Ratio**
Module C/D	-33.83	110.30	76.47	0.31
Module B/C	-32.97	78.58	45.61	0.42
Module A/B	-31.09	33.51	2.42	0.93

An example of how the values in table 4.3 are calculated, is presented below. The example is for the module between wall C and D in structure 1 where the inclination of the roof is 20° for an eight-storey building. The values for the pressure coefficients $c_{pe,10}$ for the different zones of the roof are found in table C.1.

$$U_d = (c_{pe,10,H,20} \cdot q_p) \cdot cos(\phi) \cdot 1.5 \cdot w \cdot L$$
$$= (-0.67 \cdot 1.12) \cdot cos(20) \cdot 1.5 \cdot 3.94 \cdot (8.668 + 2.438) = -45.87\ kN$$

Where the length of the distributed load is assumed to be $(8.668 + 2.438)$, i.e. the length of the module together with the width of the corridor between the two structures (from wall 1 to wall 4). This is a conservative assumption since some of the loads may be taken by the other structure. $q_p = 1.12\ kN/m^2$ according to table B.2 in appendix B.

The self-weight of the module is:

$$G_d = 11030 \cdot 1.0 \cdot 10 \cdot 10^{-3} = 110.30\ kN$$

The weight of all the modules can be found in table A.1. The weight of the roof is not taken into account but should reduce the ratio in tables 4.3 and 4.4 if it was included. Since the utilisation is less than 1 for all modules, no risk for any up-lift of the top modules are anticipated, although, anchoring of the roof structure is probably necessary and should be further researched. The same design is done for all three modules in the two different structures. Since this is the results for an eight-storey building, the design for a six- or four-storey building is not necessary because the wind load will be even lower for these buildings.

4.2 Design of capacity of shear walls

Two methods are used in the analysis, Method A from EC5-1-1 (2009) and an elastic method according to Carling et al (1992). The parameters necessary for the two methods are presented below.

In figure 4.4, the different set-ups of materials in the walls are indicated. The outer walls in the longitudinal direction of the building are constructed with two OSB (marked orange in figure 4.4, that include walls number 1, 2, 5 and 6. The outer walls in the transverse direction of the building (marked pink in figure 4.4) are constructed with one OSB and two fibre gypsum boards, which include parts of walls A, B, X and Y. The rest of the walls are constructed with two fibre gypsum boards (marked green in figure 4.4). Two fibre boards are always located next to each other and are counted as one thick fibre board in the design. An exception is the inner walls adjacent to the corridor, walls 3 and 4. They are constructed with four fibre gypsum boards, two on each side of the insulation to enclose the walls.

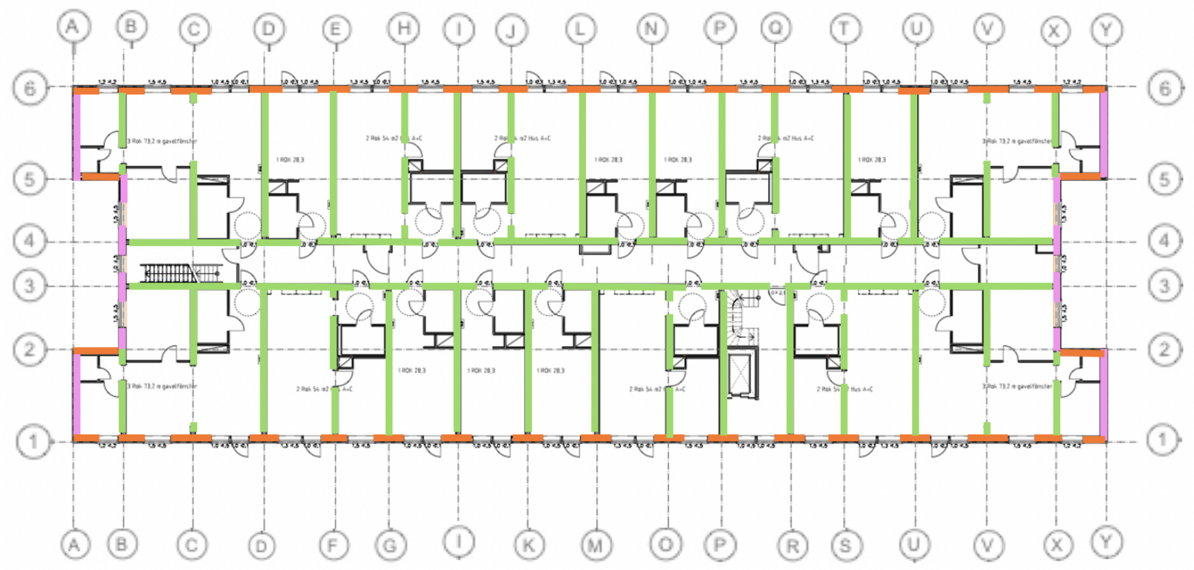
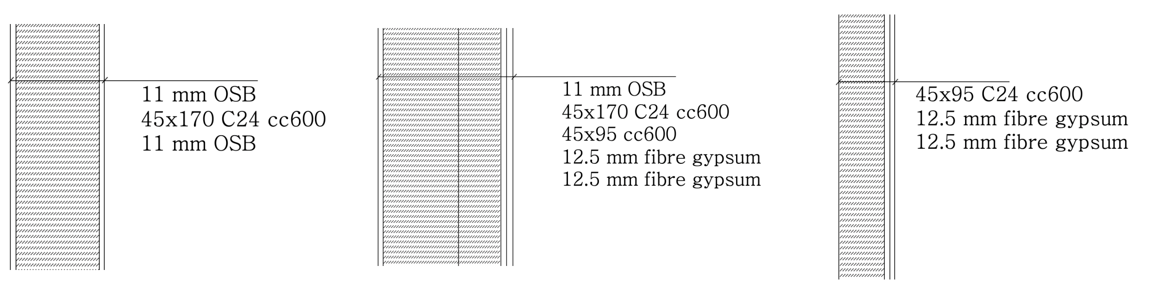


Figure 4.4: The materials of the different walls. Orange = OSB, Pink = OSB and Fibre gypsum, Green = Fibre gypsum.

The set-ups of of walls for the different locations in the building are presented in table 4.5 and in figure 4.5, please note that only the load-bearing elements are presented. For the outer walls, the insulation is located between the boards. For the inner walls between the modules, the two fibre gypsum boards are placed next to each other on the inner side of the insulation. On the outer side of the insulation the next module will be connected, and no other board will be needed to keep the insulation in place.

Table 4.5: The set-up of the different walls (only load-bearing elements included).

Outer walls, longitudinal	**Outer walls, transverse**	**Inner walls**
11 mm OSB	11 mm OSB	12.5 mm fibre gypsum
45x170 C24 cc 600	45x170 C24 cc 600	12.5 mm fibre gypsum
11 mm OSB	45x95 cc 600	45x95 C24 cc 600
	12.5 mm fibre gypsum	
	12.5 mm fibre gypsum	



(a) Outer walls, longitudinal. *(b) Outer walls, transverse.* *(c) Inner walls.*

Figure 4.5: Wall set-ups.

Both the OSB and the fibre gypsum are fastened with staples. The same type of staples are used for both materials but the shear capacity of the staples differ depending on which material is used. The shear capacities for the different materials are presented in table 4.6 together with the distances of the outer and inner staples of the sheets respectively, according to figure 3.11.

Table 4.6: Shear capacity of the staples for the different boards (Seldert, 2020).

	Denotation	OSB	Fibre gypsum
Shear capacity (kN)	$F_{f,Rd}$	0.555	0.517
Distance of outer staples (m)	s	0.08	0.08
Distance of inner staples (m)	u	0.16	0.16

4.2.1 Design according to Method A

For the design according to Method A in EC5-1-1 (2009), all the lengths of the shear walls were measured in drawings and segments less than 0.75 meters were neglected. Since the dimensions of the OSB is $2400x3000\ mm^2$ and since notice should be taken to the numbers of panels when designing the resistance of the OSB, it is assumed that all the segments less than 2400 mm only consist of one panel. For the segments wider than 2400 mm the numbers of whole panels are firstly considered, and the residual segment is controlled to be greater than 0.75 meters. If it is not, the segment is neglected.

When designing the resistance of the fibre gypsum boards, the numbers of whole panels are not of interest since they are glued together in the joint between the boards and the forces can be carried as if it was one large panel (Alipour, 2020).

The design of the resistance is done with the equations 3.9-3.11 from chapter 3. All the results can be found in appendix D. In table 4.7 below, a short selection of the results for wall 1 is presented where the lengths of the segments are noted in the first row. Segments that are shorter than 0.75 meters are underlined and written in italic to indicate the segments which are neglected.

Table 4.7: Capacity for some segments of wall 1

Wall 1	**A/B**		**B/C**		**C/D**	
b_i (m)	1.08	<u>*0.28*</u>	1.11	1.23	1.01	0.83
c_i	0.72		0.74	0.82	0.67	0.55
Capacity per segment (kN)	6.47	0.00	6.89	8.33	5.62	3.80
Capacity per module (kN)	6.47		15.22		9.41	
TOT CAP (kN)	**250.28**					

An example of how the capacity in the first module of wall 1 is designed is presented below. There are two segments of this part of the wall, the lengths of the segments are 1.08 m and 0.28 m respectively, with an opening for the door in between. The segment of 0.28 m is less than $h/4 = 0.75$ m and this segment is therefore neglected in further calculations The effective width for this section is then: $b_i = 1.08$ m. The factor c_i is calculated according to equation 3.11 and for this case where $b_i = 1.08 < b_0 = 1.5$, $c_i = \frac{b_i}{b_0} = \frac{1.08}{1.5} = 0.72$.

Thereafter, the capacity for this segment is calculated according to equation 3.10. The material in this part of the wall is OSB and the design shear capacity of a single fastener is $F_{f,Rd} = 0.555$ kN. Factor 1.2 is used according to EC5-1-1 (2009).

$$F_{i,v,Rd} = \frac{F_{f,Rd} \cdot b_i \cdot c_i \cdot 1.2}{s} = \frac{0.555 \cdot 1.08 \cdot 0.72 \cdot 1.2}{0.08} = 6.47\ kN$$

Firstly, the capacity for every segment is calculated and then added together to get the capacity for every module. To get the total capacity of wall 1, all the capacities per module are summed and multiplied with two, since there are two boards of OSB in wall 1. The same calculations are done for all the shear walls in the building, walls 1-6 and A-Y.

The risk for shear buckling of the sheets is analysed by the following statement. The statement has to be true to neglect the risk for shear buckling.

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

Where:

b_{net} is the clear distance between studs;
t is the thickness of a sheet.

The thickness of one OSB is 11 mm and the thickness for the fibre gypsum boards are $2 \cdot 12.5 = 25$ mm since there are always two thinner fibre boards next to each other. The distance between the studs is 600 mm for both materials. The analysis show that the statement is fulfilled for both materials.

4.2.2 Design according to elastic method

The design according to the elastic method was done for two different walls in the building, one in the longitudinal direction (wall 1) and one in the transverse direction (wall B). These two walls were chosen in order to have walls with both materials in the analysis and because of an indication showed that the capacity for wall B was not enough for the higher buildings when designing according to Method A and a comparison between the two methods would be interesting for this wall. The design was done according to equations 3.1-3.8 in chapter 3.

Since the building is made of modules, the panels used in the walls need to be cropped for all the openings and the width of the modules, therefore, the whole length of a panel of OSB (2.4 m) is not used in many places. It is assumed that all the segments less than 2.4 meters consist of one shortened panel. For example; in table 4.8 the width of the first segment is: $w = 1.08$ m.

The same selection of wall 1, as presented for Method A, is presented for the elastic method in table 4.8, to be able to compare the results. Since no indication was found to neglect small segments in Carling et al (1992), all segments are assumed to contribute to the capacity of the walls.

Table 4.8: Capacity of wall 1 according to the elastic method.

Wall 1	**A/B**		**B/C**		**C/D**	
L (m)	1.08	0.28	1.11	1.23	1.01	0.83
n	13.50	3.50	13.93	15.31	12.58	10.34
$\sum x^2$	24.49	1.52	26.17	31.97	21.10	14.00
$\sum y^2$	128.81	72.00	130.75	136.97	124.65	114.58
H_{Rd} (kN)	7.42	1.95	7.65	8.38	6.93	5.73
TOT CAP (kN)	**371.64**					

An example of the design is shown below for the first segment in wall 1, where the width of the segment is $w = 1.08$ m.

To design the values for $\sum x^2$ and $\sum y^2$ (equations 3.2 and 3.3), the values for n (number of fasteners at top and bottom edge of panel), m (number of fasteners at vertical edges) and p (number of fasteners at centre stud) are required (equations 3.4-3.6).

$$n = \frac{w}{s} = \frac{1.08}{0.08} = 13.50$$

$$m = \frac{h}{t} = \frac{3}{0.08} = 37.5$$

$$p = \frac{h}{u} = \frac{3}{0.16} = 18.75$$

The values for $\sum x^2$ and $\sum y^2$ can then be calculated as:

$$\sum x_i^2 = \frac{w^2}{12}(2n + 6m) = \frac{1.08^2}{12}(2 \cdot 13.50 + 6 \cdot 37.5) = 24.49$$

$$\sum y_i^2 = \frac{h^2}{12}(6n + 2m + p - 3) = \frac{3^2}{12}(6 \cdot 13.50 + 2 \cdot 37.50 + 18.75 - 3) = 128.81$$

The capacity for this segment is then:

$$H_{Rd} = \frac{F_{vd}}{h\sqrt{(\frac{x_{max}}{\sum x_i^2})^2 + (\frac{y_{max}}{\sum y_i^2})^2}} = \frac{0.555}{3\sqrt{(\frac{1.08/2}{24.49})^2 + (\frac{3/2}{128.81})^2}} = 7.42\ kN$$

Where $x_{max} = w/2$ and $y_{max} = h/2$.

The values for H_{Rd} in table 4.8 are the capacities of every segment in wall 1. These values are summed and multiplied by two since there are two boards of OSB in wall 1 to get the total capacity according to the elastic method, $H_{Rd,tot} = 371.64\ kN$.

The same design is done for wall B. The differences for wall B are that there are two different materials in this wall (both OSB and fibre gypsum) and some of the segments are greater than a whole panel. All the detailed results can be found in appendix E.

4.3 Comparison of load and resistance

The designed wind loads are compared to the capacities of the shear walls and the results can be found in chapter 5. The approach on how the wind load is assumed to be carried by the walls in the modules will be presented below.

When the wind load acts on the short side of the building, the width of influence of the load is split equally over the width of the building. Walls 2 and 5 are too small in this context and are neglected as shear walls, i.e. wall 1, 3, 4 and 6 resist the wind loads in the longitudinal direction of the building.

When the wind load is acting on the long side of the building the area of influence is the width of the modules. Each module is assumed to resist the wind load acting on that module. Further, the load on each module is distributed to the walls depending on the wall stiffness. Therefore, a percentage division depending on the stiffness of the walls is implemented. An example is shown in figure 4.6, where the module between walls C and D in structure 1 is illustrated. The wind load is acting on wall 1, which is one of the long sides of the building but the short side of the module. The percentage division is based on the capacities calculated according to Method A and the capacities are assumed to be equivalent to the stiffness of the walls. When the wind load is distributed according to the stiffness of the walls, torsion of the module should be analysed. This is neglected in this report but should be included for further research.

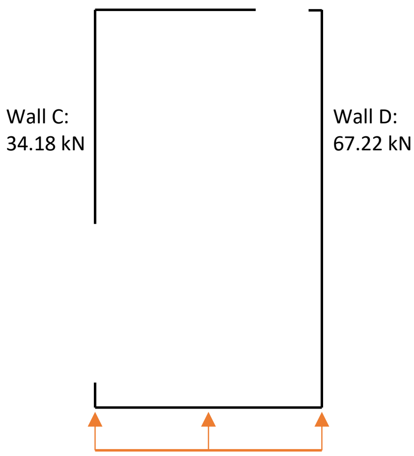


Figure 4.6: Example of how the wind load is divided according to the capacity of the walls.

As shown in figure 4.6, wall D can resist more load than wall C, and should by the theory presented, carry more load then wall C. The width of the module is 3.94 meters, the wind load is 13 kN/m and the distribution of the wind load in the different walls are then designed as:

$$H_{d,wallC} = 13 \cdot 3.94(\frac{34.18}{34.18 + 67.22}) = 13 \cdot 3.94 \cdot 0.34 = 17.25\ kN$$

$$H_{d,wallD} = 13 \cdot 3.94(\frac{67.22}{34.18 + 67.22}) = 13 \cdot 3.94 \cdot 0.66 = 33.93\ kN$$

4.4 Tilting and anchoring

As described in chapter 3.1, three problems of both tilting of the building and anchoring of modules need to be investigated. The methods for the different problems are presented below.

4.4.1 Tilting of building

The risk for tilting is analysed as described in chapter 3.1.1 according to Svenskt trä (2015). The two structures are at first analysed separately and then together as one building. An eight-storey building is investigated at first since this is the most unfavourable building and if there is no risk of tilting for this building there will be no risk of tilting for a six-storey building either. In figure 4.7, the structure (from its short side) is illustrated where Q1-Q8 are the wind loads and G is the self-weight of the structure. The wind load on the roof will be added to the load Q8. An investigation is done to ensure the distance e is less than the core boundary, $w/6 = 8.67/6 = 1.44$ m.

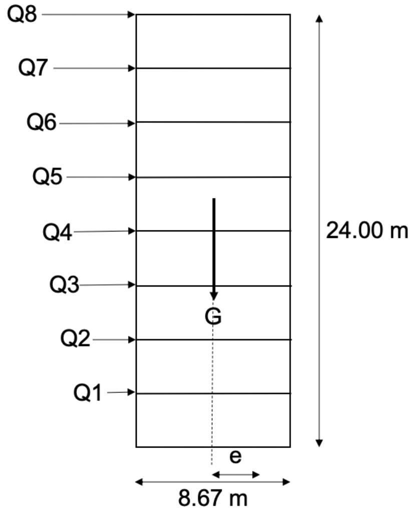


Figure 4.7: Set-up of forces acting on structure when analysing the risk of tilting. Seen from short side of building.

The weights of the different modules in the building are found in table A.1. The self-weight is multiplied with a factor 0.9 according to chapter 3.1.1. The self-weight is calculated as:

$$G = (7 \cdot V1 + 11 \cdot V2 + 4 \cdot V4 + 4 \cdot V5a + 6 \cdot V7) \cdot g \cdot n_{storeys} \cdot 0.9$$
$$= (7 \cdot 9790 + 11 \cdot 7858 + 4 \cdot 11030 + 4 \cdot 6702/2 + 6 \cdot 7173)/2 \cdot 10 \cdot 8 \cdot 0.9 \cdot 10^{-3} = 9200\ kN$$

The weight of module V5a is assumed to be half the weight of module V5, see figure A.8. The pressure wind loads acting on the structure are taken from table B.19 and the additional load on the roof are taken from table B.20 in appendix B. The results are presented in table 4.9, where H is the lever arm for every load down to the foundation.

Table 4.9: The wind load acting on every storey.

	Wind load (kN)	**H (m)**
Q8	375	24
Q7	213	21
Q6	205	18
Q5	195	15
Q4	184	12
Q3	170	9
Q2	151	6
Q1	121	3

The moment equilibrium of the loads is:

$$\sum Q_i H_i = G \cdot e$$

and the value for e can be determined by:

$$e = \frac{\sum Q_i H_i}{G}$$
$$= \frac{375 \cdot 24 + 213 \cdot 21 + 205 \cdot 18 + 195 \cdot 15 + 184 \cdot 12 + 170 \cdot 9 + 151 \cdot 6 + 121 \cdot 3}{9200} = 2.73\ m$$

The core boundary is located $w/6 = 1.44$ m from the centre line of the building and since $e = 2.73 > w/6 = 1.44$ there is a risk for tilting of an eight-storey building.

Since a risk of tilting was found for an eight-storey building the same calculations were done for a six-storey building. The factor e was then determined to $e = 1.68$ m, which is still larger then the core boundary of 1.44 m. This result in a a risk of tilting for a six-storey building as well. Further on, an investigation of a four-storey building was done and the factor e was determined to $e = 1.09$ m which is less than the core boundary, and no risk for tilting of a four-storey building, was found. This investigation is only focusing on the wind load and further research should be done to investigate the risk of tilting when accounting for sway imperfections and eccentric loads as well.

If the connections between the structures allow the forces to be taken by both structures, the structures could be analysed together as one building. The risk for tilting in this case is presented below.

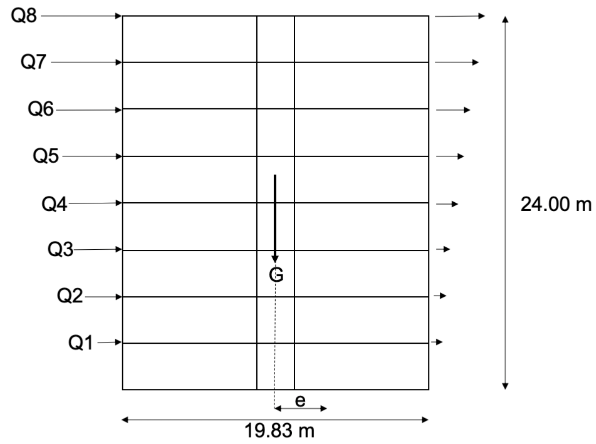


Figure 4.8: Set-up of forces acting on the building when analysing the risk of tilting for the whole building.

The self-weight of the whole building is calculated as:

$$G = (7 \cdot V1 + 11 \cdot V2 + 4 \cdot V4 + 4 \cdot V5a + 6 \cdot V7) \cdot g \cdot n_{storeys} \cdot 0.9$$
$$= (7 \cdot 9790 + 11 \cdot 7858 + 4 \cdot 11030 + 4 \cdot 6702/2 + 6 \cdot 7173) \cdot 10 \cdot 8 \cdot 0.9 \cdot 10^{-3} = 18400\ kN$$

The pressure and tension wind loads are taken from table B.19 and the additional load on the roof are taken from table B.20 in appendix B. The results are presented in table 4.10, where every Q represents both the pressure and tension loads for every storey and H is the lever arm for every load down to the foundation. The load Q8 also represents the wind load on the roof.

Table 4.10: The wind load acting on every storey, pressure and tension loads.

	Wind load (kN)	**H (m)**
Q8	605	24
Q7	352	21
Q6	339	18
Q5	323	15
Q4	305	12
Q3	281	9
Q2	250	6
Q1	200	3

e can then be determined by:

$$e = \frac{\sum Q_i H_i}{G}$$
$$= \frac{605 \cdot 24 + 352 \cdot 21 + 339 \cdot 18 + 323 \cdot 15 + 305 \cdot 12 + 281 \cdot 9 + 250 \cdot 6 + 200 \cdot 3}{18400} = 2.24\ m$$

The core boundary is located $w/6 = 19.83/6 = 3.31$ m from the centre line of the building and since $e = 2.24 < w/6 = 3.31$ there is no risk for tilting of an eight-storey building if the two structures can be connected in a way which will make them work as one building. This conclusion is drawn with regard to the assumptions and simplifications made in this report. Further investigations have to be done to account for sway imperfections and eccentric loads in order to determine the the real risk for tilting of the building.

4.4.2 Anchoring of outermost modules

The second problem for the building occurs because the building is made of modules. When the wind load acts on the long side of the building, tension forces are created on the gables. These tension forces act on the outermost modules and create a risk for the modules to tilt, if the tension forces are large enough. Anchoring between the outermost modules and the modules within may be necessary.

Two cases for the risk of tilting of modules are analysed. In the first case, the outermost small modules, between walls A-B and X-Y are considered, and in the second case the two outermost modules, between walls A-C and V-Y are analysed. The necessary capacity of the fasteners between these modules and the inner modules are designed to keep the modules in place. Two cases are considered since the results of the first case showed that fasteners were required for all heights of buildings. When the outermost modules are sufficiently fastened, these modules can work together with the second outermost modules and fasteners between the modules in wall C and V also had to be analysed to keep the two outermost modules in place.

Three different forces are of interest when analysing the risk of tilting. The self-weight of the modules (G in figure 4.9), the tension forces on the gables as a result of when the wind load acts on the long side of the building (Q1-Q4 in figure 4.9) and, if necessary, tension loads in the fasteners between the outer modules (F in figure 4.9). It is assumed that F is equal over the height of the building as stated in chapter 3.1.2. The moment equilibrium in equation 4.1 needs to be fulfilled to resist the risk of tilting.

$$M_F + M_G > M_Q \tag{4.1}$$

Where:

$$M_F = F(H_4 + 2 \cdot H_3 + 2 \cdot H_2 + 2 \cdot H_1) \tag{4.2}$$

$$M_G = G \cdot \frac{w}{2} \tag{4.3}$$

$$M_Q = Q_4 \cdot H_4 + Q_3 \cdot H_3 + Q_2 \cdot H_2 + Q_1 \cdot H_1 \tag{4.4}$$

And $H_1 = 3$ m, $H_2 = 6$ m, $H_3 = 9$ m and $H_4 = 12$ m for a four-storey building.

The wind load is designed according to appendix B, and one wind load is designed for each floor. The wind load is multiplied with the height to the floor where the load is acting, to get the rotating moment. The wind load acting on the top module represent the tension load acting on the gable of the roof structure as well. The counteracting moment from the self-weight is designed as the total self-weight of the modules piled upon each other multiplied with the lever arm to the edge of the building, as shown in figure 4.9 below.

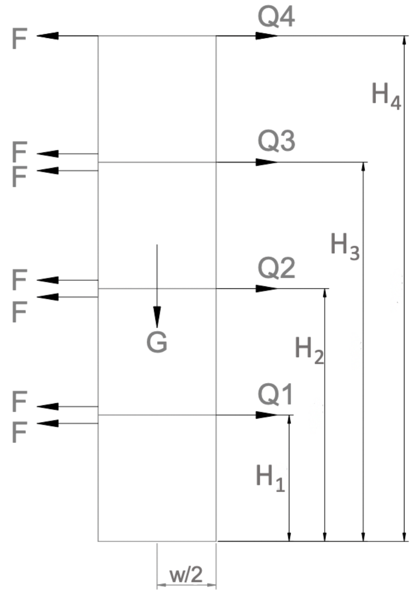


Figure 4.9: The forces of interest when analysing the need of anchoring of the outermost modules for a four-storey building.

An example describing the calculation method for the anchoring of the outermost modules for a four-storey building are presented below. The wind load acting at the top of storey four are calculated as:

$$Q_{d,H_4} = q_p \cdot c_{pe,10} \cdot 1.5 \cdot h \cdot w = 0.98 \cdot 1.2 \cdot 1.5 \cdot (1.5 + 2.32) \cdot 5.167 = 31.48\ kN$$

The height of influence for this load is $1.5 + 2.32 = 3.82$ m, where 1.5 m representing half the height of the module and the additional 2.32 m is the approximate equivalent height of the triangular gable on the roof structure, see figure 4.10.

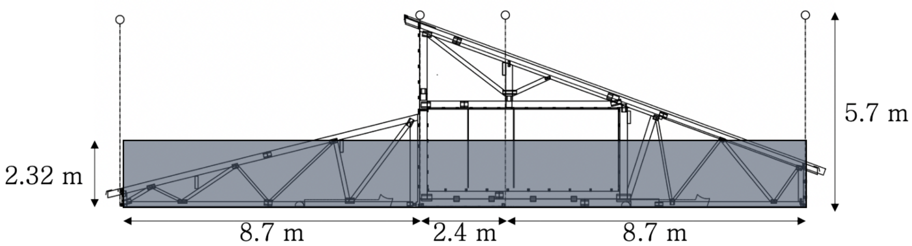


Figure 4.10: Equivalent height of triangular area of influence.

The same calculations are done for storey 3, 2 and 1, where $Q_{d,H_3} = 22.87$ kN, $Q_{d,H_2} = 20.32$ kN and $Q_{d,H_1} = 16.26$ kN. The total rotating moment from the wind load is presented as M_Q and calculated according to:

$$M_Q = Q_4 \cdot H_4 + Q_3 \cdot H_3 + Q_2 \cdot H_2 + Q_1 \cdot H_1 = 31.48 \cdot 12 + 22.87 \cdot 9 + 20.32 \cdot 6 + 16.26 \cdot 3 = 754\ kNm$$

The width of the outermost module is $w = 2.786$ m, and half this length is the lever arm for the self-weight, as shown in figure 4.9. The weight of the module is 3350 kg and there are four modules stacked on each other. The rotating moment from the self-weight is then:

$$M_G = G \cdot \frac{w}{2} = 4 \cdot 3350 \cdot 10 \cdot 10^{-3} \cdot \frac{2.786}{2} = 187\ kN$$

The fasteners along the height of the building, named F in figure 4.9, together with the self-weight of the modules, are designed to resist the overturning moment from the wind load. It is assumed that all the fasteners carry the same amount of load. The force in the fasteners can then be designed according to equation 4.1 since F is the only unknown parameter in the moment equilibrium.

$$F(H_4 + 2 \cdot H_3 + 2 \cdot H_2 + 2 \cdot H_1) + 187 > 754$$

$$F(12 + 2 \cdot 9 + 2 \cdot 6 + 2 \cdot 3) + 187 > 754$$

$$F = 12\ kN$$

The same calculations are done for a six- and eight-storey building as well and for the second case where the connection between the two outermost modules and the rest of the building are analysed, the results are presented in chapter 5 and a more detailed description of the investigation is found in appendix F.

4.4.3 Anchoring of top modules against up-lifting

The third tilting problem also occurs because the building is made up of modules. The top modules have to resist a rotating moment from the horizontal wind load as illustrated in figure 4.11, where Q is the wind load, G is the self-weight and F is the anchoring. H and L are the height and length of the module. Three modules on the top floor with different widths, lengths and weights are analysed.

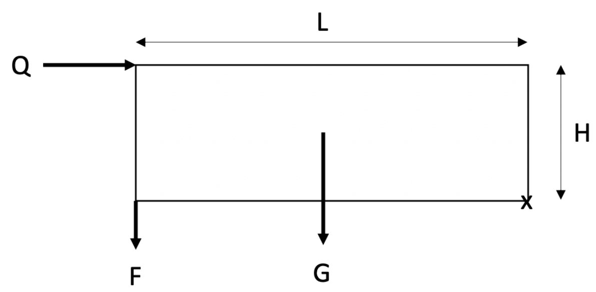


Figure 4.11: Set-up of forces acting on the uppermost module.

To be able to calculate the anchoring force F, following moment equilibrium is set up:

$$Q \cdot H = F \cdot L + G \cdot L/2$$

and F can then be determined by:

$$F = \frac{Q \cdot H}{L} - \frac{G}{2}$$

An example of the calculations for the top module between wall D and E in structure 1 is presented below. The weight of the module between wall D and E is 7858 kg according to table A.1 and the self-weight as a load is then:

$$G = 7858 \cdot 10 \cdot 10^{-3} = 78.58 \ kN$$

Q is the wind load acting on the module together with the wind load acting on the roof, values are taken from table B.19 and B.20, the design wind load Q is:

$$Q = (w_{e,pressure} \cdot \frac{h}{2} + q_{roof}) \cdot 1.5 \cdot w_{module} = (0.85 \cdot \frac{3}{2} + 3.07) \cdot 1.5 \cdot 3.94 = 25.67 \ kN$$

Where the influence height of the load acting on the module is $h/2 = 1.5$ m and the width of the module is 3.94 m.

The length of the module is 8.67 m and the height is 3 m and the anchoring force F can then be calculated as:

$$F = \frac{Q \cdot H}{L} - \frac{G}{2} = \frac{25.67 \cdot 3}{8.67} - \frac{78.58}{2} = -30.40 \ kN$$

Since F is negative, no anchoring is needed according to the calculations made in this report. A detailed description of the analysis of tilting and anchoring can be found in appendix F.

Chapter 5

Results and discussion

In this chapter, the different results of the investigations are presented and discussed. Proposals of how to improve the results are also going to be analysed. Firstly, the calculated capacities of the shear walls will be compared to the wind loads acting on the walls. Thereafter, a discussion of how to improve the results are presented, both according to the parameters in the design of wind load and for the parameters in the design of the capacity of the shear walls for the two methods, Method A and the elastic method. At the end of this chapter, the results for the up-lifting wind load on the roof structure, the risk for tilting and the need for anchoring of modules will be discussed and evaluated. As results are presented for different walls in the diagrams, the naming of the walls is again shown in figure 5.1, to ease understanding.

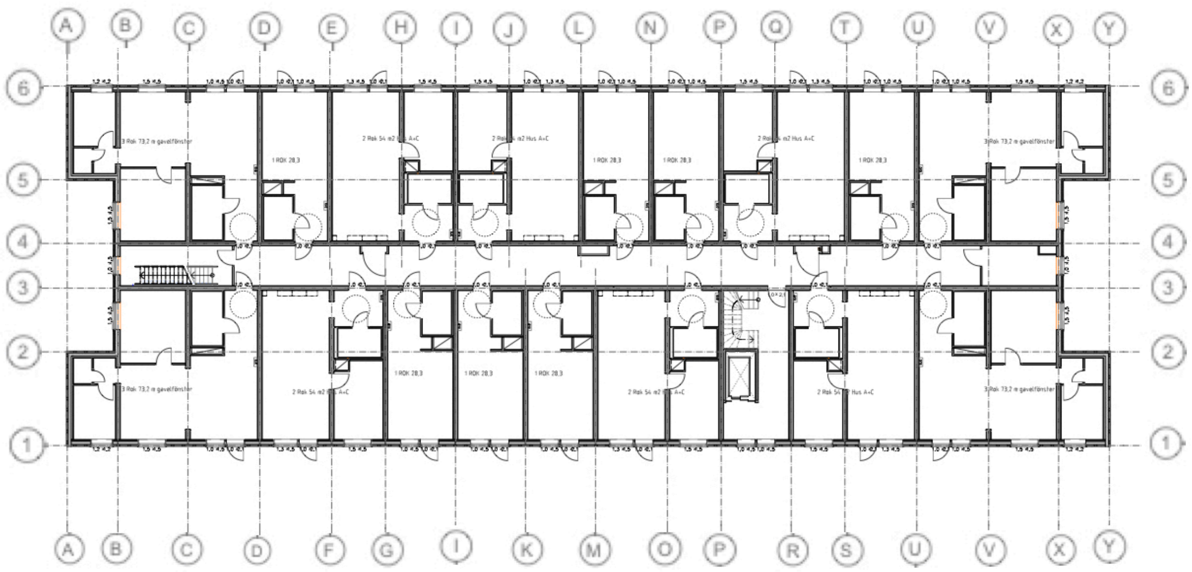


Figure 5.1: Naming of shear walls.

5.1 Comparison of wind load and design capacity

The design capacity for all the walls in the building is compared to the wind load acting on every wall, respectively. Firstly, the results for the two walls studied with both methods (wall 1 and wall B) are analysed, see figures 5.2, 5.4 and 5.5.

In figure 5.2, the capacities for wall 1 calculated with both methods are presented together with the wind load acting on the wall for all three heights of buildings, respectively. As shown in figure 5.2, there is a great difference between the wind load and the capacity for wall 1, which gives the wall a low degree of utilisation. Even though the utilisation is low for this wall, the panels in the wall may have other properties than only to resist horizontal loads, for example, to resist fire or acoustic requirements. Therefore, no suggestions for a decrease of the shear capacity is presented.

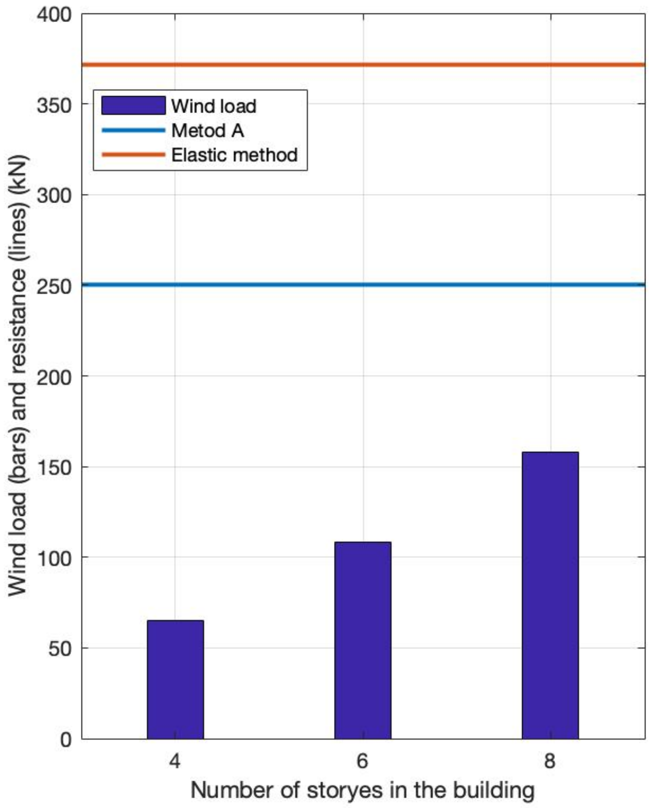


Figure 5.2: The wind load for the three different heights of buildings acting on wall 1 together with the design capacity of the wall according to both methods. Wind on wall A or Y.

In figure 5.4 the results for wall B are shown for when the wind load is acting on the long side of structure 1 (i.e. wall 1). The results for wall B when the wind load is acting on structure 2 (i.e. wall 6) is shown in figure 5.5. Wall B is divided into B1 and B2 depending on what module is analysed. This is because it is assumed that every module carries the wind load the module is exposed for. Wall B1 belongs to the module between wall A and B and wall B2 is part of the module between wall B and C, as shown in figure 5.3.

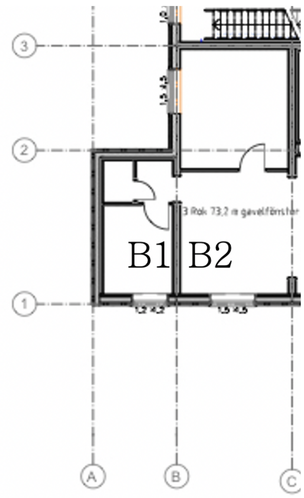
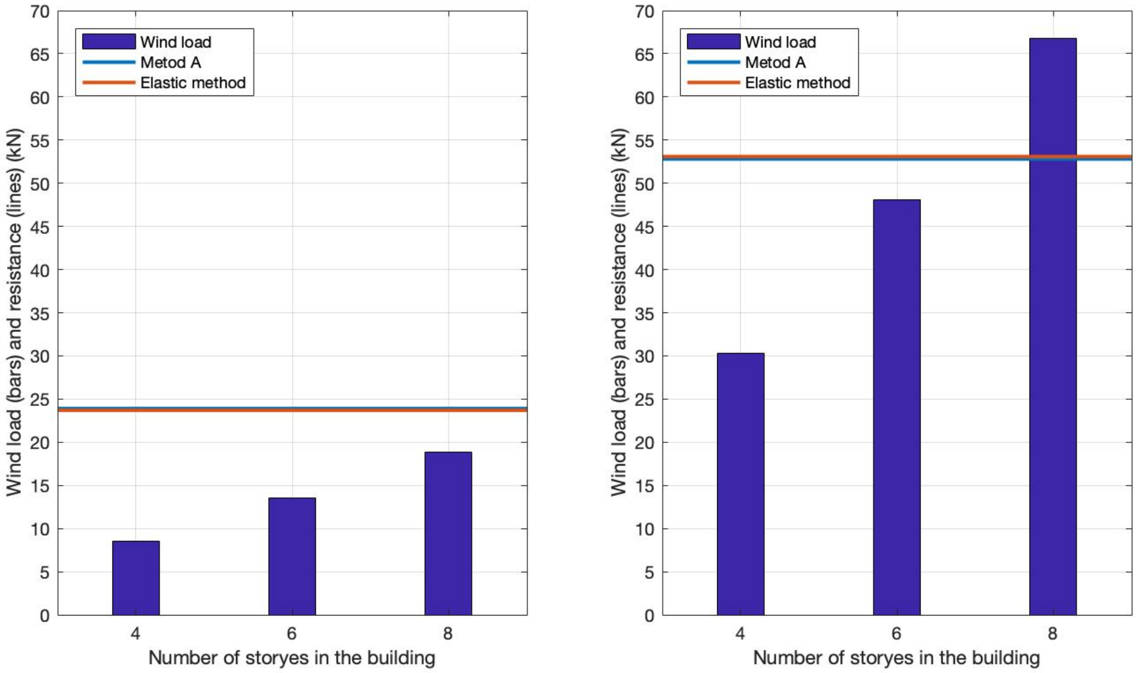


Figure 5.3: Illustration of B1 and B2.

The length of wall B1 is shorter than wall B2 since the modules between wall A and B are smaller, therefore, the capacity for wall B1 is less than wall B2. The width of the module between wall B and C is greater than the width of the module between wall A and B, therefore, the module between wall B and C are exposed for a greater wind load. As one can see in figures 5.4a and 5.5a, wall B1 will hold for all three heights of buildings and in figures 5.4b and 5.5b, it is shown that wall B2 will hold for a building with up to six storeys, but not for an eight-storey building.

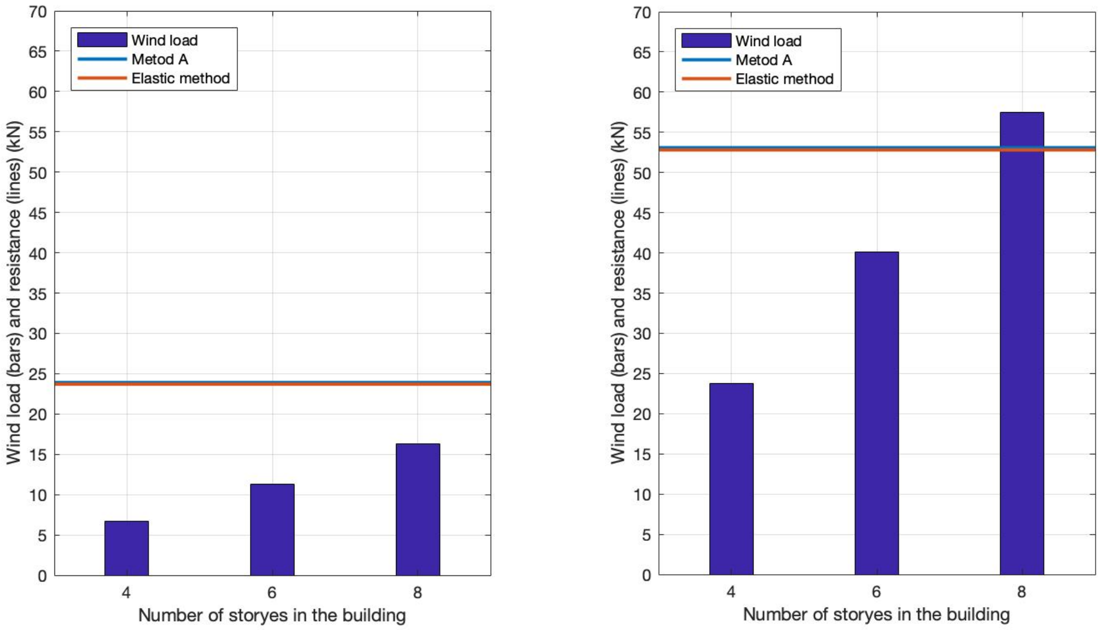
The designed capacity using the two different methods turned out to be very similar for wall B while there is a great difference in the designed capacities for wall 1. As shown in figure 5.2, the capacity for wall 1 is 250 kN according to Method A and 370 kN according to the elastic method. In figures 5.4 and 5.5 it is shown that the designed capacities for wall B are the same for both methods, 24 kN for B1 and 53 kN for B2. One possible reason why the capacities turn out to be the same for one wall and not for another, is because the elastic method does not reduce the capacity of short segments. For example, in wall 1, there are several segments less than 1.5 m and in Method A the parameter c_i reduces the capacity of segments shorter than 1.5 meters. In Method A, segments shorter than 0.75 m are even neglected. In the elastic method on the other hand, there is no parameter of this sort. This results in a greater capacity designed according to the elastic method since no segments of the wall are neglected. When neglecting the parameter c_i for Method A, almost the same capacity for wall 1 is obtained as it was for the elastic method. Wall B does not consist of as many small segments as wall 1 and is not affected by the parameter c_i in the same way.

A comparison of figures 5.4 and 5.5 shows quite similar results for the obtained wind loads depending on which long side of the building the wind load is acting. As one can see, when the wind load is acting on wall 6 (figure 5.5) the bars, symbolising the wind load, are a bit lower compared to when the wind load is acting on wall 1 (figure 5.4). This is because of the asymmetry of the roof structure. The building will be exposed to a greater wind load when the wind is acting on wall 1 compared to when it is acting on wall 6.



(a) Wall B1 (b) Wall B2

Figure 5.4: The wind load for the three different heights of buildings acting on wall B (divided into wall B1 and B2) together with the design capacity of the wall according to both methods. Wind on wall 1.



(a) Wall B1 (b) Wall B2

Figure 5.5: The wind load for the three different heights of buildings acting on wall B (divided into wall B1 and B2) together with the design capacity of the wall according to both methods. Wind on wall 6.

When analysing all the other walls in the building, only Method A is used for the design. This is because this method gives lower or the same values of the capacities compared to the elastic method (see figures 5.2, 5.4 and 5.5). This result in a more secure design. Even though this conclusion is drawn by only comparing two walls, the reducing parameter c_i for slender segments of the walls in Method A will always reduce the capacity compared to the elastic method where this parameter is absent. When neglecting this parameter in Method A, similar capacities are calculated with both methods. The presence of this reducing factor in Method A makes it more secure. It is also the method EC5-1-1 (2009) recommend for the design of shear walls, the method Derome is using today and it is also easier to use. Therefore, this is the method of most interest for the continuation of this report and further on given a greater part in the report, as a result.

Illustrations of the utilisation of all shear walls in the building will be presented (figures 5.6-5.8), where the bars represent the ratio between the wind load and the capacity, and the red line represent full utilisation (100%). A bar below the line represent a safe wall.

Firstly, all the walls in the longitudinal direction of the building are analysed. Walls 2 and 5 are neglected because of their shortness. The rest of the walls (1, 3, 4 and 6) are analysed and in figure 5.6, the ratios between the wind loads and the capacities regarding each wall are presented. Because of symmetry of the walls, walls 1 and 6 are presented together, same as for walls 3 and 4. If the bars in the graphs would be higher than 1, the wind load is greater than the capacity. This is not the case for any of the walls in the longitudinal direction and, as one can anticipate, the ratio between the capacity and the wind load increases for the higher buildings. This is because the capacity of the walls stays the same while the wind load increases for higher buildings.

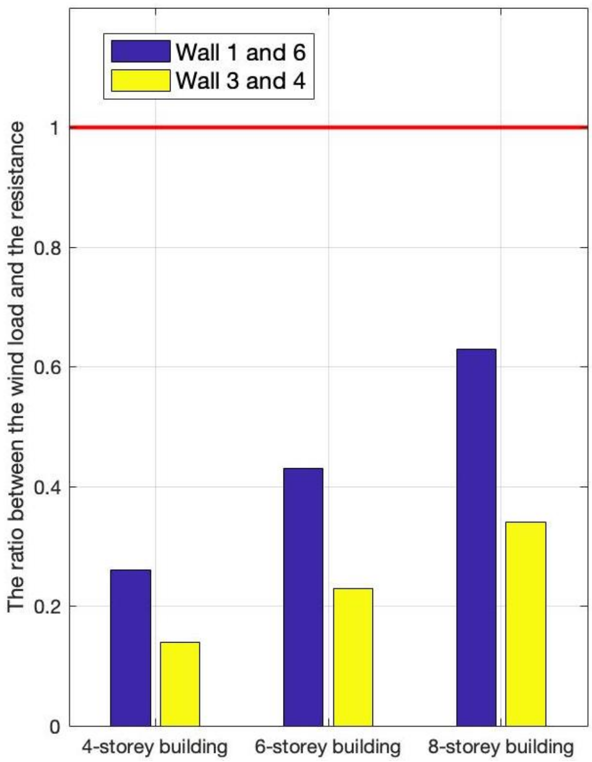


Figure 5.6: The ratio between wind load and the capacity according to Method A for the walls in the longitudinal direction of the building. Values lower than 1 indicates capacity > load. Wind on wall A or Y.

As shown in figure 5.6, the ratio between the different walls in the longitudinal direction differ. One explanation for this could be that the walls are made of different materials, walls 1 and 6 are made of OSB and walls 3 and 4 are made of fibre gypsum. The fasteners in OSB has a higher design shear capacity and this will result in a higher shear capacity of the wall, which in turn will result in a lower ratio for the OSB-walls. On the other hand, the number of openings, i.e. doors and windows, are notably lower for wall 3 and 4. The proportion of wall countable for the shear design is 42% for wall 1 and 6 and 80% for wall 3 and 4, with comparison to the total length of the walls. This could be the answer to why the ratio for walls 3 and 4 are significantly lower than for walls 1 and 6.

Secondly, all the walls in the transverse direction of the building are analysed. Because of the asymmetry of the roof structure, the ratio between the wind load and the capacity are presented both for when the wind load is acting on wall 1 (figure 5.7) and for when the wind load is acting on wall 6 (figure 5.8). Values above the line (i.e. greater than 1), indicate that the wall will fail due to lack of shear capacity. Walls B-X are divided into two parts (B1, B2, C1, C2 etc.) and since walls A and Y are only part of one module these two modules are not divided into two parts. Walls B-X are divided into two parts since it is assumed that every module carries the wind load the module is exposed for. It is possible that the two long walls in the modules carry different amount of wind load, depending on the stiffness of the walls. Walls with greater stiffness carry more load. When dividing the walls in parts 1 and 2, it is possible to observe which modules the critical walls belong to.

In figures 5.7 and 5.8 it is shown that two walls in the same module have the same utilisation. This is a result of the assumption on how the wind load is divided according to the stiffness of the walls. This assumption is probably not a conservative assumption. The assumption will lead to 100% utilisation of both walls in the module before the walls break, which is probably not true. On the other hand it is reasonable to think that the walls in a module will, in some way, collaborate but maybe not in the order of magnitude described above. The reality is maybe somewhere in between. The results of the wind load carried by the shear walls may, therefore, further on be less then the walls are exposed for in reality. If another approach of the distribution of the wind load was implemented maybe other walls also would fail due to the lack of shear capacity.

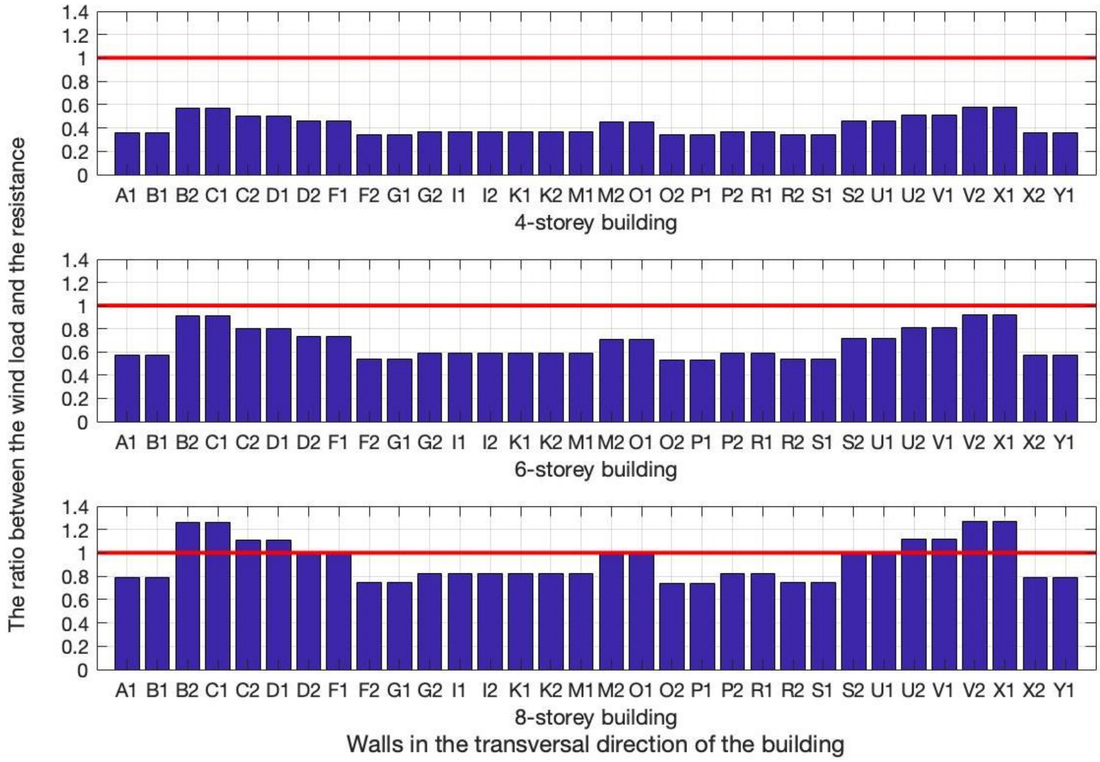


Figure 5.7: The ratio between the wind load and the capacity for all walls in the transverse direction of structure 1 for a 4-, 6- and 8-storey building. Values lower than 1 indicates capacity > load. Wind load on wall 1.

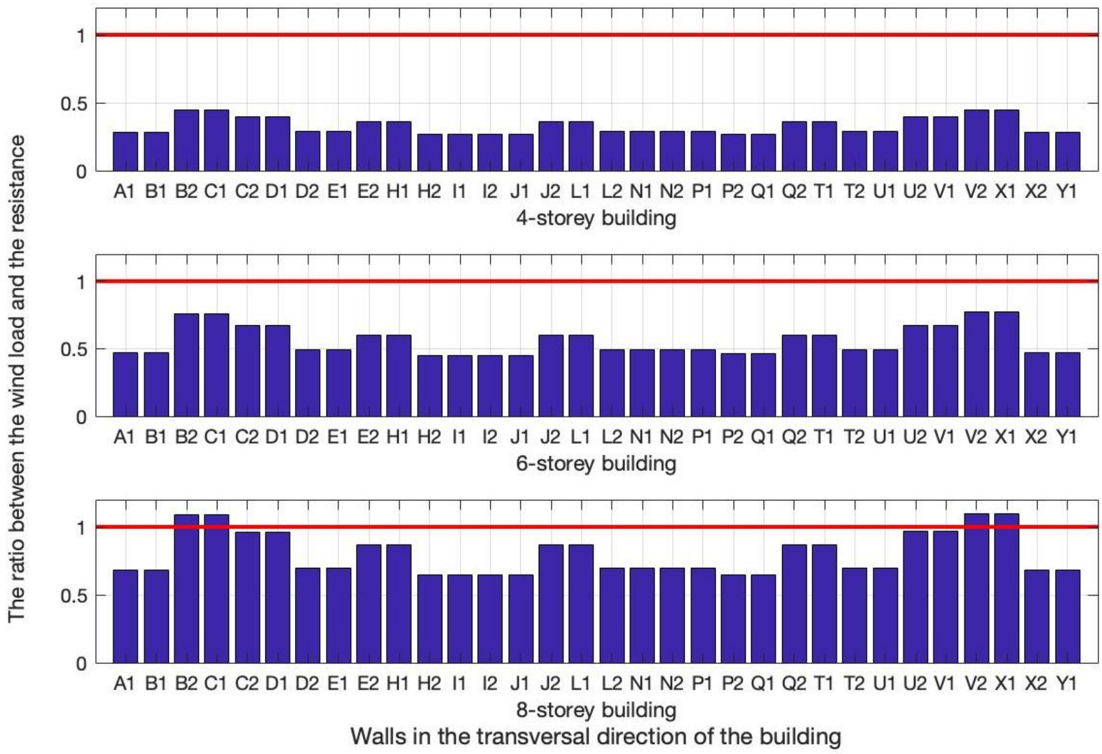


Figure 5.8: The ratio between the wind load and the capacity for all walls in the transverse direction of structure 2 for a 4-, 6- and 8-storey building. Values lower than 1 indicates capacity > load. Wind load on wall 6.

Similar results are obtained for the two cases, as shown in figures 5.7 and 5.8. The difference between the two cases is the lower wind load, due to the asymmetry of the roof structure, when the wind is acting on wall 6. Fewer walls will fail due to lack of shear capacity in structure 2 because of this. The utility (the bars in the graph) are therefore, greater when the wind load is acting on wall 1 and more walls will fail due to lack of shear capacity in structure 1.

In figure 5.7, it is shown that for an eight-storey building, many of the walls will fail due to a lack of shear capacity. An explanation why all the walls for a six-storey building will hold can be that, in reality, the investigated building was designed to be five storeys, where the first floor was made of concrete and the four floors above were made of timber. For this report, the first storey was neglected, and a four-storey timber building was analysed. The walls in this analysis of a four-storey building were therefore designed for higher loads (i.e. a five-storey building) and when raising the building to a six-storey building the difference in the wind load is not that big. This is maybe an explanation to why the walls fail only when the building is raised to eight storeys and not six storeys.

Some reasons why the walls will fail due to lack of shear capacity can be through too many or too large openings in the wall, bad placements of the openings or to wide modules which will lead to a great wind load. In chapter 5.1.1, a parameter study will be presented analysing the parameters used in the design.

When comparing the wind loads with the capacities for the different walls, it reveals that the most adverse structure is structure 1. This is because a greater wind load is acting on wall 1 compared to wall 6 because of the asymmetry of the roof. Since Derome wants the design of "AdderaPluss" to be simple and easy, both structures should be designed according to the most adverse load acting on the building. This will result in higher capacities for the walls in structure 2 which they never will be exposed for, but also easier production and assembly in the factory. Only one type on inner walls needs to be considered which make the building of the modules easier. As the walls in structure 1 are utilised to a higher degree, only the walls in structure 1 will be discussed further on in this chapter when discussing effects of different parameters.

5.1.1 Improvements of shear walls

When analysing parameters that can be changed to evolve different results, one parameter is changed at a time. Firstly, parameters according to the wind load are analysed and secondly, parameters in the two design methods for shear walls are analysed.

Throughout the report, it is important to keep in mind that the modules are factory-made and different solutions for different parts of the walls are not an option and therefore, the discussed parameters and properties are focusing on matters regarding the entire building. No great changes have been suggested, such as a core of concrete, CLT or steel. Instead, minor changes which will not affect the life cycle assessment of the buildings in any great extent will be analysed.

5.1.1.1 Wind load as a changing parameter

Parameters in the design of the wind load which were decided for this project were the basic wind velocity v_b = 25 m/s and the terrain category II. As shown in figures 5.7

and 5.8, the shear walls in both the four- and six-storey building managed to resist the horizontal loads with these input values.

An analysis of the six-storey building reveals that even if the basic wind velocity, v_b, was raised to the maximum value of 26 m/s, all the walls would hold. But if the terrain category was changed from II to I (and $v_b = 25$ m/s) the walls B1, C2, V2 and X1 would fail due to lack in shear capacity for a six-storey building.

For an eight-storey building, many of the walls would fail due to lack of shear capacity as shown in figures 5.7 and 5.8, but if the basic wind velocity, v_b, was lowered to the value of 21 m/s all the walls would hold. There are, however, only a few places in Sweden for which the value of v_b is this low. If the terrain category was changed instead, from II to III, all the walls would hold (for $v_b = 25$ m/s). A change in terrain category has more influence on the outcome than a change of basic wind velocity. This means that the terrain where the building is located is of great importance. An area with more obstacles, such as vegetation and buildings, reduce the exposure of the wind load acting on the building. Areas like this would for example be in a city. Higher buildings are often built near or in a city where the land is expensive and the buildings in the city could help the investigated building to reduce the exposure of the wind load. The eight-storey building should be built in areas where the terrain category is high to reduce the load acting on the building.

Another way to try to make sure the walls can carry all the loads created by the wind load is to make sure the load can be carried over the space between the two structures. If the connection were designed to be able to carry the loads between the structures, the loads could be carried by the shear walls in both structures. In that case, both the pressure and tension loads created by the wind load, needs to be accounted for in the design. An analysis of wall B2 for an eight-storey building when the wind load is acting on wall 1 will be presented below, regarding this change.

The wind loads presented below are taken from table B.21 in appendix B.

$$H_{d,pressure} = 28.65\ kN/m$$

$$H_{d,tension} = 18.51\ kN/m$$

The resistance of wall B is taken from table D.6 in appendix D.

$$F_{v,Rd} = 52.81\ kN$$

Firstly, the ratio between the wind load and the capacity is presented for the case where the two structures do not cooperate, see equation 5.1. Secondly, the ratio for the case where the two structures work together is presented in equation 5.2. w is the influence width of the wind load and it is depending on resistance of the two long walls in every module.

$$\frac{H_{d,pressure} \cdot w}{F_{v,Rd}} = \frac{28.65 \cdot 2.33}{52.81} = 1.26 \tag{5.1}$$

$$\frac{(H_{d,pressure} + H_{d,tension}) \cdot w}{F_{v,Rd} \cdot 2} = \frac{(28.65 + 18.51) \cdot 2.33}{52.81 \cdot 2} = 1.04 \tag{5.2}$$

As shown in equation 5.1 and 5.2, wall B will still fail due to the lack of shear capacity when the two structures work together. On the other hand, the ratio became smaller and some other minor changes can be done for the wall to hold. In addition to this, the connectors need to be designed to make sure that the loads can be carried over the corridor between the structures to be able to do this design.

As stated in the limitations of the report, the internal wind loads are neglected. This is often done when investigating a whole building but when focusing on elements in the structure, both the external and internal wind loads are included. When analysing the modules it is unclear whether the module should be analysed as an element or not. In this report it is assumed that every module is a part of the building and not a single element, which lead to that the internal wind loads are neglected. On the other hand, when the wind load is acting on the long side of the building, one structure is analysed at a time and the modules should perhaps be analysed as an element. The internal wind loads should then be included and added to the external pressure loads acting on the module. This will result in higher wind loads on the walls and other walls may fail due to lack of shear capacity. The issue whether to include the internal wind load or not should be further analysed in order to design for the proper loads.

5.1.1.2 Parameters in the design of shear capacity

The parameters included in the design of shear capacity will be investigated and proposals of how to improve the capacity according to these parameters will be presented.

Method A

The parameters needed for a design according to Method A are mentioned in the equations 5.3 and 5.4.

$$F_{i,v,Rd} = \frac{F_{f,Rd} \cdot b_i \cdot c_i}{s} \tag{5.3}$$

Where:

$F_{f,Rd}$	is the lateral design capacity for a single fastener;
b_i	is the wall panel width;
s	is the spacing of fasteners.

and

$$c_i = \begin{cases} 1 & \text{for } b_i \geq b_0 \\ \frac{b_i}{b_0} & \text{for } b_i < b_0 \\ 0 & \text{for } b_i < \frac{b_0}{2} \end{cases} \tag{5.4}$$

Where:

b_0 =h/2;
h is the height of the wall.

As shown in equation 5.3, a greater value on either the design capacity for a single fastener, $F_{f,Rd}$, or the width of the panel b_i will give a higher shear capacity of the panel. Likewise, a smaller spacing of the fasteners increases the capacity of the panel. To make sure that the parameter c_i is as big as possible, i.e. equal to 1, the width of the panel should be greater than half the height of the wall. The number of panels with a width less than $h/4$ should be minimised since these panels are excluded when designing the capacity of the wall.

The positions of openings in the walls are therefore a major opportunity to make sure no parts, or as few as possible, are neglected or decreased in the design.

As shown in figures 5.7 and 5.8, walls B2, C1, C2, D1, D2, F1, M2, O1, S2, U1, U2, V1, V2 and X1 are the critical walls for a building of eight storeys. The reason why only one side of a wall can be critical is that the wind load acting on every module, is distributed according to the stiffness of the walls in the module. If one wall in a module has higher capacity, that wall will have to carry more load. A more detailed analysis of the critical walls and the locations of the openings (i.e. doors) are therefore a good way to try to increase the capacity.

An example of the importance of the location of openings is analysed for walls F and S. There is a 0.6 m segment in these walls and since it is less than $h/4 = 0.75$ m, it is neglected when designing the capacity of the walls (EC5-1-1, 2009). If the openings next to these short segments were moved 0.15 m to make the length of these small segments 0.75 m instead, the segments would contribute to the capacity of the walls. When making this change, both wall F and S get a capacity exceeding the wind load, even for an eight-storey building. The problematic part about these changes is that the walls may look like this for another reason. The openings are maybe located in a certain way to make sure that common furniture can fit in the different rooms or to fulfil requirements about adaption of disabled etc.

An analysis of the part of wall B that is between wall 1 and 2 (see figure 5.9) is done to find the most optimal location for the opening in the wall. The two segments of this part of the wall are named b1 and b2, see figure 5.9. The total length of b1 and b2 together is 3.7 m.

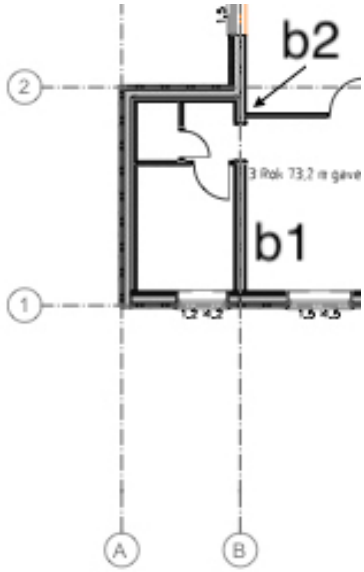


Figure 5.9: Definition of b1 and b2. For the whole picture, see figure 5.1.

In figure 5.10, the optimal placements of the opening are shown. On the x-axis, the length of wall b1 or wall b2 is shown and the y-axis represent the shear capacity. For example, if it is assumed that the x-axis represents the length of b1, the blue line is segment b1, the red line is segment b2 and the orange line is the summed capacity of the two segments (b1+b2). For the case when the length of b1 is 0.75 m the capacity is 3 kN (blue), the capacity of the segment b2 is 23 kN (red) and the total capacity of both b1 and b2 is 26 kN (orange).

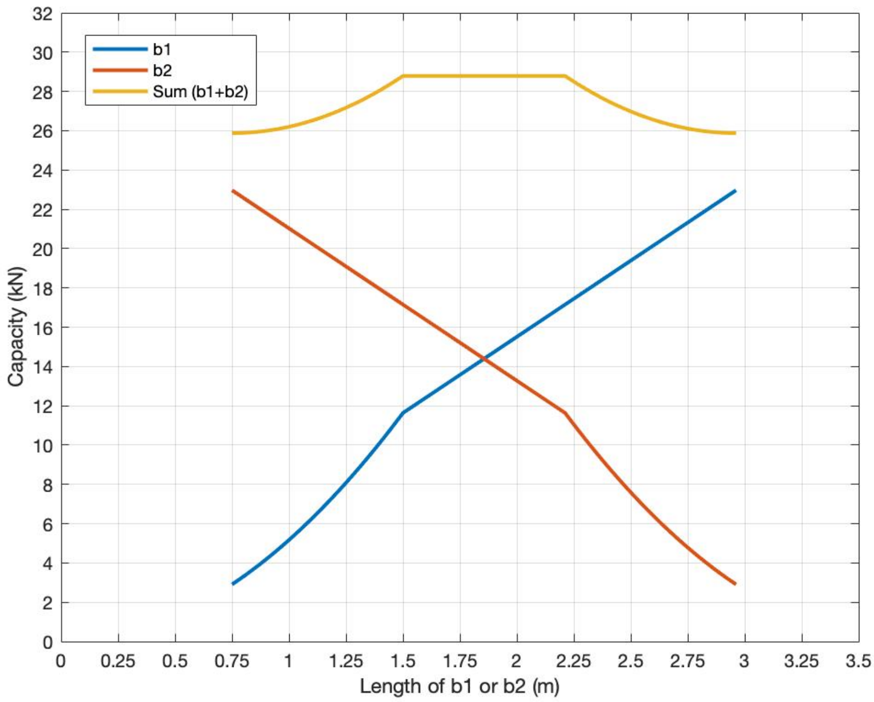


Figure 5.10: The optimal placements of the opening.

Figure 5.10 reveals that the optimal placement of the opening is when both segments are greater than 1.5 m. This is because of the parameter c_i which reduces the capacity for segments shorter than $h/2 = 1.5$ m. If the length of both segments is greater than 1.5 m the capacity of this part of the wall will constantly be 28.80 kN. If one of the segments is less than 1.5 m, the capacity will be reduced. A limitation of the length of the segments is selected to be 0.75 m, since segments less than this length are neglected.

In the design in this report, no wall segments less than $h/4 = 0.75$ m were taken into account, as recommended in EC5-1-1 (2009). When this problem was discussed with Derome, it was revealed that they usually use a different approach. According to the results of tests they have performed, the capacity of segments between 0.6-0.75 m can be reduced by half their capacity respectively. It makes sense that a segment of 0.74 m still contributes to the capacity even if it is not wide enough according to Eurocode. When that approach is used for the walls investigated in this report, the capacity of walls D2, F1, S2 and U1 for an eight-storey building, turns out to be greater than the wind load. Even if it is a small addition (0.93 kN/m) of the total capacity of wall D, the small contribution changes the utilisation of wall D2 from 1.01 to 0.999. It is still a high degree of utilisation and other changes maybe have to be done as well to make sure the walls will resist the loads.

The risk of shear buckling of the sheets are calculated and presented below. As shown, the values calculated are significant lower than the limit value and since the statement is fulfilled for both materials, no further consideration will be taken to the shear buckling of the sheets.

$$\frac{b_{net}}{t_{OSB}} = \frac{600}{11} = 54.5 \leq 100$$

$$\frac{b_{net}}{t_{fibregypsum}} = \frac{600}{25} = 24 \leq 100$$

Elastic method

The design of shear capacity according to the elastic method is dependent on the parameters shown in equations 5.5-5.10. The method is more in detail described in appendix E.

$$H_{Rd} = \frac{F_{vd}}{h\sqrt{(\frac{x_{max}}{\sum x_i^2})^2 + (\frac{y_{max}}{\sum y_i^2})^2}} \tag{5.5}$$

Where $x_{max} = w/2$ and $y_{max} = h/2$ according to figure 5.11.

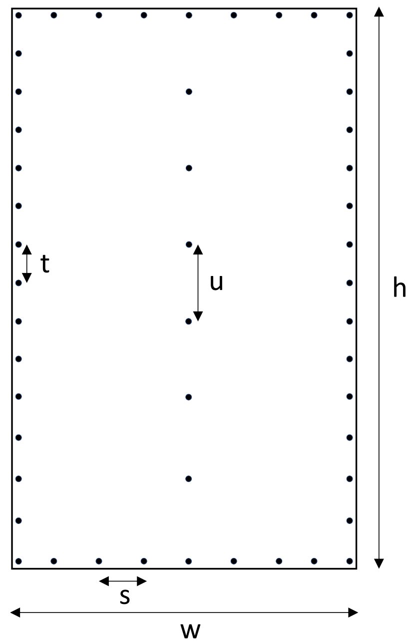


Figure 5.11: The dimensions of a panel.

Expressions for $\sum x_i^2$ and $\sum y_i^2$ are taken from Carling et al (1992) and for a panel like the one in figure 5.11, the expressions are:

$$\sum x_i^2 = \frac{w^2}{12}(2n + 6m) \tag{5.6}$$

$$\sum y_i^2 = \frac{h^2}{12}(6n + 2m + p - 3) \tag{5.7}$$

Where:

$$n = \frac{w}{s} \tag{5.8}$$

$$m = \frac{h}{t} \tag{5.9}$$

$$p = \frac{h}{u} \tag{5.10}$$

Parameters which can be changed to obtain different results with the elastic method are mostly regarding the distances between the staples in the panel. If either of the distances t, s and u are reduced, the capacity of the panel is increased.

An analysis is done to obtain which parameter, t, s or u, affects the capacity the most. 10 % more staples on the panel are used for one parameter at a time. The analysis is done for the first segment in wall 1, with a length of 1.08 m, where the original capacity was 7.42 kN. When the values calculated in equations 5.11-5.13 where used instead for the parameters t, s and u, one at a time, the results presented in table 5.1 were provided. The results reveal the location on where to increase the number of staples on the sheet to increase the shear capacity the most.

As shown in table 5.1, parameter t generates the greatest effect (8 %) on the shear capacity according to the elastic method, while the parameter s generates 2 % higher capacity and the parameter u only generates 0.3 % higher capacity. The best location to put extra staples to increase the shear capacity is, by the results from table 5.1, on the vertical edges on the long sides of the panel. The wall has an upper limit of the capacity according to the resistance of shear buckling in the sheet, therefore, the number of staples cannot increase the capacity above this limit.

$$s_{10\%} = 0.08/1.1 = 0.072\ m \tag{5.11}$$

$$t_{10\%} = 0.08/1.1 = 0.072\ m \tag{5.12}$$

$$u_{10\%} = 0.16/1.1 = 0.145\ m \tag{5.13}$$

Table 5.1: The change of capacity when increasing the number of staples for one parameter (t, s or u) at a time.

	s	**t**	**u**
Distance between staples (m)	0.072	0.072	0.145
H_{Rd} (kN)	7.56	8.00	7.44
Increase of capacity (%)	1.9	7.8	0.3

Parameters regarding both methods

The value for $F_{f,Rd}$ in Method A equals the value F_{vd} in the elastic method and these values, used in the analysis, were obtained from Derome,:

$$F_{f,Rd,OSB} = 0.555\ kN$$

$$F_{f,Rd,fibre\,gypsum} = 0.517\ kN$$

These values are dependent on the material of the sheet and the type of staples. The design capacity of a single fastener is calculated according to chapter 8.2 in EC5-1-1 (2009). The calculations done for the fibre gypsum boards are obtained from Derome and after analysing the results, the failure mode of the staples is mode (f), see figure 5.12, i.e. full plasticity in the fastener. The equation to determine the characteristic capacity of a single fastener with failure mode (f) is found in equation 5.14.

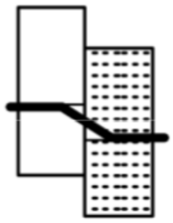


Figure 5.12: Failure mode (f) (EC5-1-1, 2009). Used with permission.

$$F_{v,Rk,(f)} = 1.15\sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk}f_{h,1,k}d} + \frac{F_{ax,Rk}}{4} \quad (5.14)$$

Where

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

and

$f_{h,i,k}$ is the characteristic embedment strength in wood member i;
β is the ratio between embedment strengths of members;
$M_{y,Rk}$ is the characteristic yield moment in fastener;
$F_{ax,Rk}$ is the characteristic withdrawal capacity of the fastener.

An increase of the capacity of a single fastener will increase the capacity of the shear walls, see equations 5.3 and 5.5. An increase of the capacity of the fasteners can be done through the following actions:

– Increase $M_{y,Rk}$ (i.e. change the steel to a higher class).

– Increase $F_{ax,Rk}$. According to EC5-1-1 (2009), the contribution of $F_{ax,Rk}/4$ should not exceed a certain percentage (depending on type of fastener) of the first term in equation 5.14. $F_{ax,Rk}$ can be increased by a greater penetration depth or a greater withdrawal strength of the staple.

– Increase $f_{h,i,k}$, either for the boards or for the structural timber. $f_{h,i,k}$ is dependent on the density and the increase of density for either the boards or for the structural timber will increase $f_{h,i,k}$. To increase the density of the structural timber, a higher strength class is necessary. These changes require new types of materials and will cause a totally new design of the shear walls.

The diameter of the staples should not be increased in any great extent because the type of failure mode can then be changed to a brittle failure, i.e. the staples are too stiff compared to the sheet. In design, a ductile failure mode (such as failure mode (f)) should be aimed for.

5.2 Up-lifting loads

As shown in tables 4.3 and 4.4, the up-lifting forces were lower than the self-weight of the uppermost modules. The decisive module, in this case, is the module between wall A and B in structure 2 where the ratio between the up-lifting load and the self-weight is 0.93. For these calculations, no account where taken to the self-weight of the roof structure, only the self-weight of the uppermost modules. If the self-weight of the roof-structure were added to the self-weight of the modules, the ratio would be lowered and more secure. This, on the other hand, indicates that during the assembling of modules on the building site, there is no risk for up-lifting forces on the upper modules, not even before the roof structure is put in place.

An investigation where the up-lifting loads on the roof are acting at the same time as the up-lifting loads from the horizontal wind loads will be presented in chapter 5.3.3.

5.3 Tilting and anchoring

Three problems of tilting and anchoring are presented in chapter 4 and the results for those problems are given below.

5.3.1 Tilting of building

The results of the first problem, where the risk for tilting of one structure of eight floors was analysed, showed risk for tilting since the parameter e was greater than the core boundary. e was determined by:

$$e = \frac{\sum Q_i H_i}{G} = 2.73\ m$$

The core boundary is located $w/6 = 8.67/6 = 1.44$ m from the centre line of the building and since $e = 2.73 > w/6 = 1.44$ a risk for tilting is found, therefore, anchoring to the foundation is necessary.

The same result was found for a six-storey structure where the value of e was calculated to 1.68 m which is still greater than the core boundary of 1.44 m and an anchoring to the foundation is necessary for a six-storey structure as well. When the number of storeys was decreased to four storeys, the value of e was calculated to 1.09 which is lower than the core boundary and no risk for tilting of a four storey structure was found.

To decrease the risk for tilting the two structures can be connected in a way that allows the loads to be carried by both structures. If this is done the whole building can be investigated for the risk of tilting. The width of the building can then be taken as 19.83 m (the width of both structures and the corridor in between). This result in a core boundary of $w/6 = 19.83/6 = 3.31$ m and as described in chapter 4.4.1, the value of e for an eight-storey building was determined to 2.24 m. Since e is less than the core boundary no risk for tilting was found.

Due to this results it is important to make sure the two structures can be analysed as one building, especially when the building is higher. This can be done through a verified connection between the two structures, with for example beams implemented between the structures. These beams should be designed for the compression load which must be carried from one structure to the other. The width of the building can then be taken as the whole building (19.83 m) and is then large enough to secure the risk of tilting.

5.3.2 Anchoring of outermost modules

As described in chapter 4, the anchoring of the outermost modules in the building are analysed because of the tension forces created on the gables when the wind load is acting on one of the long sides of the building. Table 5.2 shows how much the connectors between the modules need to resist due to these tension loads. The analysis is in detail described in appendix F. The connections are illustrated in figure 5.13.

The results show that the connectors in wall B and X need to be designed to resist 12, 14 or 15 kN depending on the height of the building, since the wind load increases for the higher buildings. If the connectors are designed for these loads, connectors in wall C and V are only needed for an eight-storey building. The connectors in these walls will have to resist a load of 5 kN to reduce the risk of tilting. An investigation of the connections between wall D1-D2 and U1-U2 are also done (the third outermost modules), but no connectors are needed there, not even for an eight-storey building. Since the self-weight of the modules act as a counteracting moment for the wind load, it is perceivable that the outermost modules need to be fastened more than the inner modules. A lot of weight is contributing to the counteracting moment when analysing the two or three outermost modules compared to the analysis of the outermost module alone. The results presented in table 5.2 depend on the assumptions and simplifications made for this report and this matter should be further analysed taking into account sway imperfections and eccentric loads.

Table 5.2: Tension forces in fasteners for the different buildings.

	Force F in walls B and X (kN)	**Force F in walls C and V (kN)**
4-storey	12	0
6-storey	14	0
8-storey	15	5

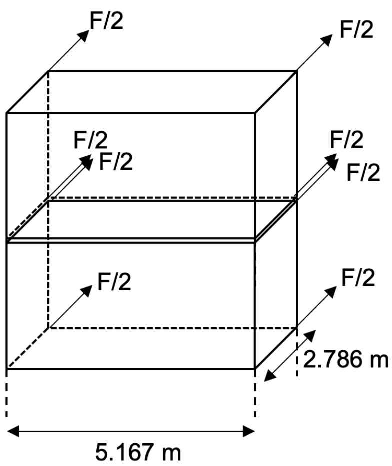


Figure 5.13: The distribution of the load for the fasteners holding the two top modules in wall B or wall X in place.

The modules are today connected with steel straps. It is a strap of steel with several holes in it to be able to fasten it with nails or screws. Since the connection will have to resist more loads for the higher buildings (2 kN more for a six-storey building and 3 kN more for an eight storey-building) the steel straps maybe have to be fastened with more screws, depending on the capacity of the already used steel straps.

5.3.3 Anchoring of the top modules against uplifting

The top modules are analysed to investigate whether it is necessary with an anchoring of these modules or not. The anchoring force F is determined by:

$$F = \frac{Q \cdot H}{L} - \frac{G}{2}$$

Where Q is the wind load, L is the length of the module, H is the height of the module and G is the self-weight of the module. Three modules in structure 1 on the top floor of an eight-storey building are analysed; the modules between walls A-B, D-E and F-G. These three modules are chosen because of their variation in length, width and weight. The load and the necessary anchoring are presented in table 5.3. The force F is in all three cases negative, i.e. no anchoring of the top modules for an eight-storey building is required when analysing this matter individually. Since this is the worst case, no anchoring of the top modules for a six-storey building is required either as the wind load for six storeys is less than for eight storeys (all other factors constant). The small modules between walls A-B and X-Y are the most vulnerable, but according to the design, no risk of up-lifting is determined, and no anchoring is necessary.

Since this is an up-lifting force on the top module this result should be analysed together with the up-lifting forces created on the roof when the wind load is acting on the long sides of the building as described in chapter 4.1.1. The up-lifting forces on the roof are only analysed for when the wind load is acting on the gables, since this was the worst case. When the same analysis is done for the up-lift of the module between walls A and B, but for when the wind load is acting on the long side of the building, an up-lifting force of 6.43 kN is found. This is compared to the force F in table 5.3 for the module between walls A and B, and as one can see $F = 6.22 < 6.43$ kN. This shows that there is a risk for up-lifting of this module when the two up-lifting forces are analysed together. However, the self-weight of the roof structure is not taken into account in these calculations which will benefit the risk for no up-lifting. However, further analysis are required to certain the risk for up-lifting of the top modules when the wind load is acting on the long sides of the building.

Table 5.3: The anchoring force of the top modules of an eight-storey building.

	A-B	**D-E**	**F-G**
G (kN)	33.51	78.58	97.90
L (m)	5.17	8.67	8.67
Q (kN)	18.15	25.67	19.16
F (kN)	-6.22	-30.40	-42.32

Chapter 6

Conclusions

The conclusions for this report are made with accordance to the assumptions done in the investigation and the results are indications of the parameters which should be further analysed to be able to build both a six-storey and an eight-storey building.

Two methods for the design of shear capacity of the walls have been analysed, Method A (EC5-1-1, 2009) and the elastic method (Carling et al, 1992). The elastic method has only been used for two walls in the investigated building, in contrast to Method A, which has been used on all the shear walls in the building. Both methods show that the existing walls are capable to carry the wind load for a six-storey building, and not just the five-storey building they were designed for. When the building is raised to an eight-storey building, some of the walls in the transverse direction of the building will fail due to lack of shear capacity.

Different solutions and changes are discussed to increase the capacity of the shear walls or to decrease the wind load. The design wind load is a decisive factor regarding the stability of light-weight modular buildings. Therefore, the maximum number of floors largely depends on the location of the building. Depending on where in Sweden the building is located, the basic wind velocity changes, although the terrain category for the building, i.e. the type of surroundings such as presence of obstacles and vegetation, has the most impact on how great the proportion of wind load acting on the building will be.

The number of floors also depends on the capacity of the shear walls. A way to increase the capacity of the shear walls is if the connections between the two structures can be designed to allow the wind load acting on the long sides of the building to be carried by both structures, and not just one. This creates additional shear walls in the transverse direction of the building to resist the wind load. On the other hand, this solution involves that the shear walls in the transverse direction will have to resist both the pressure and tension forces created by the wind load and not just the most unfavourable load, as assumed previously. The results from this analysis still showed signs of lack of shear capacity in the transverse walls, but the ratio between load and capacity was decreased.

The location of openings in the walls have a big impact on the capacity of the walls. In the design according to Method A, the parameter c_i reduces the capacities for all segments of the walls less than 1.5 meters. If the openings can be placed to minimise the number of segments less than 1.5 meters the capacity would increase. Segments less than 0.75 m are neglected according to EC5-1-1 (2009) and Derome has chosen to reduce the capacities of segments between 0.6-0.75 m by half, therefore the number of segments this short should also be minimised.

The capacity of the shear walls is also dependent on the shear capacity for a single fastener in the wall. Different parameters according to the design of the fasteners were discussed in chapter 5 to raise the capacity of the fastener and through this, raise the capacity of the shear walls.

Except for the design of capacity of the shear walls, the risk of tilting and the need for anchoring between modules have been analysed and the results reveal a risk for tilting of both a six- and an eight-storey building. These results are based on the fact that the two structures are not combined in way which make them work together, and in this case, either the self-weight of the structure needs to be increased or the modules needs to be fastened into the foundation to resist the risk of tilting. If the building can be evaluated as one system, where the two structures are connected and work together, the analysis shows an indication that the risk of tilting is eliminated, even for an eight-storey building.

According to the calculations done for the analysis of the anchoring of the top modules, indications show a risk for up-lifting for an eight-storey building but further investigation are necessary to determine the required capacities of the anchoring. The outermost modules need to be anchored to the modules within to resist the tension loads created on the gables of the building. For an eight-storey building, the second outermost modules also have to be anchored to the modules within. The dimensions of the fasteners have to be further analysed to be able to resist the greater loads for the higher buildings since the results presented in chapter 5 depend on assumptions and simplifications done for this report.

6.1 Further research questions

The content of this report has been one part of the design of the stabilising system for buildings made with volume building technology. Many other aspects need to be considered to be able to design an eight-storey building with this technology. Further research questions for the ability to construct higher buildings with volume building technology are stated below:

- Since a six-storey building will hold for the horizontal loads according to this report, will this design meet the requirements for fire-safety and acoustics?
- Can the change of type of the boards result in an increase of the capacity of the shear walls?
- A more robust core (for example made of CLT) is maybe needed for the higher buildings to withstand the loads, how can it be designed?
- Do the connections within the modules resist the higher loads and if not, how can the connections be designed?

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Appendix A

AdderaPluss

AdderaPluss is a concept at Derome's with the agenda to make the design easy. By following the five steps numerated below, the buyers have configured the whole building (Plusshus, 2018).

1. Plan your site.
2. Choice of type of building.
3. Combination of modules.
4. Exterior parts of the building.
5. Interior parts of the building.

By having a standardised basic supply of modules, the production process gets more efficient and the price can be reduced. The idea of AdderaPluss is to have a high degree of standardisation and, at the same time, make sure there is a great opportunity to variation (Plusshus, 2018).

A.1 Type of building

For the concept AdderaPluss there are two types of buildings to choose from, either slab blocks or point blocks. In figure A.1 and A.2 the different houses are presented.

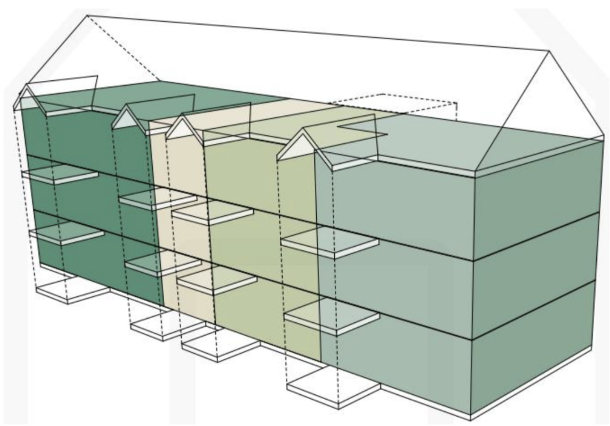


Figure A.1: An example of a slab block (Plusshus, 2018)

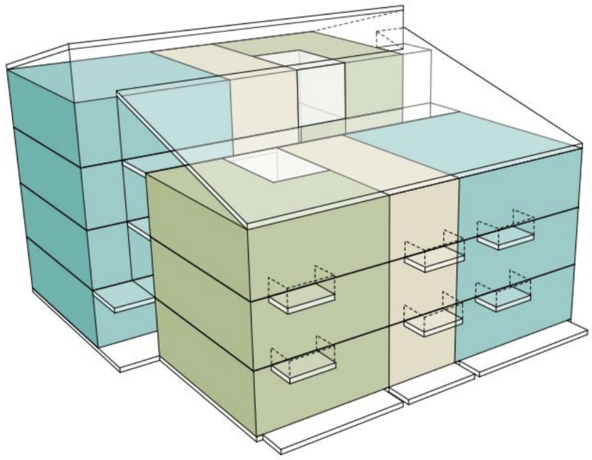


Figure A.2: An example of a point block (Plusshus, 2018)

When building with slab blocks, all the apartments are assured to get sunlight from at least two sides of the apartment. The stairwell for this type of house is constructed on the outside of the building. The house type named point blocks are two linked slab blocks with the staircase between the blocks. If any building is more than three storeys high, an elevator is required. The elevator is then located in the building, but the shaft is not used as a stabilising system. The investigated building for this report is a point block, where the two slab blocks will be called structures and when discussing them together it will be referred to as the building.

A.2 The different modules

Derome offers seven different modules that are presented in figures A.3 and A.4. The modules can be combined into ten different apartments, these are presented in figures A.5 and A.6. The smallest size for an apartment is a one-bedroom studio apartment and the largest is a three bedrooms apartment. Module number five and seven can be divided into two parts which allows making one and a half room apartments, for example. When one floor is designed, the same formation will usually be set for all floors to make sure that the walls are directly upon each other and the load can easily be carried down to the foundation. The maximum height of AdderaPluss is today four storeys and if the building is set to this height a certain length of the building is required to make sure there is enough walls to carry the horizontal loads. One example of a floor combined with different modules is presented in figure A.7.

Figure A.3: Volume type 1, 2, 5 and 6. Translation: Yttermått=Outer dimensions and kvm=m²(Plusshus, 2017).

Figure A.4: Volume type 3,4 and 7. Translation: Yttermått=Outer dimensions and kvm=m^2(Plusshus, 2017).

Figure A.5: Apartment types 1-6 (Plusshus, 2018).

Figure A.6: Apartment types 7-10 (Plusshus, 2018).

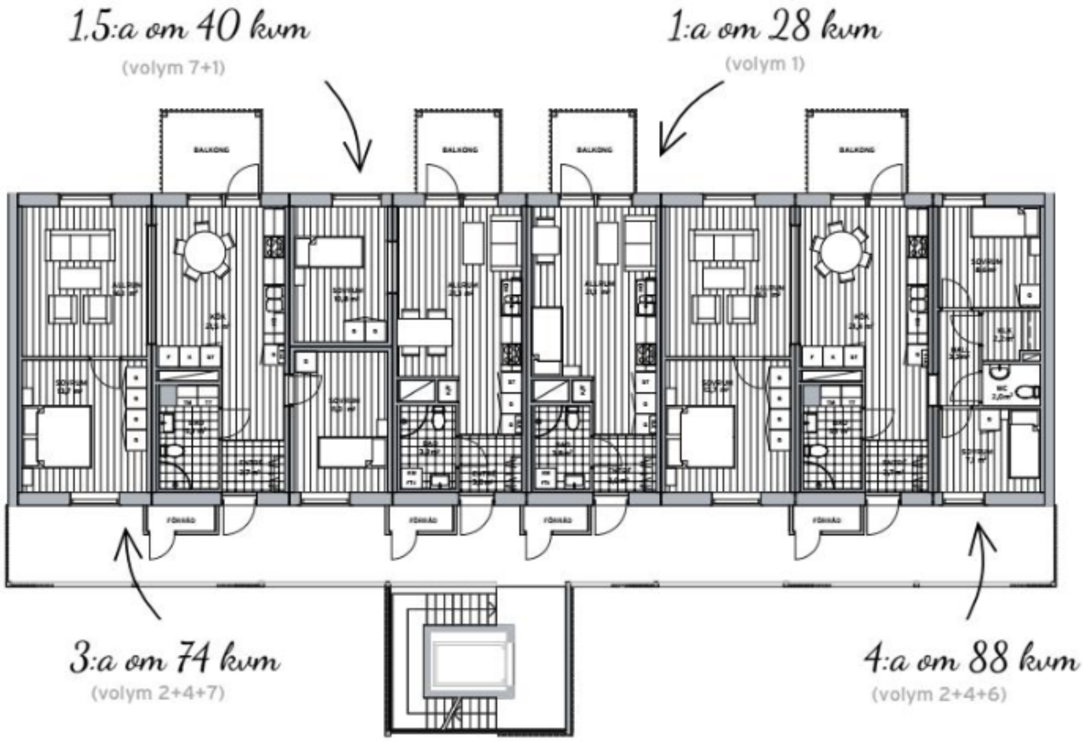


Figure A.7: Example of combined modules (Plusshus, 2018).

A.3 Investigated building

The investigated example building is a construction with a 1+4 storey structure. It means that the first floor is made of concrete and all the floors above are made of timber. In the calculations for this report, the first concrete floor will be neglected and the investigated building will be a four storey timber building. The example building illustrates how Derome chooses to place the modules and the stabilisation of the walls can be investigated for the actual building. The results in the investigation can later be implemented in the concept of AdderaPluss.

A.3.1 Overall information

As shown in the figure A.8, the reference building is a point block building with corridors in between. Both structures are based on fourteen normal-sized modules and two smaller modules that are located at the ends of the row of modules. A greater plan picture with the naming of all the shear walls can be found at the end of this chapter, in figure A.15.

The section-view of the building is presented in figure A.9. All the modules are 3 meters high, the roof structure is 5.7 meters and the total height of the four-storey building is 17.70 meters. For the analysis of the higher buildings, the same dimensions are used, only the height of the building is changing.

Figure A.8: Floor plan of the investigated building.

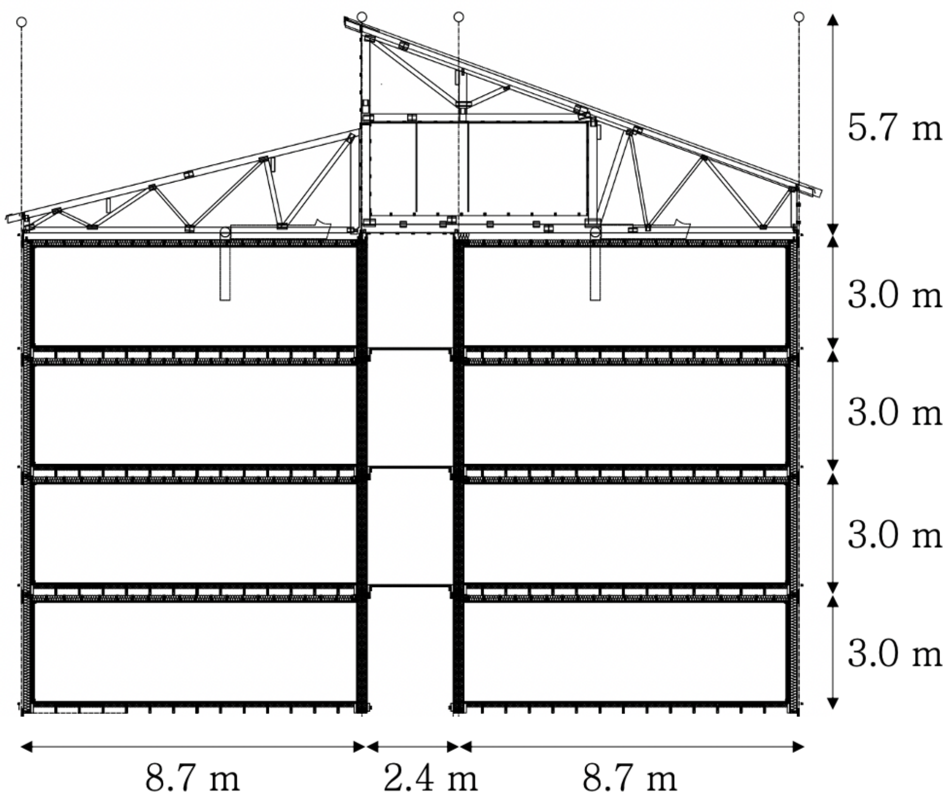


Figure A.9: Section plan of the investigated building.

In figure A.8, it is shown which type of module are placed where (V1, V2, V4, V5 or V7) and in the table A.1 the weight of the the different types of modules are presented. The values the characteristic values of the weights and they are provided by Derome.

Table A.1: The weights of the different modules used in the investigated building, provided by Derome. Characteristic values.

Type of module	Weight (kg)
V1	9790
V2	7858
V4	11030
V5	6702
V7	7173

A.3.2 The design of the walls

The thick black lines in figure A.8 represent the stabilising system of the building. It consists of all the outer walls of every module. The outer walls for every individual module are constructed in the same way with the same layers. The modules are put on top or next to each other and the short side of a module will be a part of the long side of the buildings and the long side of the module will be parallel to the short side of the building, see figure A.8. After the assembling of the modules a facade, containing a climate layer, insulation and a timber framework with OSB (oriented strand board), is attached. This facade on outer walls of the building is a part of the stabilising system.

Depending on the direction of the wind load, different walls carry the load down to the foundation. If the wind load acts on the short side of the building the marked walls in yellow in figure A.10 will act as stabilising system. The openings for windows and doors will be taken into account. The short walls 2 and 5 are neglected when determining the capacity of the shear walls because of their shortness compared to walls 1, 3, 4 and 6.

If the wind load acts on the long side of the building, see figure A.11, the transverse walls will act as a stabilising system.

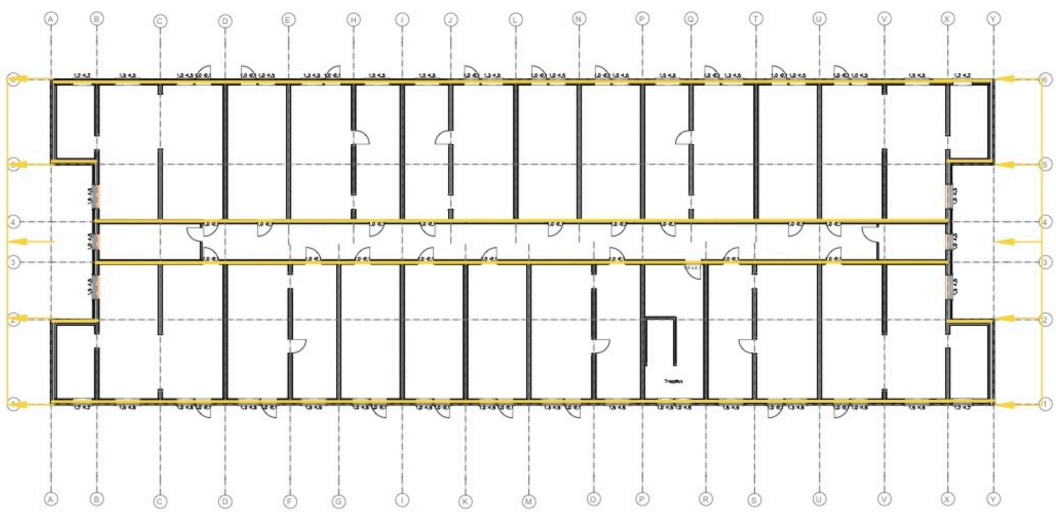


Figure A.10: Stabilising walls if wind load acts on the short side of the building.

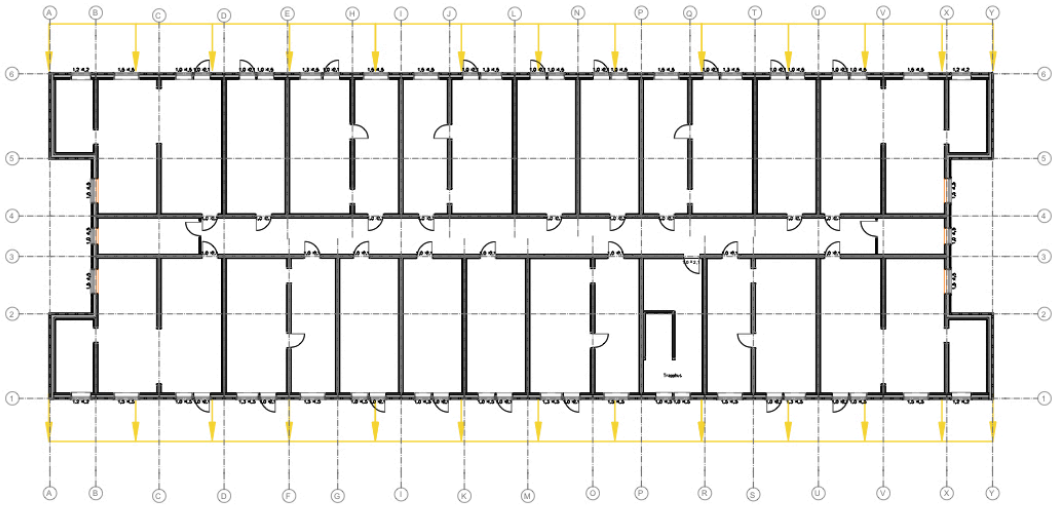


Figure A.11: Wind load acts on the long side of the building.

The two combined structures do not act together, and the wind load cannot be transferred from one structure to the other. In figure A.12, it is shown that only one of the structures is carrying the pressure load and the other structure is carrying the tension load created from the wind load when acting on one of the long sides of the building. The distribution of the load in the shear walls within the modules depends on the wall stiffness. A wall with greater stiffness is assumed to carry more load than a wall with less stiffness.

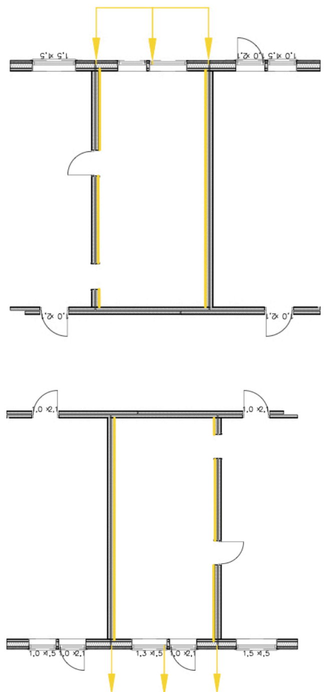
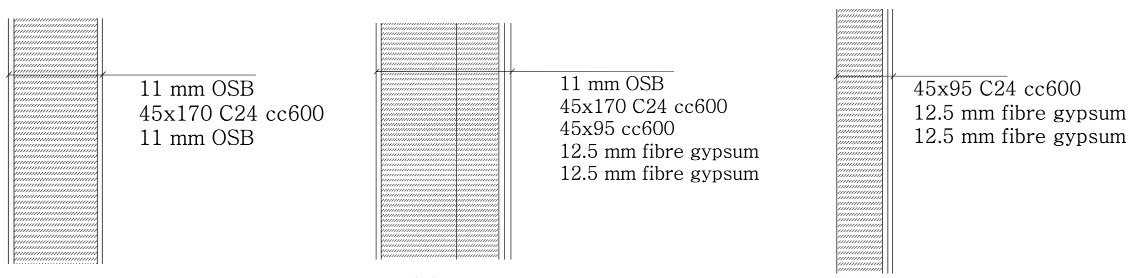


Figure A.12: How the modules in the two structures carry the wind load, when the wind load acts on one of the long sides of the building.

There are three different setups of walls in the stabilising system, as shown in table A.2 and as illustrated in figure A.13. Figure A.14 shows the location of the different wall set-ups in the building. The outer walls in the longitudinal direction of the building are constructed with two OSB (marked orange in figure A.14, that include walls number 1, 2, 5 and 6. The outer walls in the transverse direction of the building (marked pink in figure A.14) are constructed with one OSB and two fibre gypsum boards, which include parts of walls A, B, X and Y. The rest of the walls are constructed with two fibre gypsum boards (marked green in figure A.14). An exception is the inner walls adjacent to the corridor, walls 3 and 4. They are constructed with four fibre gypsum boards, to enclose the insulation in the walls.

Table A.2: The set-up of the different walls (only load-bearing elements named).

Outer wall, long sides	**Outer wall, short sides**	**Inner wall**
11 mm OSB	11 mm OSB	12.5 mm fibre gypsum
45x170 C24 cc 600	45x170 C24 cc 600	12.5 mm fibre gypsum
11 mm OSB	45x95 cc 600	45x95 C24 cc 600
	2x12.5 mm fibre gypsum	



(a) Outer walls, longitudinal. *(b) Outer walls, transverse.* *(c) Inner walls.*

Figure A.13: Wall set-ups.

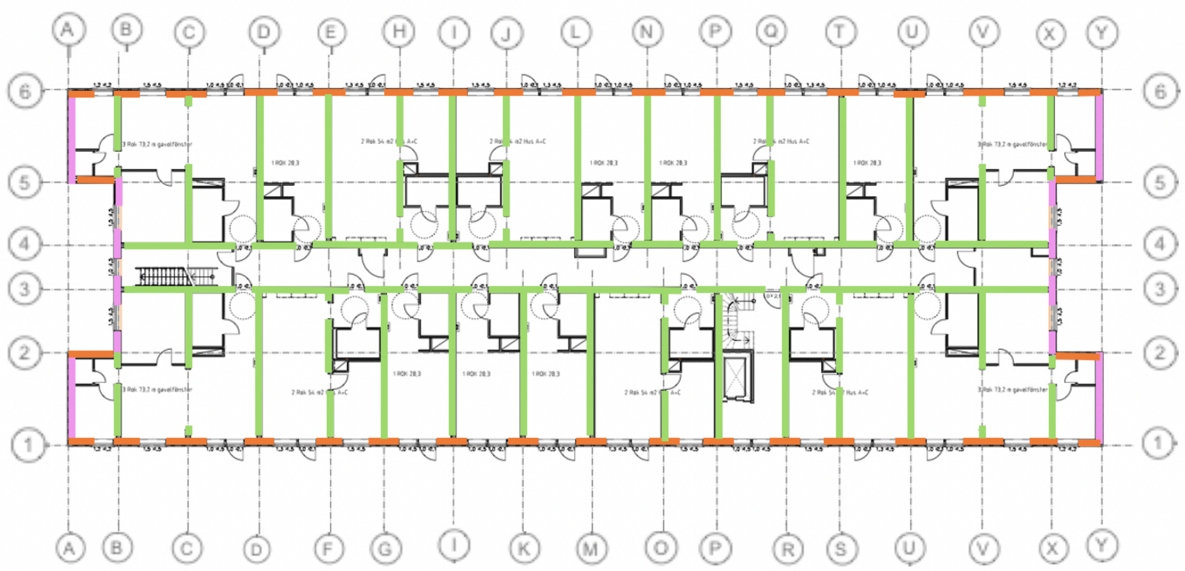


Figure A.14: The materials of the different walls. Orange = OSB, Pink = OSB and Fibre gypsum, Green = Fibre gypsum.

The boards, in the different setups of walls, are in all cases fastened with staples. The same type of staples are used for both materials of boards but for the different materials of boards, the staples have different shear capacity. These are presented in table A.3. The values are provided by Derome. The distances of the outer and inner staples are also presented in table A.3.

Table A.3: The shear capacity of the staples for the different boards (Seldert, 2020).

	Denotation	**OSB**	**Fibre gypsum**
Shear capacity (kN)	$F_{f,Rd}$	0.555	0.517
Distance of outer staples (m)	s	0.08	0.08
Distance of inner staples (m)	u	0.16	0.16

Another difference between the boards is that the fibre gypsum boards are glued together in the joints. The capacity for these walls can, therefore, be calculated with the assumption that it is one long wall without any breaks. For the OSB this is not the case. For these walls, the number of boards have to be taken into account. Boards that are shorter than $h/4$ can according to EC5-1-1 (2009) not be considered when determining the capacity. The dimensions of the OSB-boards Derome use is *2400 x 3000 mm*. If one segment of a wall containing OSB is bigger than 2.4 or 4.8 meters the rest of the wall has to be investigated to make sure it is not less than $h/4$. If that is the case the effective width of the wall is determined. If a wall segment is less than 2.4 meters, it is assumed that only one board is used.

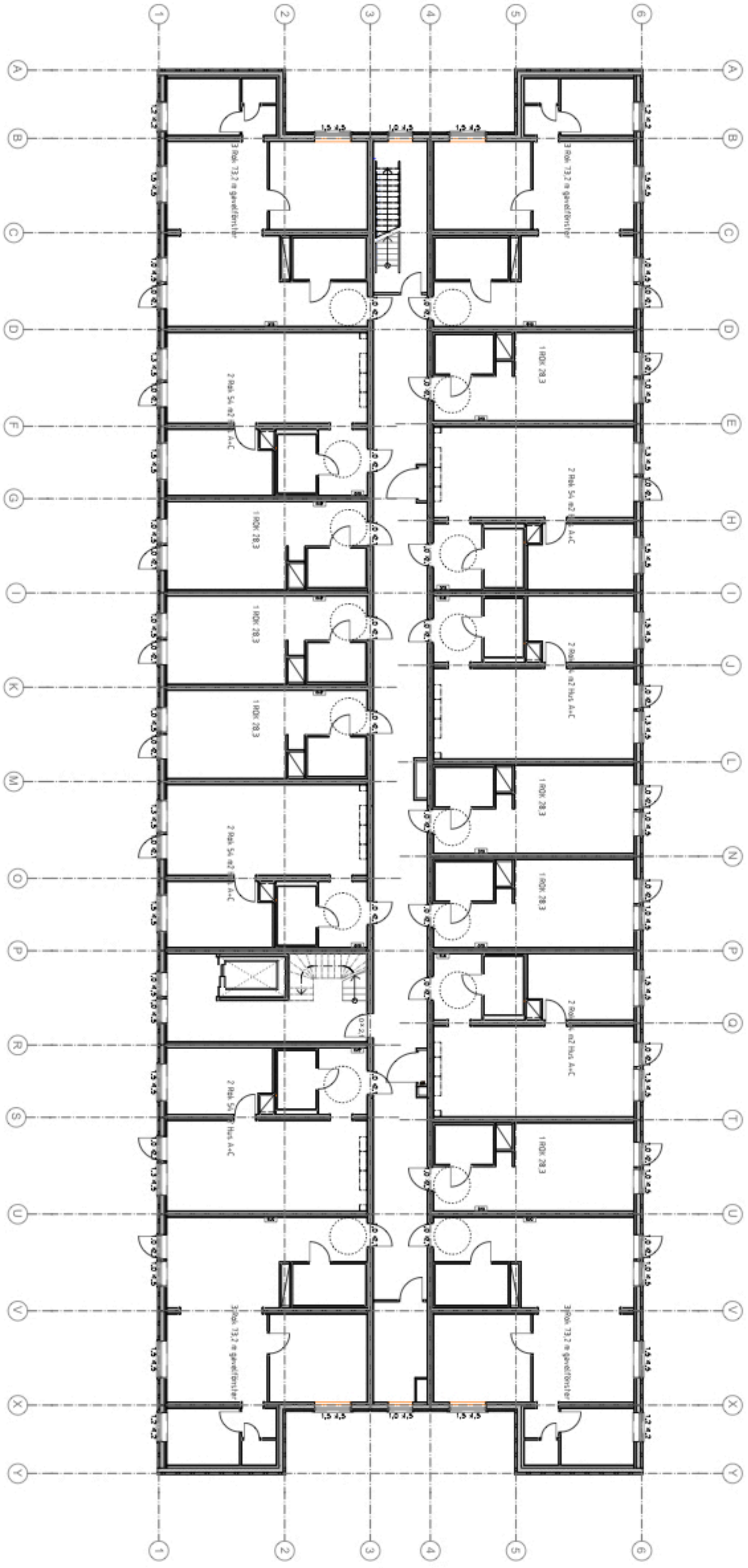


Figure A.15: Floor plan of the investigated building with naming of stabilising walls.

Appendix B

Wind load

The wind load acting on the building is calculated according to EC1-1-4 (2008) and EKS 11 (2019). When analysing the exposure of wind load for a building, only the external wind loads have to be taken into account. The internal wind loads do only matter for the design of individual elements. The wind load is expressed as load per square meter.

The wind load is simplified to be a static load for this report. Since this report only focuses on the horizontal loads, the variation of wind load along the height of the building is of interest. One wind load for every floor and for the the roof is calculated. The wind load of interest is the accumulated wind load, distributed from the top of the building down to 1.5 m (half the height of a module) above ground. The wind load acting on the lower half of the lowest module will be carried by the foundation directly. The accumulated wind load will be compared to the capacity of the walls of the first floor in the building.

The external wind load is calculated as:

$$w_e = q_p(z_e) \cdot c_{pe} \tag{B.1}$$

Where:

$q_p(z_e)$	is the peak velocity pressure;
z_e	is the reference height for the external pressure;
c_{pe}	is the pressure coefficient for the external pressure.

The velocity pressure q_p is based on a reference velocity v_b. The reference velocity describes the wind circumstances for different regions (Isaksson et al., 2017).

According to EKS 11 (2019) the peak velocity pressure $q_p(z)$ can be designed as:

$$q_p(z_e) = (1 + 2 \cdot k_p \cdot I_v(z)) \cdot (k_r \cdot ln(\frac{z}{z_0}) \cdot c_0(z))^2 \cdot q_b \tag{B.2}$$

Where:

k_p	is the peak factor, $k_p = 3$ according to EKS 11 (2019);
$I_v(z)$	is the turbulence intensity;
k_r	is the terrain factor;
z_0	is the roughness length;
c_0	is the orography factor, $c_0 = 1$ according to EC1-1-4 (2008);
q_b	is the basic velocity pressure.

The turbulence intensity $I_v(z)$ is designed according to EKS 11 (2019):

$$I_v(z) = \frac{1}{c_0(z) \cdot ln(\frac{z}{z_0})} \tag{B.3}$$

The terrain factor k_r is designed according to EC1-1-4 (2008).

$$k_r = 0.19(\frac{z_0}{z_{0,II}})^{0.07} \tag{B.4}$$

Where $z_{0,II} = 0.05$ m, according to EC1-1-4 (2008). The roughness length, z_0, can be found in figure B.1.

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

Figure B.1: Values for z_0 and z_{min} depending on terrain type (EC1-1-4, 2008). Used with permission.

The basic velocity pressure, q_b, is designed according to EC1-1-4 (2008):

$$q_b = \frac{1}{2} \cdot \rho \cdot v_b^2 \tag{B.5}$$

Where ρ is the air density expected in the region during windstorms. ρ is dependent on altitude, temperature and barometric pressure. The recommended value for the density is 1.25 kg/m^3 (EKS 11, 2019). v_b can be found in figure C-4 in EKS 11 (2019). v_b is the basic wind velocity and it is dependent on the location of the building. Since this project is a cooperation with Derome AB in Varberg, the value $v_b = 25$ m/s will be used.

The pressure coefficients for external pressure, c_{pe}, can be divided into two different cases. $c_{pe,1}$ is intended for small areas and fixings less than 1 m^2 and $c_{pe,10}$ is a global pressure coefficient for loaded areas of 10 m^2. $c_{pe,10}$ is used for the design of load-bearing structure of a building and will further on be used in this report. The pressure coefficients for the different areas of the building can be decided by figures B.2 and B.3 (EC1-1-4, 2008).

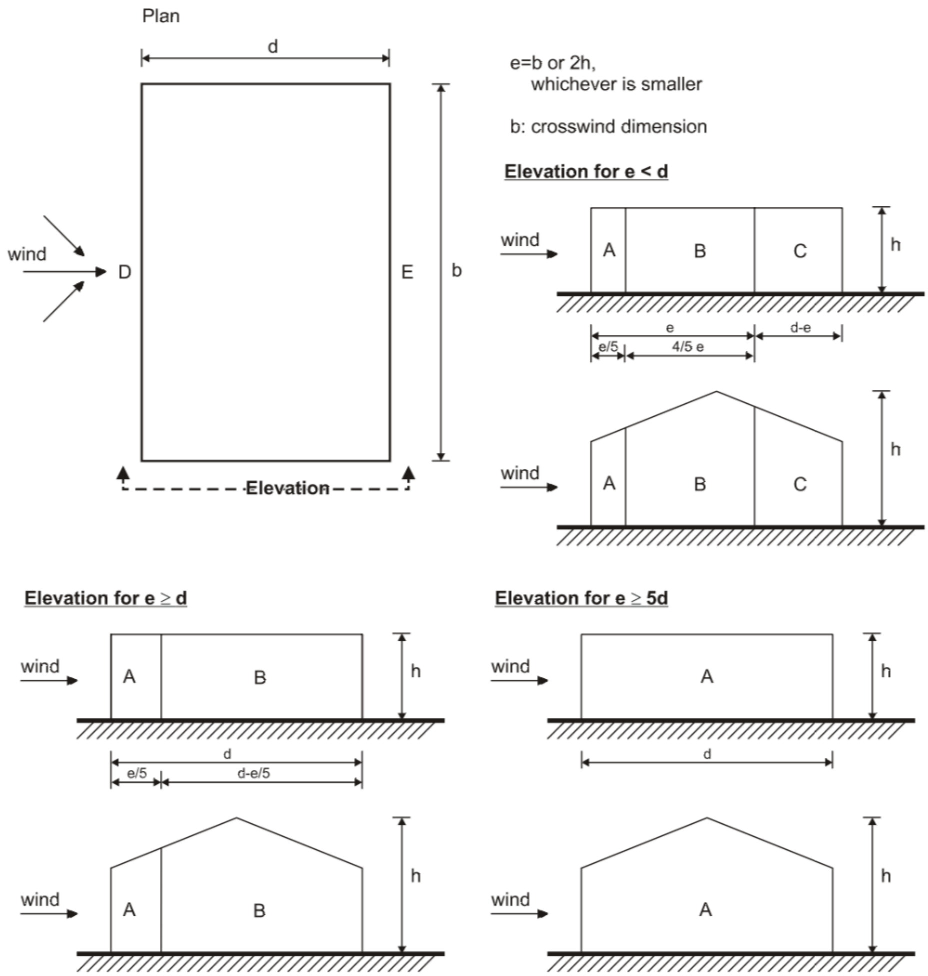


Figure B.2: Key for zone division and pressure coefficients for vertical walls (EC1-1-4, 2008). Used with permission.

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

Figure B.3: Recommended values for external pressure coefficients for vertical walls (EC1-1-4, 2008). Used with permission.

In figure B.3, h/d is the total height of the building divided by the length of the side of the building that is parallel to the wind load. If this value turns out to be something in between the noted values, a linear interpolation is done. The value d is the width of the building (19.83 m) for the case when the wind load acts on the long side and when the wind load acts on the short side of the building d equals the length of the building (57.58 m).

The length for the different wind load areas at the wall located parallel to the wind load, are decided according to figure B.2 and stated below:

$$L_A = e/5$$

$$L_B = e \cdot 4/5$$

$$L_C = L - L_A - L_B$$

If the real length of the building (L) is less than e, L_B is the remaining length of the building.

The pressure coefficients for zone D and E are the most interesting for the investigation of shear walls. When the wind load is acting on the short side of the building, the coefficients for zone D and E are added together, but when the wind load is acting on the long side of the building only the coefficient for the most adverse zone of D and E is used. This is because the two structures are not connected in a way which will make them work together. The wind load is therefore calculated for pressure and tension separately for this case.

Wind load for zone D is:

$$w_{D,pressure} = q_p \cdot c_{pe,D} \tag{B.6}$$

Wind load for zone E is:

$$w_{E,tension} = q_p \cdot c_{pe,E} \tag{B.7}$$

In addition to the wind load acting on the walls of the building, there will also be a wind load acting on the roof. The roof is a duo-pitch roof and the pressure coefficients for the different areas of the roof can be decided by figures B.4 and B.5. Figure B.5 only gives the pressure coefficients for the different zones on the roof for when the wind load is acting on the long side of the building, the pressure coefficients for when the wind load is acting on the gable is presented in figure C.1.

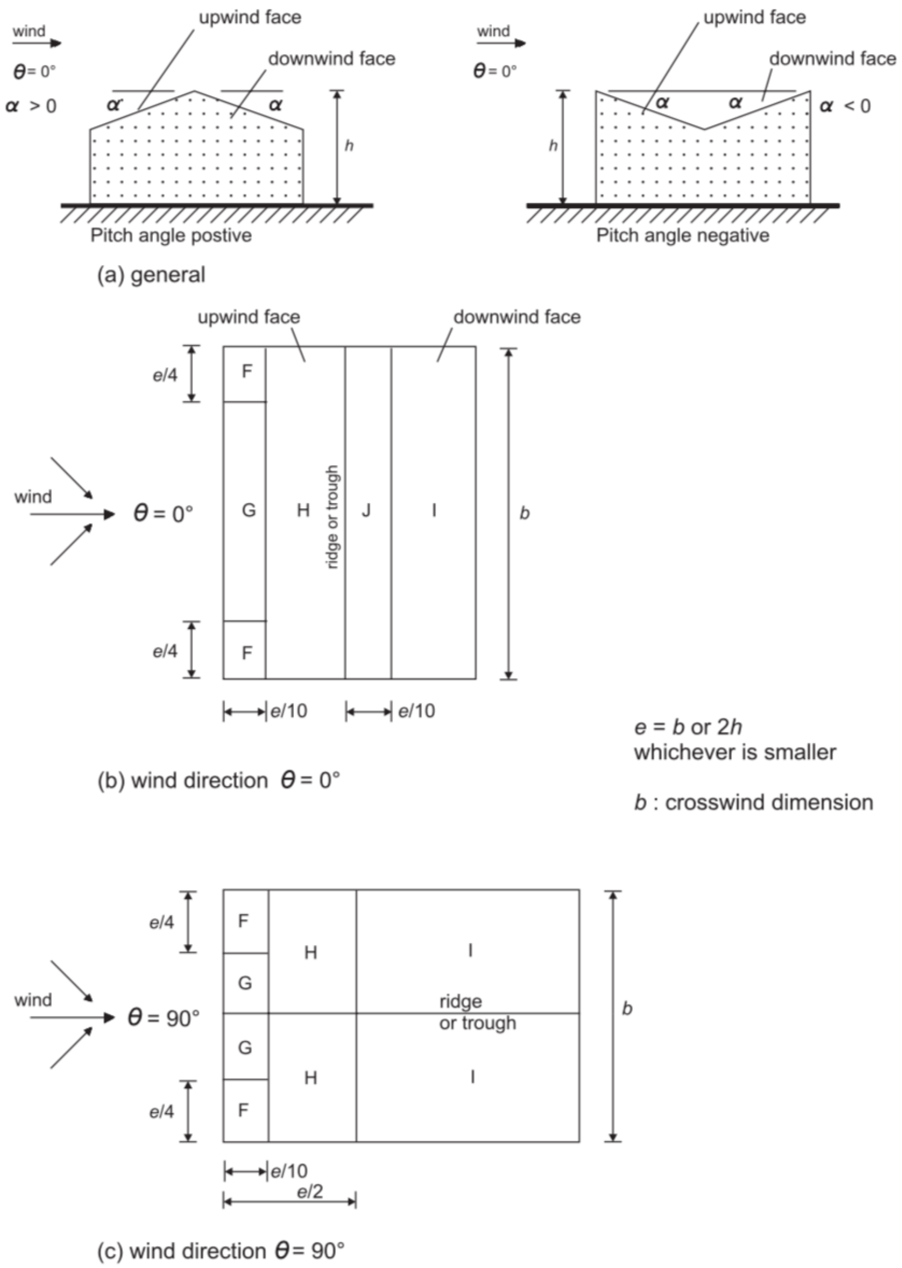


Figure B.4: Key for zone division and pressure coefficients for duo-pitch roofs (EC1-1-4, 2008). Used with permission.

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

NOTE 1 At θ = 0° the pressure changes rapidly between positive and negative values on the windward face around a pitch angle of α = -5° to +45°, so both positive and negative values are given. For those roofs, four cases should be considered where the largest or smallest values of all areas F, G and H are combined with the largest or smallest values in areas I and J. No mixing of positive and negative values is allowed on the same face.

NOTE 2 Linear interpolation for intermediate pitch angles of the same sign may be used between values of the same sign. (Do not interpolate between α = +5° and α = -5°, but use the data for flat roofs in 7.2.3). The values equal to 0,0 are given for interpolation purposes

Figure B.5: Recommended values for external pressure coefficients for duo-pitch roofs (EC1-1-4, 2008). Used with permission.

As mentioned in NOTE 1 in figure B.5, four cases should be considered when deciding the worst case for how the wind load is distributed over the roof of the building. Depending on whether the wind load is acting on wall 1 or wall 6, different cases need to be considered. No mixing of positive and negative values on the same face are allowed. One $c_{pe,10}$ is determined for each zone (F, G, H, I and J) in figure B.5 for the four different cases and the lengths of the zones are designed according to figure B.4. To determine the wind pressure for the different zones on the roof, equation B.1 is used and these loads acts perpendicular to the surface of the roof. Only the horizontal vectors of these loads are of interest when designing the shear walls.

The wind load is assumed to be the leading action and the characteristic wind load is therefore multiplied with the factor 1.5 to obtain the design load, the load combination 6.10b is used (Isaksson et al., 2017). The wind load for the different storeys are multiplied with the influence height for every storey respectively. The design load acting at the top of every storey is then designed as:

$$Q_{Ed,pressure} = 1.5 \cdot h_{influence} \cdot w_{E,pressure} \tag{B.8}$$

$$Q_{eq:Ed,tension} = 1.5 \cdot h_{influence} \cdot w_{E,tension} \tag{B.9}$$

The wind load of interest for this report is the accumulated wind load, acting from 1.5 meters above the ground to the top of the building. The design wind loads (Q_{Ed}) for every storey are added together with the load acting on the roof and named H_{Ed}. The capacity of the walls on the first floor of the building is required to be greater than this load.

$$H_{Ed,4floors} = \frac{1}{2}Q_{Ed,floor1} + Q_{Ed,floor2} + Q_{Ed,floor3} + Q_{Ed,floor4} + Q_{Ed,roof} \tag{B.10}$$

B.1 Calculations of wind load

In the following section, there will be a detailed presentation of the calculated wind load. First, a more detailed analysis for a four-storey building and then a less thorough analysis for six- and eight-story buildings since the same approach is used for all building heights.

The terrain category is chosen to be category number II, and according to figure B.1 this provides the values $z_0 = 0.05$ m and $z_{min} = 2$ m. The reference velocity is chosen to be 25 m/s. The distribution of the wind pressure is calculated according to equation B.2.

In the following table, the calculation progress is presented and final values for q_p are presented for all storeys.

Table B.1: Calculation progress of q_p at vertical walls.

Storey	**z$_e$ (m)**	**z$_0$ (m)**	**z$_{min}$ (m)**	**I$_v$**	**k$_r$**	**q$_b$ (N/m^2)**	**q$_p$ (kN/m^2)**
H_8	24.00	0.05	2.00	0.16	0.19	390.63	1.06
H_7	21.00	0.05	2.00	0.17	0.19	390.63	1.03
H_6	18.00	0.05	2.00	0.17	0.19	390.63	0.99
H_5	15.00	0.05	2.00	0.18	0.19	390.63	0.94
H_4	12.00	0.05	2.00	0.18	0.19	390.63	0.89
H_3	9.00	0.05	2.00	0.19	0.19	390.63	0.82
H_2	6.00	0.05	2.00	0.21	0.19	390.63	0.73
H_1	3.00	0.05	2.00	0.24	0.19	390.63	0.58
H_0	2.00	0.05	2.00	0.27	0.19	390.63	0.50

The same calculations are done for the heights of the roof tops for the different buildings. The roof structure itself is 5.70 m high and this height is added to the height of the different buildings. For example:

$$z_{e,Roof4} = 12 + 5.70 = 17.70\ m$$

Table B.2: Calculation progress of q_p at roof tops for buildings with 4, 6 and 8 storeys.

Roof	**z$_e$ (m)**	**z$_0$ (m)**	**z$_{min}$ (m)**	**I$_v$**	**k$_r$**	**q$_b$ (N/m^2)**	**q$_p$ (kN/m^2)**
$Roof_8$	29.70	0.05	2.00	0.16	0.19	390.63	1.12
$Roof_6$	23.70	0.05	2.00	0.16	0.19	390.63	1.06
$Roof_4$	17.70	0.05	2.00	0.16	0.19	390.63	0.98

The designed values for the wind pressure according to EC1-1-4 (2008) and EKS 11 (2019) at the vertical walls are compared to values taken from table 2.7 in Isaksson et al. (2017). These tables are made to simplify the design of wind loads. Intermediate values in these tables have been interpolated. The results for the different methods and the heights of every storey are presented in table B.3.

Table B.3: Wind pressure. Comparison of calculated values according to EKS 11 (2019) and values taken from table 2.7 in Isaksson et al. (2017).

Storey	**Height (m)**	**Calculated values (kN/m^2)**	**Values from table 2.7 in Isaksson et al. (2017) (kN/m^2)**
H_8	24	1.06	1.06
H_7	21	1.03	1.02
H_6	18	0.99	0.99
H_5	15	0.94	0.94
H_4	12	0.89	0.89
H_3	9	0.82	0.82
H_2	6	0.73	0.72
H_1	3	0.58	0.57
H_0	0	0.50	0.50

As can be seen in table B.3, similar results have been found for the two methods and the values calculated according to EKS 11 (2019) will further on be used because of the distribution of the wind pressure shown in figure B.6. The wind pressure changes over the height of the building as a non-linear function, and therefore it is more accurate to use the values according to EKS 11 (2019), than to linearly interpolate between the values given in Isaksson et al. (2017).

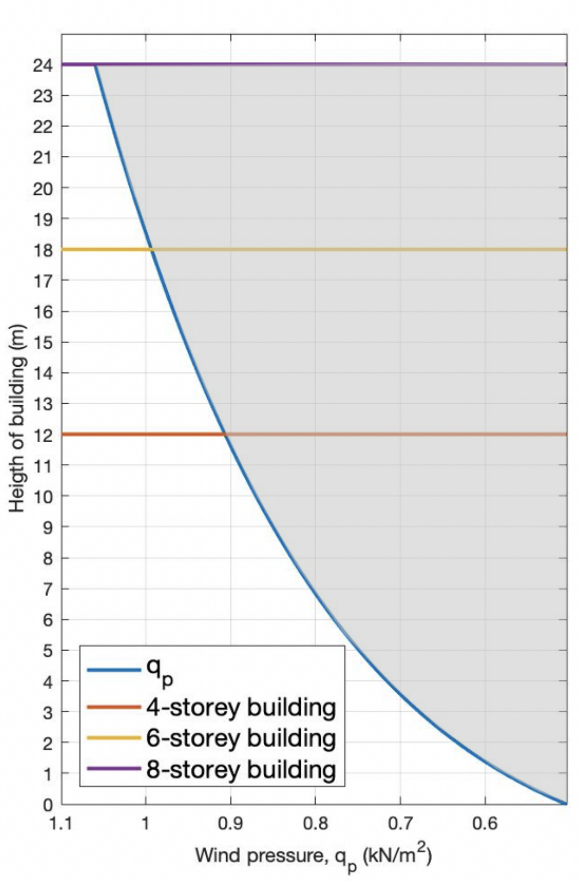


Figure B.6: Wind pressure for different heights. The three different storeys of buildings are marked. The grey area is the wind load.

With MATLAB, the integrals in figure B.6 from 1.5 meters over the ground to the top of the building can be calculated and this is the load that the lowest wall in the building will have to carry. The rest of the load, distributed from the ground to 1.5 m above the ground, is carried directly by the foundation. A comparison is done for the calculated values of the integral in MATLAB with the values contained according to EKS 11 (2019). As a simplification, the influence area is equally split over the height of the building, which is not completely true since the variation of wind pressure is non-linear between two floors, which can be seen in figure B.6. The results are presented in table B.4.

Table B.4: Comparison of summarized wind pressure acting on the lowest wall in the building for the three different heights of buildings.

	Equally split area (kN/m) (EKS11)	**Integral by MATLAB (kN/m)**
4-storey	7.72	7.67
6-storey	13.35	13.31
8-storey	19.50	19.46

As shown in table B.4, similar values are obtained and since the values designed with equally split influence area is this close to the results by MATLAB, these values will further on be used when comparing the loads with the capacity of the shear walls, partly because the EKS11 values are on the safe side and also because it is an assured simplification.

B.1.1 Detailed calculation of wind pressure for a height of 12 meters

The detailed calculations for the wind pressure at height 12 meters are presented below. The turbulence intensity is calculated according to equation B.3 and for $c_0 = 1$, $z = 12$ and $z_0 = 0.05$ the intensity is:

$$I_v(z) = \frac{1}{c_0(z) \cdot ln(\frac{z}{z_0})} = \frac{1}{1 \cdot ln(\frac{12}{0.05})} = 0.18 = 18\%$$

The turbulence intensity changes over the height of the building but both the terrain factor and the basic velocity pressure are constant over the height of the building.

The terrain factor k_r is calculated according to equation B.4, where $z_0 = 0.05$ and $z_{0,II} = 0.05$.

$$k_r = 0.19(\frac{z_0}{z_{0,II}})^{0.07} = 0.19(\frac{0.05}{0.05})^{0.07} = 0.19$$

The basic velocity pressure is calculated according to equation B.5.

$$q_b = \frac{1}{2} \cdot \rho \cdot v_b^2 = \frac{1}{2} \cdot 1.25 \cdot 25^2 = 391\ N/m^2$$

The wind pressure can now be calculated according to equation B.2.

$$q_p(z_e) = (1 + 2 \cdot k_p \cdot I_v(z)) \cdot (k_r \cdot ln(\frac{z}{z_0}) \cdot c_0(z))^2 \cdot q_b =$$
$$(1 + 2 \cdot 3 \cdot 0.18) \cdot (0.19 \cdot ln(\frac{12}{0.05}) \cdot 1)^2 \cdot \frac{391}{1000} = 0.89\ kN/m^2$$

B.1.2 Four-storey building

The pressure coefficients are depending on the height of the building and on which side of the building the wind load is acting.

B.1.2.1 Wind load on the long sides of the building

Firstly, the design for wind load acting on the long sides of the building will be presented. The values and lengths for the pressure coefficients are determined and presented in table B.5, where e equals the minimum of $2h = 35.4$ m or $L = 57.58$ m, which gives $e = 35.4$ m, $d = B = 19.83$ m and $h = 17.70$ m.

Table B.5: Pressure coefficients with respective lengths for vertical walls.

	e (m)	**d (m)**	**h/d**	**c$_{pe,10,A}$**	**L$_A$ (m)**	**c$_{pe,10,B}$**	**L$_B$ (m)**	**c$_{pe,10,}$**	**L$_C$ (m)**	**c$_{pe,10,D}$**	**c$_{pe,10,E}$**
4-storey	35.40	19.83	0.89	-1.20	7.08	-0.80	12.75	-0.50	0.00	0.79	-0.47

The pressure coefficients are multiplied with the corresponding wind pressure q_p, see equation B.1. When designing the wind load on the long side of the building, the load is divided into two cases, pressure and tension. This is because only one of the structures is carrying the pressure load and the other the tension load. For example:

$$w_{e,pressure,H_4} = q_{p,H_4} \cdot c_{pe,10,D,H_4} = 0.89 \cdot 0.79 = 0.70\ kN/m^2$$

Thereafter, the design load is obtained by multiplying w_e with the influence height of the load and the safety factor 1.5, see equations B.8 and B.9. It is assumed that the top floor has a loaded height of 2.0 meters. This represents half of the height of the top module and another 0.5 meters from the roof structure before the roof starts to incline. The design wind load at height H_4:

$$Q_{d,pressure} = w_{e,pressure} \cdot 2 \cdot 1.5 = 0.7 \cdot 2 \cdot 1.5 = 2.09\ kN/m$$

The accumulated load acting on the bottom floor is marked in bold in tables B.6 and B.7 below. Note that the values in table B.6 are compression values and the values in table B.7 are tension values (suction). For example, the accumulated pressure load is calculated as:

$$H_{d,pressure,H_1} = 2.09 + 2.90 + 2.57 + 2.06 = 9.62\ kN/m$$

Table B.6: Design pressure wind load for vertical walls.

Storey	**Loaded height**	$\mathbf{w}_{e,pressure}$ **(kN/m^2)**	$\mathbf{Q}_{d,pressure}$ **(kN/m)**	$\mathbf{H}_{d,pressure}$ **(kN/m)**
H_4	2.00	0.70	2.09	2.09
H_3	3.00	0.64	2.90	4.99
H_2	3.00	0.57	2.57	7.56
H_1	3.00	0.46	2.06	**9.62**
H_0	1.50	0.40	0.89	10.52

Table B.7: Design tension wind load for vertical walls.

Storey	**Loaded height**	$\mathbf{w}_{e,tension}$ **(kN/m^2)**	$\mathbf{Q}_{d,tension}$ **(kN/m)**	$\mathbf{H}_{d,tension}$ **(kN/m)**
H_4	2.00	-0.42	1.25	1.25
H_3	3.00	-0.39	1.74	2.99
H_2	3.00	-0.34	1.54	4.54
H_1	3.00	-0.27	1.24	**5.77**
H_0	1.50	-0.24	0.53	6.31

After designing the wind load acting on the walls of the building, the wind load acting on the inclined roof is designed. As mentioned in figure B.5, four cases for the pressure coefficients needs to be considered. Depending on what side the wind load acts on, different cases are found to be the worst case.

Firstly, the case when the wind load is acting on wall 1 is presented, see figure B.7. For this case, the worst distribution of the wind load acting on the roof is when there is a pressure load on the roof of structure 1 and an up-lifting load on structure 2. The horizontal resultants of these loads are added to the load acting on the shear walls on the upper floor. The vertical resultants creating an up-lifting load need to be secured in the connection between the top module and the second top module from the top, this will be discussed further in Appendix C.

The second case, when the wind load acts on wall 6, the worst case is as shown in figure B.8, where there is a pressure load on the roof of structure 2 and an up-lifting load on structure 1. The horizontal resultants of the load acting on the roof are added to the wind load acting on the vertical walls.

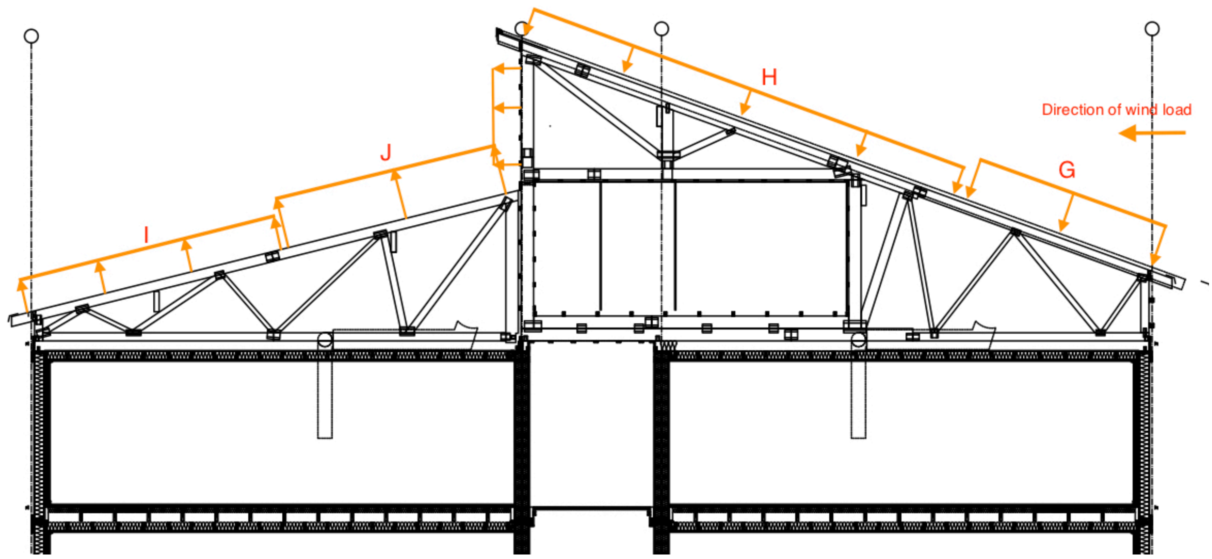


Figure B.7: Worst case for pressure coefficients when the wind load acts on wall 1.

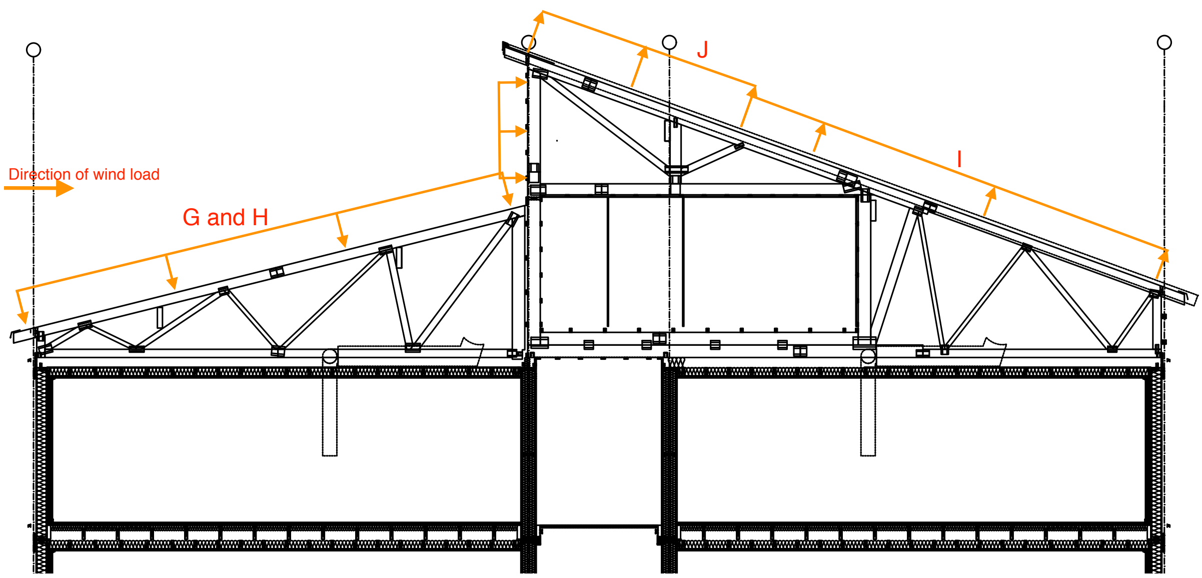


Figure B.8: Worst case for pressure coefficients when the wind load acts on wall 6.

The values for the pressure coefficients on the roof are presented in table B.8 together with the lengths for every coefficient respectively.

Table B.8: Pressure coefficients with respective lengths for the roof.

	$c_{pe,10,F}$	W_F (m)	$c_{pe,10,G}$	L_G (m)	$c_{pe,10,H}$	L_H (m)	$c_{pe,10,I}$	L_I (m)	$c_{pe,10,J}$	L_J (m)
Wind at wall 1	0.37	8.85	0.37	3.54	0.27	7.57	-0.42	5.13	-0.96	3.54
Wind at wall 6	0.18	8.85	0.18	3.54	0.18	5.13	-0.4	7.57	-0.83	3.54

The pressure coefficients are multiplied with the wind pressure acting at the top of the roof and summarised in table B.9. For example:

$$w_{e,G,wall\,1} = c_{pe,10,G,wall\,1} \cdot q_{p,roof\,4} = 0.37 \cdot 0.98 = 0.36\ kN/m^2$$

$$w_{e,G,wall\,6} = c_{pe,10,G,wall\,6} \cdot q_{p,roof\,4} = 0.18 \cdot 0.98 = 0.18\ kN/m^2$$

Table B.9: Characteristic wind load in the different zones. Note that the zones change over the roof depending on which side the wind load is acting on.

	F	G	H	I	J
w_e, wind at wall 1 (kN/m^2)	0.36	0.36	0.26	-0.41	-0.94
w_e, wind at wall 6 (kN/m^2)	0.18	0.18	0.18	-0.39	-0.82

The wind load acting at the vertical zone of the roof structure is calculated in the same way as for the vertical walls. When the wind load is acting on wall 1 there will be a tension load acting at this part and if the wind load acts on wall 6, there will be a pressure load. In table B.10 the values for the vertical part of the roof are presented for a four-storey building. For example, when the wind load is acting on wall number 1:

$$h/d = 17.7/19.83 = 0.89$$

$$c_{pe,E} = -0.47\ (from\ figure\ B.5)$$

$$w_e = c_{pe,E} \cdot q_{p,roof\,4} = -0.47 \cdot 0.98 = -0.46\ kN/m^2$$

Table B.10: Wind load at vertical part of roof.

	h/d	$c_{pe,E}$	$w_{e,vertical}$ (kN/m^2)
Wind at wall 1	0.89	-0.47	-0.46
Wind at wall 6	0.89	0.79	0.77

The horizontal resultants of the loads acting on the roof are added together, depending on if it is a pressure or tension load. The additional horizontal loads from the roof structure are presented in table B.11.

Table B.11: Additional characteristic horizontal loads from the roof structure.

	Total horizontal pressure load (kN/m^2)	**Total horizontal tension load (kN/m^2)**
Wind at wall 1	2.24	-1.32
Wind at wall 6	0.37	-3.89

The final horizontal load acting on the lowest floor of the building is presented in table B.12. The design load is different depending on which side of the building the wind load is acting because of the asymmetry of the roof. As shown in table B.12, the pressure load is always the design value and the shear walls in structure 1 need to be designed for a load of 12.99 kN/m and the shear walls in structure 2 needs to be designed for 10.81 kN/m.

Table B.12: Design wind load acting on the shear walls at floor 1.

	$H_{d,pressure}$ (kN/m)	**$H_{d,tension}$ (kN/m)**
Wind at wall 1	**12.99**	7.75
Wind at wall 6	**10.18**	11.61

An example how the values in table B.12 are calculated is shown below.

$$H_{d,pressure,wall1} = 9.62 + 1.5 \cdot 2.24 = 12.99 \; kN/m$$

Where the value 9.62 is taken from table B.6 and the value 2.24 is taken from table B.11. Since simplicity is one of the key concepts for "AdderaPluss", all the walls are supposed to look the same and therefore also resist the same loads. All the walls should therefore be designed to resist the maximum load, 13 kN/m.

B.1.2.2 Wind load on short side of building

When the wind load is acting on the short side of the building almost the same calculations are done. The walls to resist the wind load acting on the short side of the building are the longitudinal walls. The difference between the longitudinal walls compared to the transverse walls, is that the longitudinal walls go all the way through the building and as a result, the walls have to resist both pressure and tension loads, see figure A.10. In table B.13, the pressure coefficients with respective lengths are presented, where e equals the minimum of $2h = 35.4$ or $B = 19.83$, which gives $e = 19.83$ m, $d = L = 57.58$ m and $h = 17.70$ m.

Table B.13: Pressure coefficients for all zones with respective lengths.

	e (m)	**d (m)**	**h/d**	**$c_{pe,10,A}$**	**L_A (m)**	**$c_{pe,10,B}$**	**L_B (m)**	**$c_{pe,10,}$**	**L_C (m)**	**$c_{pe,10,D}$**	**$c_{pe,10,E}$**
4-storey	19.83	57.58	0.31	-1.20	3.97	-0.80	15.87	-0.50	37.75	0.71	-0.32

The influence height for the top shear walls are, for this case, 3.82 meters. This includes 1.5 meters that is half the top module height and the equivalent height of the area of influence from the roof structure at the gable, see figure B.9.

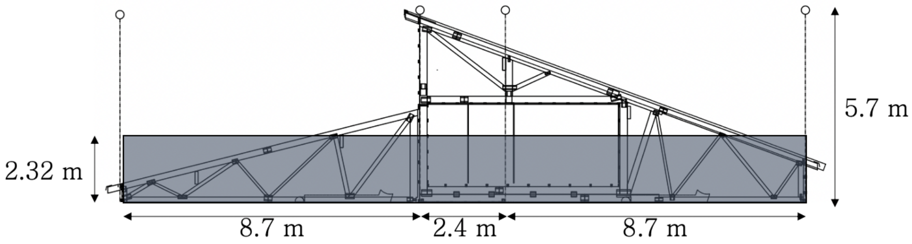


Figure B.9: Equivalent height of triangular area of influence.

The results for the characteristic wind loads, design wind loads, and the accumulated wind load are presented in table B.14. The total load of 15 kN is the pressure and tension loads added together.

Table B.14: Results of design wind load acting on the short side of the building.

Storey	**Loaded height (m)**	$\mathbf{w}_{e,pressure}$ **(kN/m²)**	$\mathbf{Q}_{d,pressure}$ **(kN/m)**	$\mathbf{H}_{d,pressure}$ **(kN/m)**		$\mathbf{w}_{e,tension}$ **(kN/m²)**	$\mathbf{Q}_{d,tension}$ **(kN/m)**	$\mathbf{H}_{d,tension}$ **(kN/m)**	**Total load (kN/m)**
H_4	3.82	0.63	3.59	3.59		-0.28	1.60	1.60	
H_3	3.00	0.58	2.61	6.20		-0.26	1.16	2.76	
H_2	3.00	0.52	2.32	8.52		-0.23	1.03	3.80	
H_1	3.00	0.41	1.86	**10.38**		-0.18	0.83	**4.62**	**15.00**
H_0	1.50	0.36	0.80	11.18		-0.16	0.36	4.98	

As shown in table B.14, the total design wind load acting on the lowest modules at the short side of the building is 15 kN/m. Thus, the walls in the longitudinal direction of the building are required to resist this load.

When the wind load acts on the short side of the building, it is creating up-lifting forces over the whole roof structure. These loads will not be taken into account when designing the wind loads affecting the shear walls. This is because this load is less than the load acting on the shear walls when the wind load is acting on the long sides of the building. However, an analysis of whether this up-lifting load is greater than the self-weight of the top modules will be done in appendix C.

B.1.3 Six-storey building

A similar design is done for a six-storey building. The total height of this building is 23.7 meters. The pressure and tensions forces created on every storey is presented in table B.15, where $H_{d,pressure}$ and $H_{d,tension}$ are the accumulated loads. The additional horizontal loads from the roof structure are presented in table B.16. When the wind load is acting on the long side of the building, the loads presented in table B.17 will be the design loads for the shear walls in the transverse direction of the building.

When the wind load is acting on the short side of the building the values for the design wind load are presented in table B.18, containing both the loads on the vertical walls and the loads on the roof. The total load for the walls in the longitudinal direction of the building to resist is 25 kN/m, see table B.18.

Table B.15: Pressure and tension loads acting on each floor of a six-storey building.

Storey	Loaded height (m)	$w_{e,pressure}$ (kN/m^2)	$Q_{d,pressure}$ (kN/m)	$H_{d,pressure}$ (kN/m)		$w_{e,tension}$ (kN/m^2)	$Q_{d,tension}$ (kN/m)	$H_{d,tension}$ (kN/m)
H_6	2.00	0.79	2.37	2.37		-0.50	1.51	1.51
H_5	3.00	0.75	3.39	5.76		-0.48	2.16	3.67
H_4	3.00	0.71	3.19	8.95		-0.45	2.04	5.70
H_3	3.00	0.66	2.95	11.90		-0.42	1.88	7.58
H_2	3.00	0.58	2.62	14.52		-0.37	1.67	9.25
H_1	3.00	0.47	2.10	**16.62**		-0.30	1.34	**10.59**
H_0	1.50	0.40	0.91	17.53		-0.26	0.58	11.17

Table B.16: Additional characteristic horizontal loads from the roof structure.

	Total horizontal pressure load (kN/m^2)	Total horizontal tension load (kN/m^2)
Wind at wall 1	2.66	-1.58
Wind at wall 6	0.40	-4.41

Table B.17: Design wind load acting on the shear walls at floor 1.

	$H_{d,pressure}$ (kN/m)	$H_{d,tension}$ (kN/m)
Wind at wall 1	**20.62**	12.97
Wind at wall 6	**17.22**	17.21

Table B.18: Results of design wind load acting on the short side of the building.

Storey	Loaded height (m)	$w_{e,pressure}$ (kN/m^2)	$Q_{d,pressure}$ (kN/m)	$H_{d,pressure}$ (kN/m)		$w_{e,tension}$ (kN/m^2)	$Q_{d,tension}$ (kN/m)	$H_{d,tension}$ (kN/m)	Total load (kN/m)
H_6	3.82	0.71	4.07	4.07		-0.34	1.94	1.94	
H_5	3.00	0.68	3.06	7.13		-0.32	1.45	3.39	
H_4	3.00	0.64	2.88	10.01		-0.30	1.37	4.76	
H_3	3.00	0.59	2.66	12.67		-0.28	1.27	6.03	
H_2	3.00	0.53	2.36	15.04		-0.25	1.12	7.15	
H_1	3.00	0.42	1.89	**16.93**		-0.20	0.90	**8.05**	**24.98**
H_0	1.50	0.36	0.82	17.75		-0.17	0.39	8.44	

B.1.4 Eight-storey building

For a building consisting of eight storeys, the total height is 29.7 meters. The pressure and tensions forces created on every storey is presented in table B.15, where $H_{d,pressure}$ and $H_{d,tension}$ are the accumulated loads. The additional horizontal loads from the roof structure are presented in table B.20. In table B.21 the results for wind load acting on the long side of the building are presented, containing both the loads from the vertical walls and the roof.

In table B.22 the results for the design wind load when acting on the short side of the building are presented. The total load for the walls in the longitudinal direction of the building to resist is 36.5 kN/m, see table B.22

Table B.19: Pressure and tension loads acting on each floor of an eight-storey building.

Storey	Loaded height (m)	$w_{e,pressure}$ (kN/m^2)	$Q_{d,pressure}$ (kN/m)	$H_{d,pressure}$ (kN/m)		$w_{e,tension}$ (kN/m^2)	$Q_{d,tension}$ (kN/m)	$H_{d,tension}$ (kN/m)
H_8	2.00	0.85	2.54	2.54		-0.56	1.67	1.67
H_7	3.00	0.82	3.69	6.24		-0.54	2.42	4.09
H_6	3.00	0.79	3.55	9.79		-0.52	2.33	6.42
H_5	3.00	0.75	3.39	13.18		-0.49	2.22	8.64
H_4	3.00	0.71	3.19	16.37		-0.47	2.10	10.74
H_3	3.00	0.66	2.95	19.32		-0.43	1.94	12.68
H_2	3.00	0.58	2.62	21.94		-0.38	1.72	14.40
H_1	3.00	0.47	2.10	**24.04**		-0.31	1.38	**15.77**
H_0	1.50	0.40	0.91	24.95		-0.26	0.60	16.37

Table B.20: Additional characteristic horizontal loads from the roof structure.

	Total horizontal pressure load (kN/m^2)	Total horizontal tension load (kN/m^2)
Wind at wall 1	3.07	-1.82
Wind at wall 6	0.42	-4.82

Table B.21: Design wind load acting on the shear walls at floor 1.

	$H_{d,pressure}$ (kN/m)	$H_{d,tension}$ (kN/m)
Wind at wall 1	**28.65**	18.51
Wind at wall 6	**24.67**	23.01

Table B.22: Results of design wind load acting on the short side of the building.

Storey	Loaded height (m)	$w_{e,pressure}$ (kN/m^2)	$Q_{d,pressure}$ (kN/m)	$H_{d,pressure}$ (kN/m)		$w_{e,tension}$ (kN/m^2)	$Q_{d,tension}$ (kN/m)	$H_{d,tension}$ (kN/m)	Total load (kN/m)
H_8	3.82	0.78	4.46	4.46		-0.39	2.25	2.25	
H_7	3.00	0.75	3.39	7.85		-0.38	1.71	3.96	
H_6	3.00	0.73	3.27	11.12		-0.37	1.65	5.61	
H_5	3.00	0.69	3.12	14.24		-0.35	1.57	7.18	
H_4	3.00	0.65	2.94	17.17		-0.33	1.48	8.66	
H_3	3.00	0.60	2.71	19.88		-0.30	1.37	10.03	
H_2	3.00	0.54	2.41	22.29		-0.27	1.22	11.24	
H_1	3.00	0.43	1.93	**24.22**		-0.22	0.97	**12.22**	**36.44**
H_0	1.50	0.37	0.83	25.06		-0.19	0.42	12.64	

Appendix C

Up-lifting loads on roof structure

As mentioned in Appendix B, the vertical resultants of the wind load on the roof need to be analysed to know if (and in that case how much) the top modules need to be secured. The wind load creates uplifting forces on the roof, both for when the wind load is acting on the long side and on the short side of the building. When comparing figures C.1-C.2 showing the values and locations for the pressure coefficients when the wind load is acting on the long side of the building, and figures B.4-B.5, showing the values and locations for the pressure coefficients when the wind load is acting on the gable, it appears that more uplifting forces will be created when the wind load is acting on the gable, therefore, only this case is further investigated.

From the pressure coefficients for the different zones on the roof given in figure C.1, the values between the given pitch angels are interpolated and the results are presented in table C.1. The zones and the outspread of the different coefficients are shown in figure C.2.

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

Figure C.1: Recommended values for external pressure coefficients for duo-pitch roofs. Wind load acting on gable (EC1-1-4, 2008). Used with permission.

Table C.1: Pressure coefficients ($c_{pe,10}$) for the different zones for both parts of the roof, where the inclination is 14° and 20°.

	F	**G**	**H**	**I**
20°	-1.23	-1.33	-0.67	-0.50
14°	-1.33	-1.30	-0.61	-0.51

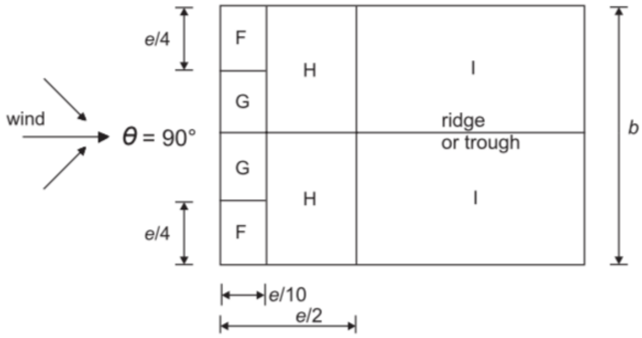


Figure C.2: The outspread of the different zones when the wind load is acting on the gable (EC1-1-4, 2008). Used with permission.

For the investigated building, the following values are obtained:

$e = b$	19.83 m;
$e/2$	9.92 m;
$e/4$	4.96 m;
$e/10$	1.98 m;

As shown in table C.1 and figure C.2, the worst up-lifting load is in the corner of the building, closest to the gable where the wind load is acting. Therefore, the three outermost modules are analysed. Figure C.3 shows the modules and all the dimensions necessary. The width of the three modules together is 10.56 m. It is assumed that the outermost module is in zone F and the other two modules are in zone H, as a simplification. Zone G is neglected because of the floor plan of the building where there are no modules at the location for zone G. The self-weights of the three outermost modules are presented in table C.2.

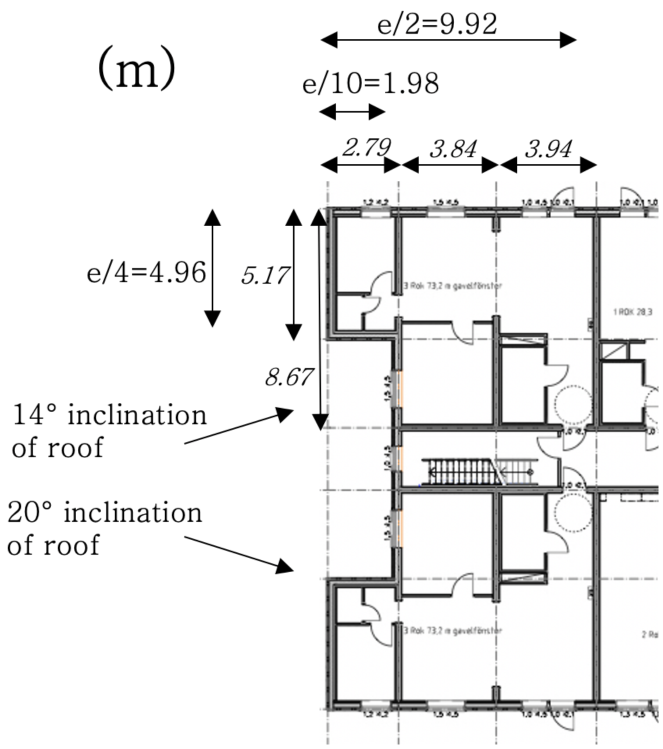


Figure C.3: The three outermost modules that are analysed for uplifting forces.

Table C.2: The weights of the modules most vulnerable for uplift.

Module	**Weight (kg)**
Module between Wall A-B and X-Y	3350
Module between Wall B-C and V-X	7860
Module between Wall C-D and U-V	11 030

Firstly, an eight-storey building is analysed. If the top modules have enough self-weight to resist the up-lifting load the top modules for a six- and four-storey building will also resist the up-lifting loads since the wind load reduces for the lower building heights. In tables C.3 and C.4, the uplifting forces acting on the three modules respectively are presented together with the resisting force, the self-weight. To the right in the tables, the differences between the forces are presented. A positive number indicates that the self-weight is greater than the up-lifting load. In the fifth column, the utilisation of the self-weight is presented for an eight-storey building. A value lower than 1 indicates that there is no risk for up-lift of the top modules. As shown in the tables, the highest utilisation can be found in the module between wall A/B in both structure 1 and 2.

Table C.3: Comparison of up-lifting force and self-weight for modules in structure 1 (roof inclination is 20°).

Structure 1 (20°)	**Up-lifting force (kN)**	**Self-weight (kN)**	**Difference (kN)**	**Ratio**
Module C/D	-45.87	110.30	64.43	0.42
Module B/C	-44.71	78.58	33.87	0.57
Module A/B	-27.92	33.51	5.59	0.83

Table C.4: Comparison of up-lifting force and self-weight for modules in structure 2 (roof inclination is 14°).

Structure 2 (14°)	Up-lifting force (kN)	Self-weight (kN)	Difference	Ratio
Module C/D	-33.83	110.30	76.47	0.31
Module B/C	-32.97	78.58	45.61	0.42
Module A/B	-31.09	33.51	2.42	0.93

Two examples of how the values in tables C.3 and C.4 are found are presented below. The first example is for the module between wall C and D in structure 1 where the inclination of the roof is 20°.

$$U_d = (c_{pe,10,H,20} \cdot q_p) \cdot cos(\phi) \cdot 1.5 \cdot w \cdot L$$
$$= (-0.67 \cdot 1.12) \cdot cos(20) \cdot 1.5 \cdot 3.94 \cdot (8.668 + 2.438) = -45.87\ kN$$

Where the length of the distributed load (8.668 + 2.438) is the length of the module together with the width of the corridor between the two structures since the roof on this structure reaches from wall 1 to wall 4. $q_p = 1.12\ kN/m^2$ according to table B.2 in appendix B.

The self-weight of the module is:

$$G_d = 11030 \cdot 1.0 \cdot 10 \cdot 10^{-3} = 110.30\ kN$$

Since $G_d - U_d > 0$ there is no risk for up-lift of the top module.

The second example is for the module between wall A and B in structure 2 where the inclination of the roof is 14°.

$$U_d = (c_{pe,10,F,14} \cdot q_p) \cdot cos(\phi) \cdot 1.5 \cdot w \cdot L$$
$$= (-1.33 \cdot 1.12) \cdot cos(14) \cdot 1.5 \cdot 2.786 \cdot 5.167 = -31.09\ kN$$

The self-weight of the module is:

$$G_d = 3350 \cdot 1.0 \cdot 10 \cdot 10^{-3} = 33.51\ kN$$

Since $G_d - U_d > 0$ there is no risk for up-lift of the top module.

As shown in tables C.3 and C.4, the outermost module (module A/B) has the lowest margin and should maybe be secured to the underlying modules. On the other hand, the self-weight of the roof structure is not taken into account and will increase the difference and thus add some safety against up-lifting.

Appendix D

Capacity of shear walls according to Method A

For the design of the capacity of the shear walls according to Method A (EC5-1-1, 2009), the following values are required:

$F_{f,Rd,OSB} = 0.555$ kN;
$F_{f,Rd,fibre} = 0.517$ kN;
$h_{boards} = 3$ m;
$h/4 = 0.75$ m;
$b_0 = h/2 = 1.5$ m;
$s_{OSB} = 0.08$ m;
$s_{fibre} = 0.08$ m;

The lengths of all the shear walls are measured in drawings obtained from Derome and the walls are named according to figure A.15. As mentioned in chapter 3, the segments of the walls made of OSB have to fulfil the requirement that the width of the segment must be greater the one-fourth of the height. The segments less than this width are neglected for the calculation of the capacity.

When the wind load is acting on the short side of the building, all the walls in the longitudinal direction of the building will carry both pressure and tension loads. Tables D.1 -D.4 show the capacity calculations for the walls in the longitudinal direction of the building. The calculations for the segment of wall 1 between walls A and B are presented below:

There are two segments of this part of the wall, the segments are 1.08 m and 0.28 m respectively. The segment of 0.28 m is less than $h/4 = 0.75$ m and is therefore not included in the calculations, the effective width for this section is then: $b_i = 1.08m$. The factor c_i is calculated according to equation 3.11 and for this case where $b_i = 1.08 < b_0 = 1.5$, $c_i = \frac{b_i}{b_0} = \frac{1.08}{1.5} = 0.72$.

Thereafter, the capacity for this segment is calculated according to equation 3.10. The material in this part of the wall is OSB and $F_{f,Rd} = 0.555\ kN$. Factor 1.2 is used according to EC5-1-1 (2009).

$$F_{i,v,Rd} = \frac{F_{f,Rd} \cdot b_i \cdot c_i \cdot 1.2}{s} = \frac{0.555 \cdot 1.08 \cdot 0.72 \cdot 1.2}{0.08} = 6.47\ kN$$

All the segments of wall 1 are calculated in the same way and the segments that are not included are written in italics and underlined. The capacities for each module are added together and then multiplied with a factor 2 since there are two OSB sheets in wall 1. The total capacity of wall 1 is:

$$F_{v,Rd} = 250\ kN$$

The same calculations are done for walls number 3, 4 and 6. For walls number 3 and 4, $F_{f,Rd} = 0.517\ kN$ since these walls are made with fibre gypsum boards and the capacities for each module is also multiplied with a factor 2 since there are fibre gypsum on both sides of the insulation in these walls.

Table D.1: Capacity of wall 1

Wall 1	**A/B**		**B/C**		**C/D**		**D/F**		**F/G**		**G/I**		**I/K**		**K/M**		**M/O**		**O/P**		**P/R**		**R/S**		**S/U**		**U/V**		**V/X**		**X/Y**	
b_i (m)	1.08	*0.28*	1.11	1.23	1.01	0.83	0.77	0.77	*0.67*	0.77	0.82	0.91	0.82	0.91	0.82	0.91	0.77	0.77	*0.67*	0.77	0.87	0.87	0.77	*0.67*	0.77	0.77	0.83	1.01	1.23	1.12	*0.28*	1.08
c_i	0.72		0.74	0.82	0.67	0.55	0.51	0.51		0.51	0.55	0.61	0.55	0.61	0.55	0.61	0.51	0.51		0.51	0.58	0.58	0.51		0.51	0.51	0.55	0.67	0.82	0.74		0.72
Capacity per segment (kN)	6.47	0.00	6.89	8.33	5.62	3.80	3.27	3.25	0.00	3.28	3.77	4.60	3.77	4.60	3.77	4.60	3.27	3.25	0.00	3.28	4.20	4.20	3.28	0.00	3.25	3.27	3.80	5.63	8.33	6.90	0.00	6.47
Capacity per module (kN)	6.47		15.22		9.41		6.52		3.28		8.36		8.36		8.36		6.52		3.28		8.40		3.28		6.52		9.42		15.23		6.47	
TOT CAP (kN)	**250.28**																															

Table D.2: Capacity of wall 3

Wall 3	**B/C**	**C/D**		**D/F**	**F/G**		**G/I**		**I/K**		**K/M**		**M/O**	**O/P**		**P/R**		**R/S**		**S/U**	**U/V**		**V/X**
b_i (m)	3.84	2.60	*0.34*	3.94	0.90	1.03	0.92	1.91	0.92	1.91	0.92	1.91	3.94	0.90	1.03	2.56	*0.27*	1.03	0.90	3.94	*0.34*	2.59	3.84
c_i	1.00	1.00		1.00	0.60	0.69	0.61	1.00	0.61	1.00	0.61	1.00	1.00	0.60	0.68	1.00		0.69	0.60	1.00		1.00	1.00
Capacity per segment	29.78	20.19	0.00	30.55	4.21	5.46	4.39	14.80	4.39	14.80	4.39	14.80	30.55	4.21	5.43	19.85	0.00	5.46	4.21	30.55	0.00	20.07	29.78
Capacity per module	59.56	40.37		61.11	19.34		38.38		38.38		38.38		61.11	19.28		39.71		19.34		61.11	40.14		59.56
TOT CAP (kN)	**595.77**																						

Table D.3: Capacity of wall 4

Wall 4	**B/C**	**C/D**		**D/E**		**E/H**	**H/I**		**I/J**		**J/L**	**L/N**		**N/P**		**P/Q**		**Q/T**	**T/U**		**U/V**		**V/X**
b_i (m)	3.84	2.59	*0.34*	1.91	0.92	3.94	0.90	1.03	1.03	0.90	3.94	1.91	0.92	1.91	0.92	1.03	0.90	3.94	1.91	0.92	*0.34*	2.59	3.84
c_i	1.00	1.00		1.00	0.61	1.00	0.60	0.69	0.69	0.60	1.00	1.00	0.61	1.00	0.61	0.69	0.60	1.00	1.00	0.61		1.00	1.00
Capacity per segment	29.78	20.07	0.00	14.80	4.39	30.55	4.21	5.47	5.47	4.21	30.55	14.80	4.39	14.80	4.39	5.46	4.21	30.55	14.80	4.39	0.00	20.07	29.78
Capacity per module	59.56	40.14		38.38		61.11	19.36		19.36		61.11	38.38		38.38		19.34		61.11	38.38		40.14		59.56
TOT CAP (kN)	**594.32**																						

Table D.4: Capacity of wall 6

Wall 6	**A/B**		**B/C**		**C/D**		**D/E**		**E/H**		**H/I**		**I/J**		**J/L**		**L/N**		**N/P**		**P/Q**		**Q/T**		**T/U**		**U/V**		**V/X**		**X/Y**	
b_i (m)	1.08	*0.28*	1.11	1.23	1.01	0.83	0.91	0.82	0.77	0.77	*0.67*	0.77	0.77	*0.67*	0.77	0.77	0.91	0.82	0.91	0.82	0.77	*0.67*	0.77	0.77	0.91	0.82	0.83	1.01	1.23	1.11	*0.28*	1.08
c_i	0.72		0.74	0.82	0.67	0.55	0.61	0.55	0.51	0.51		0.51	0.51		0.51	0.51	0.61	0.55	0.61	0.55	0.51		0.51	0.51	0.61	0.55	0.55	0.67	0.82	0.74		0.72
Capacity per segment	6.47	0.00	6.89	8.33	5.63	3.80	4.60	3.77	3.27	3.25	0.00	3.28	3.28	0.00	3.25	3.27	4.60	3.77	4.60	3.77	3.28	0.00	3.25	3.27	4.60	3.77	3.79	5.62	8.33	6.89	0.00	6.47
Capacity per module	6.47		15.22		9.42		8.36		6.52		3.28		3.28		6.52		8.36		8.36		3.28		6.52		8.36		9.40		15.22		6.47	
TOT CAP (kN)	**250.15**																															

Similar calculations are done for the walls in the transverse direction of the building. When the wind load is acting on the long side of the building, one of the two structures carries the pressure load and the other structure carries the tension load that is created. Since the pressure load is greater than the tension load, all the walls will be designed for the pressure load. Because of the asymmetric roof, depending on whether the wind load is acting on wall 1 or wall 6, there will be different design loads for the different structures.

Calculations for the part of wall C that is between walls 1 and 3 are presented below. This part is divided into two different segments because of the opening in the wall. The first segment is 0.564 m and since this is less than $h/4 = 0.75\ m$ this segment will be neglected. The other segment is 4.408 m. Since this wall is made with fibre gypsum boards, the amount of boards is not of interest. The effective width for this wall is therefore 4.408 m.

The factor c_i is calculated according to equation 3.11 and for this case where $b_i = 4.408 > b_0 = 1.5$, $c_i = 1$.

The capacity can thereafter be calculated with equation 3.10. The shear capacity in the staples are $F_{f,Rd} = 0.517\ kN$, since the wall is made with fibre gypsum boards. Factor 1.2 is used according to EC5-1-1 (2009).

$$F_{i,v,Rd} = \frac{F_{f,Rd} \cdot b_i \cdot c_i \cdot 1.2}{s} = \frac{0.517 \cdot 4.408 \cdot 1 \cdot 1.2}{0.08} = 34.18\ kN$$

As seen in table D.7, the same capacity is designed for the other segment that is located between wall 4 and wall 6. When the same transverse wall goes from wall 1 to wall 6, it is always the same capacity in the two structures.

The capacity for all the walls in the transverse direction of the building are presented in tables D.5-D.10. The total capacity mentioned in the tables are for one segment and will be compared to the pressure wind load acting on the long sides of the building.

Table D.5: Capacity of walls A and Y.

Wall A/Y	**1/2**	**5/6**
b_i (m)	5.167	5.167
b_{eff}	4.80	4.80
c_i	1.00	1.00
Capacity per segment	77.18	77.18
Capacity per module	77.18	77.18
TOT CAP (kN)	**77.18**	

Table D.6: Capacity of walls B and X.

Wall B/X	**1/2**		**2/3**		**3/4**		**4/5**		**5/6**	
b_i (m)	3.082	*0.632*	1.45	0.771	*0.719*	*0.717*	0.771	1.455	*0.637*	3.078
c_i	1.00		0.97	0.51			0.51	0.97		1.00
Capacity per segment	23.90	0.00	22.54	6.37	0.00	0.00	6.37	22.69	0.00	23.87
Capacity per module	23.90		28.91		0.00		29.07		23.87	
TOT CAP (kN)	**52.81**									

Table D.7: Capacity of walls C and V.

Wall C/V	**1/3**		**4/6**	
b_i (m)	*0.564*	4.408	4.408	*0.565*
c_i		1.00	1.00	
Capacity per segment	0.00	34.18	34.18	0.00
Capacity per module	34.18		34.18	
TOT CAP (kN)	**34.18**			

Table D.8: Capacity of walls D, I and P.

Wall D/I/P	**1/3**	**4/6**
b_i (m)	8.668	8.667
c_i	1.00	1.00
Capacity per segment	67.22	67.21
Capacity per module	67.22	67.21
TOT CAP (kN)	**67.21**	

Table D.9: Capacity of walls E, G, K, L, M, N, R, T and U

Wall E/G/K/L/M/N/R/T/U	**4/6**
b_i (m)	8.667
c_i	1.00
Capacity per segment	67.21
Capacity per module (kN)	67.21
TOT CAP (kN)	**67.21**

Table D.10: Capacity of walls F, H, J, O, Q and S

Wall F/H/J/O/Q/S	**1/3**		
b_i (m)	2.745	3.033	*0.6*
c_i	1.00	1.00	
Capacity per segment	21.29	23.52	0.00
Capacity per module	44.81		
TOT CAP (kN)	**44.81**		

As a final analysis, the risk for shear buckling of the panel is investigated. According to EC5-1-1 (2009), the following statement has to be true to neglect to risk for shear buckling.

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

Where:

b_{net}	is the clear distance between studs;
t	is the thickness of a sheet.

This is tested for both OSB and fibre gypsum. The thickness of one OSB is 11 mm and the thickness for the fibre gypsum boards is 12.5 mm. There are always two fibre gypsum boards next to each other and therefore the total thickness of the fibre gypsum board is $2 \cdot 12.5 = 25\ mm$. b_{net} is the width between the studs, $b_{net} = 600$ mm for both materials.

$$\frac{b_{net}}{t_{OSB}} = \frac{600}{11} = 54.5 \leq 100$$

$$\frac{b_{net}}{t_{fibregypsum}} = \frac{600}{25} = 24 \leq 100$$

The statement is fulfilled for both materials.

Appendix E

Capacity of shear walls according to Elastic method

The elastic method according to Carling et al (1992) will be analysed on two different walls in the building to see the differences to Method A from EC5-1-1 (2009). The walls that will be analysed with the elastic method are wall 1 and wall B. Wall 1 is made of OSB and wall B is in some parts made of fibre gypsum and in some parts made of both fibre gypsum and OSB.

The shear resistance designed with the elastic method is according to Carling et al (1992) also limited by the strength of the fasteners in the wall. The shear capacity of a wall segment is designed according to equation 3.1, also shown below:

$$H_{Rd} = \frac{F_{vd}}{h\sqrt{(\frac{x_{max}}{\sum x_i^2})^2 + (\frac{y_{max}}{\sum y_i^2})^2}}$$

Where F_{vd} is the design capacity of the fasteners, $F_{vd,OSB} = 0.555$ kN and $F_{vd,fibre\,gypsum} = 0.517$ kN. h is the height of the panels, $h = 3m$, and $x_{max} = w/2$ and $y_{max} = h/2$.

Approximate expressions for $\sum x_i^2$ and $\sum y_i^2$ are taken from Carling et al (1992) and for a panel like the one in figure E.1 the expressions are:

$$\sum x_i^2 = \frac{w^2}{12}(2n + 6m) \tag{E.1}$$

$$\sum y_i^2 = \frac{h^2}{12}(6n + 2m + p - 3) \tag{E.2}$$

Where:

$$n = \frac{w}{s} \tag{E.3}$$

$$m = \frac{h}{t} \tag{E.4}$$

$$p = \frac{h}{u} \tag{E.5}$$

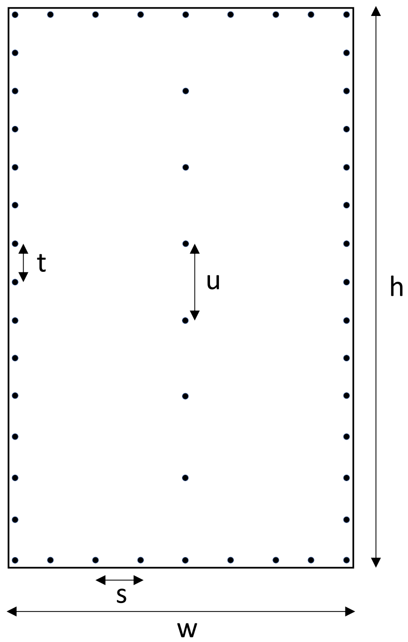


Figure E.1: Illustration of panel with denotation of the dimensions.

For a panel without the stud in the middle the expressions are:

$$\sum x_i^2 = \frac{w^2}{12}(2n + 6m) \tag{E.6}$$

$$\sum y_i^2 = \frac{h^2}{12}(6n + 2m) \tag{E.7}$$

Wall 1 contains many segments. The total width of the OSB is 2.4 m but there is no place in wall 1 where a whole panel can fit. Instead, the boards are assumed to be the same width as the segments. It is not specified in Carling et al (1992) whether to neglect small segments or not. Instead, small segments are assumed to reach between two wooden studs, without any stud in the middle.

For example, for the part of wall 1 between walls A and B, there is a segment of 1.08 m and a segment of 0.28 m. The first segment is assumed to be one panel with the width of 1.08 m reaching over three studs as shown in figure E.1 and the second segment is supposed to be 0.28 m wide, but only reaching over two studs. All segments with the width less than 0.75 m are assumed to reach over two studs instead of three. The widest segment in wall 1 is 1.23 m and is assumed to reach over three studs.

When there are many segments working together in a wall the capacities for all segments are summarised to get the total capacity of the wall. For a wall consisting of multiple panels, the capacity is multiplied with the number of panels.

An example of the design is shown below for the first segment in wall 1, where $w = 1.08$ m.

To design the values for $\sum x^2$ and $\sum y^2$ (equations E.1 and E.2), the values for n, m and p are required.

$$n = \frac{w}{s} = \frac{1.08}{0.08} = 13.50$$

$$m = \frac{h}{t} = \frac{3}{0.08} = 37.5$$

$$p = \frac{h}{u} = \frac{3}{0.16} = 18.75$$

The values for $\sum x^2$ and $\sum y^2$ are:

$$\sum x_i^2 = \frac{w^2}{12}(2n + 6m) = \frac{1.08^2}{12}(2 \cdot 13.50 + 6 \cdot 37.5) = 24.49$$

$$\sum y_i^2 = \frac{h^2}{12}(6n + 2m + p - 3) = \frac{3^2}{12}(6 \cdot 13.50 + 2 \cdot 37.50 + 18.75 - 3) = 128.81$$

The capacity for this segment is then:

$$H_{Rd} = \frac{F_{vd}}{h\sqrt{(\frac{x_{max}}{\sum x_i^2})^2 + (\frac{y_{max}}{\sum y_i^2})^2}} = \frac{0.555}{3\sqrt{(\frac{1.08/2}{24.49})^2 + (\frac{3/2}{128.81})^2}} = 7.42\ kN$$

All the capacities for the segments in wall 1 are presented in table E.1 below.

Table E.1: Capacity of wall 1 according to the elastic method.

Wall 1	**A/B**		**B/C**		**C/D**		**D/F**		**F/G**		**G/I**		**I/K**		**K/M**	
L (m)	1.08	0.28	1.11	1.23	1.01	0.83	0.77	0.77	0.67	0.77	0.82	0.91	0.82	0.91	0.82	0.91
n	13.50	3.50	13.93	15.31	12.58	10.34	9.60	9.56	8.38	9.61	10.30	11.38	10.30	11.38	10.30	11.38
$\sum x^2$	24.49	1.52	26.17	31.97	21.10	14.00	12.00	11.91	9.04	12.04	13.90	17.10	13.90	17.10	13.90	17.10
$\sum y^2$	128.81	72.00	130.75	136.97	124.65	114.58	111.26	111.09	93.94	111.32	114.41	119.25	114.41	119.25	114.41	119.25
H_{Rd} (kN)	7.42	1.95	7.65	8.38	6.93	5.73	5.33	5.31	4.59	5.34	5.71	6.28	5.71	6.28	5.71	6.28
TOT CAP (kN)	**371.64**															

M/O		**O/P**		**P/R**		**R/S**		**S/U**		**U/V**		**V/X**		**X/Y**	
0.77	0.77	0.67	0.77	0.87	0.87	0.77	0.67	0.77	0.77	0.83	1.01	1.23	1.12	0.28	1.08
9.60	9.56	8.38	9.61	10.88	10.88	9.61	8.39	9.56	9.60	10.34	12.59	15.31	13.94	3.50	13.50
12.00	11.91	9.04	12.04	15.56	15.56	12.04	9.07	11.91	12.00	14.00	21.14	31.97	26.20	1.52	24.49
111.26	111.09	93.94	111.32	117.00	117.00	111.32	93.99	111.09	111.26	114.58	124.71	136.97	130.78	72.00	128.81
5.33	5.31	4.59	5.34	6.02	6.02	5.34	4.59	5.31	5.33	5.73	6.93	8.38	7.65	1.95	7.42

The total capacity of wall 1 according to the elastic method is then the sum of capacities of the segments multiplied by two, since there are two OSB sheets in wall 1.

$$H_{Rd,tot} = \sum H_{Rd,i} \cdot 2 = 371.64\ kN$$

Wall B is made of both OSB and fibre gypsum. The parts of wall B that reach between wall 1-2 and 5-6 are made of fibre gypsum and the other segments are made of OSB. As shown in table E.2, there are two segments with the approximate length of 3 m which are made of fibre gypsum. These two segments are assumed to consist of one panel reaching over three studs and one panel reaching over two studs.

Table E.2: Capacity of wall B according to the elastic method.

Wall B	**1/2**		**2/3**		**3/4**		**4/5**		**5/6**	
L (m)	3.082	0.632	1.45	0.771	0.719	0.717	0.771	1.455	0.637	3.078
H_{Rd} (kN)	19.66	4.04	19.06	10.33	4.91	4.90	10.33	19.13	4.07	19.63

To be able to compare to Method A, only the segments of wall B between walls 1-3 will be taken into account when designing the total capacity of wall B.

$$H_{Rd,tot} = 19.66 + 4.04 + 19.06 + 10.33 = 53.09 \; kN$$

Appendix F

Tilting and anchoring

The three problems: tilting of building, anchoring of outermost modules and anchoring of top modules will be presented in detail in this appendix.

F.1 Tilting of building

As described in chapter 3.1.1, the risk of tilting of the entire building is done by comparing the location of the vertical ground reaction and the core boundary. The core boundary is approximately located one-sixth of the width of the building from the centre line. The two structures are at first analysed separately and then together as one building to see if there is any differences in the risk of tilting. In figure F.1, one structure (from its short side) is illustrated where Q1-Q8 are the wind loads and G is the self-weight of the structure. The wind load on the roof will be added to load Q8. An investigation of an eight-storey building is done to ensure the distance e is less than the core boundary $w/6 = 8.67/6 = 1.44$ m.

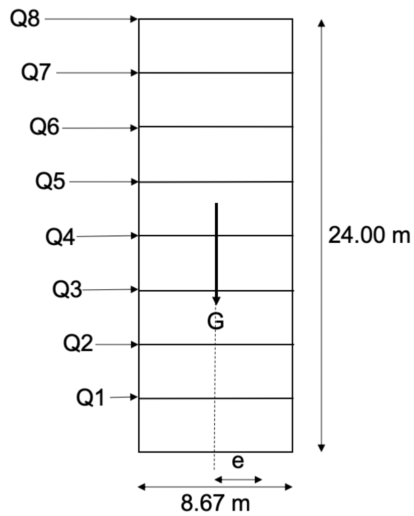


Figure F.1: Set-up of forces acting on structure when analysing the risk of tilting. Seen from short side of building.

The weights of the different modules in the building are found in table A.1. The self-weight is multiplied with a factor 0.9 according to chapter 3.1.1. The self-weight is calculated as:

$$G = (7 \cdot V1 + 11 \cdot V2 + 4 \cdot V4 + 4 \cdot V5a + 6 \cdot V7)/2 \cdot g \cdot n_{storeys} \cdot 0.9$$
$$= (7 \cdot 9790 + 11 \cdot 7858 + 4 \cdot 11030 + 4 \cdot 6702/2 + 6 \cdot 7173)/2 \cdot 10 \cdot 8 \cdot 0.9 \cdot 10^{-3} = 9200\ kN$$

The pressure wind loads acting on the structure are taken from table B.19 and the additional load on the roof are taken from table B.20 in appendix B and the results are presented in table F.1, where H is the lever arm for every load down to the foundation.

Table F.1: The wind load acting on every storey.

	Wind load (kN)	**H (m)**
Q8	375	24
Q7	213	21
Q6	205	18
Q5	195	15
Q4	184	12
Q3	170	9
Q2	151	6
Q1	121	3

For example, the wind load Q_7 is calculated as follow, where L is the length of the building:

$$Q_7 = Q_{d,pressure} \cdot L = 3.69 \cdot 57.58 = 213\ kN$$

The moment equilibrium of the loads is:

$$\sum Q_i H_i = G \cdot e$$

and the value for e can be determined by:

$$e = \frac{\sum Q_i H_i}{G}$$
$$= \frac{375 \cdot 24 + 213 \cdot 21 + 205 \cdot 18 + 195 \cdot 15 + 184 \cdot 12 + 170 \cdot 9 + 151 \cdot 6 + 121 \cdot 3}{9200} = 2.73\ m$$

The core boundary is located $w/6 = 1.44$ m from the centre line of the building and since $e = 2.73 > w/6 = 1.44$, there is a risk for tilting of an eight-storey building.

Since a risk of tilting was found for an eight storey building the same calculations are done for a six-storey building. The factor e was then determined to $e = 1.68$ m which is still larger then the core boundary of 1.44 m. This result in a risk of tilting for a six-storey building as well. Further on, an investigation of a four-storey building was done as well and the factor e was determined to $e = 1.09$ m which is less than the core boundary and no risk for tilting of a four-storey building was found.

If the connections between the structures allow the forces to be taken by both structures, the structures could be analysed together as one building. The risk for tilting in this case is presented below.

In figure F.2, the building (from its short side) is illustrated where Q1-Q8 are the wind loads, both pressure and tension loads, and G is the self-weight of the building. An investigation of an eight-storey building is done to ensure the distance e is less than the core boundary of the whole building, $w/6 = 19.83/6 = 3.31$ m.

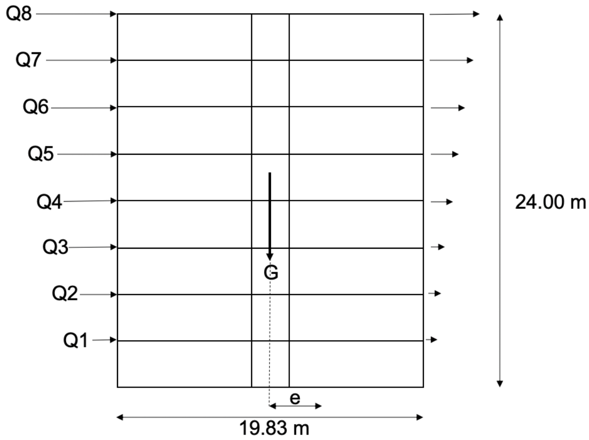


Figure F.2: Set-up of forces acting on the building when analysing the risk of tilting. Seen from short side of building.

The weights of the different modules in the building are found in table A.1. The self-weight of the building is calculated as:

$$G = (7 \cdot V1 + 11 \cdot V2 + 4 \cdot V4 + 4 \cdot V5a + 6 \cdot V7) \cdot g \cdot n_{storeys} \cdot 0.9$$
$$= (7 \cdot 9790 + 11 \cdot 7858 + 4 \cdot 11030 + 4 \cdot 6702/2 + 6 \cdot 7173) \cdot 10 \cdot 8 \cdot 0.9 \cdot 10^{-3} = 18400\ kN$$

The pressure and tension wind loads are taken from table B.19 and the additional load on the roof is taken from table B.20 in appendix B and the results are presented in table F.2, where every Q_i represents both the pressure and tension loads for every storey and H is the lever arm for every load down to the foundation. The load Q8 also includes the wind load on the roof.

Table F.2: The wind load acting on every storey, pressure and tension loads.

	Wind load (kN)	**H (m)**
Q8	605	24
Q7	352	21
Q6	339	18
Q5	323	15
Q4	305	12
Q3	281	9
Q2	250	6
Q1	200	3

For example, the wind load Q_7 is calculated as:

$$Q_7 = (Q_{d,pressure} + Q_{d,tension}) \cdot L = (3.69 + 2.42) \cdot 57.58 = 352\ kN$$

The moment equilibrium of the loads is:

$$\sum Q_i H_i = G \cdot e$$

and the value for e can be determined by:

$$e = \frac{\sum Q_i H_i}{G}$$
$$= \frac{605 \cdot 24 + 352 \cdot 21 + 339 \cdot 18 + 323 \cdot 15 + 305 \cdot 12 + 281 \cdot 9 + 250 \cdot 6 + 200 \cdot 3}{18400} = 2.24\ m$$

The core boundary is located $w/6 = 3.31$ m from the centre line of the building and since $e = 2.24 < w/6 = 3.31$ there is no risk for tilting of an eight-storey building when analysing the whole building. The conclusions made in this section does only hold for these specific examples and with all the assumptions made in this report.

F.2 Anchoring of outermost modules

Tilting of a building is a problem when the wind load creates tension forces acting on the short sides of the building. If these forces create a greater rotating moment than the self-weight of the modules, there is a risk for tilting. In this chapter an analysis of how much the modules on the short sides of the building needs to be fastened to the inner modules are presented.

Figure F.3 shows the forces acting on the outermost modules. Three different forces are of interest when analysing the risk of tilting. The self-weight of the modules (G in figure F.3), the tension forces on the gables as a result of when the wind load acts on the long side of the building (Q1-Q4 in figure F.3) and, if necessary, anchoring between the outermost modules and the modules within (F in figure F.3). It is assumed that F is equal over the height of the building. A moment equilibrium for the wind load and the self-weight together with the fasteners to the inner modules needs to be fulfilled, see equation F.1.

$$M_F + M_G > M_Q \tag{F.1}$$

Where:

$$M_F = F(H_4 + 2 \cdot H_3 + 2 \cdot H_2 + 2 \cdot H_1) \tag{F.2}$$

$$M_G = G \cdot \frac{w}{2} \tag{F.3}$$

$$M_Q = Q_4 \cdot H_4 + Q_3 \cdot H_3 + Q_2 \cdot H_2 + Q_1 \cdot H_1 \tag{F.4}$$

And $H_1 = 3$ m, $H_2 = 6$ m, $H_3 = 9$ m and $H_4 = 12$ m.

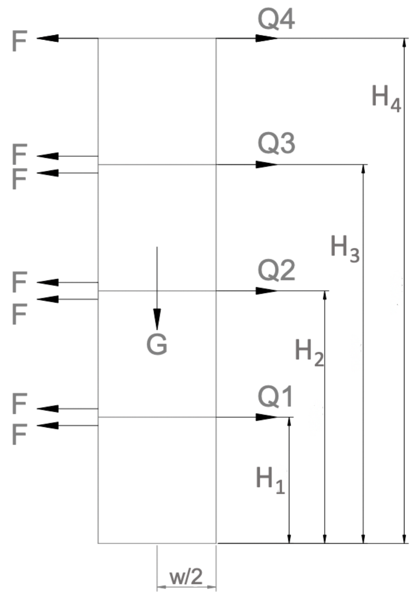


Figure F.3: Forces acting on the outermost modules.

The analysis of the risk for tilting is divided into two cases, where the first case is when only the outer modules, the modules between walls A-B and X-Y, are of interest. The necessary capacity of the connection between these modules and the inner modules, i.e. the connection in walls B and X will be designed.

The second case is when it is assumed that the outermost modules are appropriately connected to the next modules. The necessary capacity of the connection between these modules and the inner modules, in turn, i.e. the connection in walls C and V will be designed.

The weights of the modules in the analysis for tilting is presented in table F.3.

Table F.3: The weights of modules.

Module	**Weight (kg)**
Modules between Wall A-B and X-Y	3350
Modules between Wall B-C and V-X	7860

The wind load acting as a tension load on the short side of the building is calculated according to appendix B and all the values for wind pressure and pressure coefficients are taken from the same appendix. The maximum pressure coefficient is -1.2 according to table B.5, it is zone A and the length of this zone is 7.08 m. The wind load at the top of the building (Q4 for a four-storey building) includes the tension loads created on the gable of the roof structure as well as the tension loads on the upper half of the top module.

F.2.1 Case 1

A detailed design for a four-storey building will be presented below. The results for all the different building heights are presented in table F.5.

Since the length of the small modules on the short sides of the building is less than 7.08 m, (5.167 m), the whole module will be in zone A with a pressure coefficient of -1.2. In table F.4, M_Q is designed by taking the point loads from the wind load multiplied with the lever arms respectively.

Table F.4: Total rotating moment from wind load.

Storey	Wind load (kN)	Lever arm (m)	M_Q (kNm)
4	31.48	12.00	
3	22.87	9.00	
2	20.32	6.00	
1	16.26	3.00	**754**

An example of the wind load acting at the top of storey four will be presented below:

$$Q_{d,H_4} = q_p \cdot c_{pe,10} \cdot 1.5 \cdot h \cdot w = 0.98 \cdot 1.2 \cdot 1.5 \cdot (1.5 + 2.32) \cdot 5.167 = 31.48\ kN$$

The height of influence for this load is $1.5 + 2.32 = 3.82$ m, where 1.5 m representing half the height of the module and the additional 2.32 m is the approximate equivalent height of the triangular gable on the roof structure, see figure F.4.

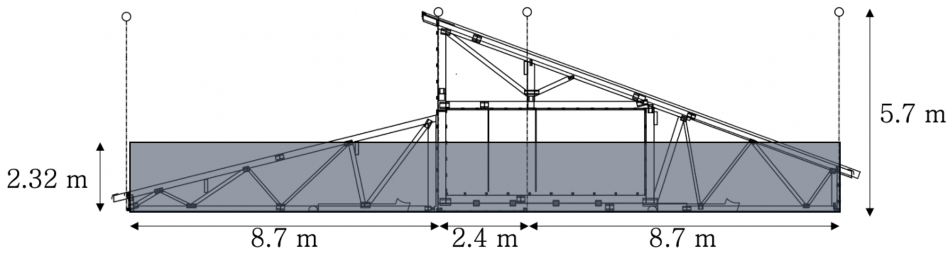


Figure F.4: Equivalent height of triangular area of influence.

The same calculations are done for storey 3, 2 and 1 as shown in table F.4. The total rotating moment from the wind load is presented as M_Q and designed according to:

$$M_Q = Q_4 \cdot H_4 + Q_3 \cdot H_3 + Q_2 \cdot H_2 + Q_1 \cdot H_1 = 31.48 \cdot 12 + 22.87 \cdot 9 + 20.32 \cdot 6 + 16.26 \cdot 3 = 754\ kNm$$

The width of the outermost module is $w = 2.786$ m, and half this length is the lever arm for the self-weight, as shown in figure F.3. The weight of the module is 3350 kg and there are four modules stacked upon each other. The rotating moment from the self-weight is then:

$$M_G = G \cdot \frac{w}{2} = 4 \cdot 3350 \cdot 10 \cdot 10^{-3} \cdot \frac{2.786}{2} = 187\ kN$$

The fasteners along the height of the building, named F in figure F.3, together with the self-weight of the modules, are designed to resist the overturning moment from the wind load. It is assumed that all the fasteners carry the same amount of load. The force in the fasteners can then be designed according to equation F.1 since the load F is the only unknown parameter in the moment equilibrium.

$$F(H_4 + 2 \cdot H_3 + 2 \cdot H_2 + 2 \cdot H_1) + 187 > 754$$

$$F(12 + 2 \cdot 9 + 2 \cdot 6 + 2 \cdot 3) + 187 > 754$$

$$F = 12\ kN$$

Every fastener (F in figure F.3) connecting the modules in wall B and X must resist a tension load of 12 kN for a four-storey building. If two fasteners (one on each side) are used instead, the resistance of the fasteners only have to be 12/2 = 6 kN, see figure F.5. The results for the higher buildings are presented in table F.5.

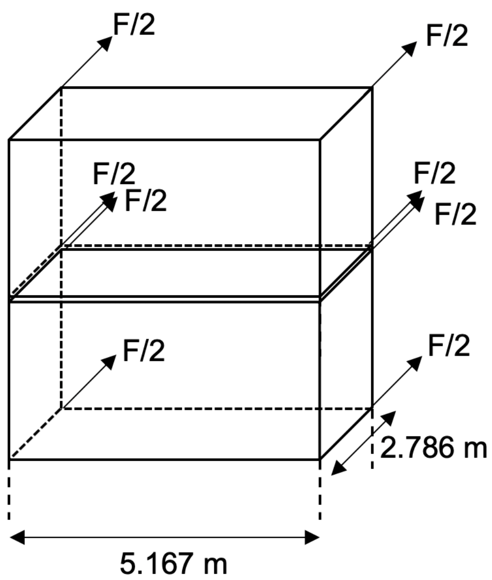


Figure F.5: The division of fasteners for the two top modules in wall B or wall X.

Table F.5: Tension forces in fasteners for the different buildings.

	Tension force in fasteners (kN)
4-storey	12
6-storey	14
8-storey	15

F.2.2 Case 2

The differences with case 2 compared to case 1 are that the self-weight and the wind load are higher since there are two combined modules. The length of this combined module is

$5.167 + 3.5 = 8.667$ m. The length of zone A, where the pressure coefficient is -1.2, is 7.08 m. Even though the modules for case 2 do not completely fit into this span, it is assumed that the pressure coefficient for zone A acts all over the length of 8.667 m.

The wind load and the lever arms are presented in table F.6 together with the total rotating moment of the wind load, M_Q.

Table F.6: Total rotating moment from wind load.

Storey	**Wind load (kN)**	**Lever arm (m)**	**M$_Q$ (kNm)**
4	52.81	12.00	
3	38.36	9.00	
2	34.08	6.00	
1	27.28	3.00	**1265**

The counteracting moment from the self-weight is designed as:

$$M_G = (4 \cdot 3350 + 4 \cdot 7860) \cdot 10 \cdot 10^{-3} \cdot 4.74 = 2125 \; kN$$

Where 4.74 m is the combined lever arm for the self-weight of the two modules.

The tension forces in the fasteners can be designed as:

$$F(12 + 2 \cdot 9 + 2 \cdot 6 + 2 \cdot 3) + 2125 > 1265$$

$$F = 0 \; kN$$

For a four-storey building, it is not necessary to connect the modules in walls C and V regarding tilting. In table F.7 the results for the higher buildings are presented. Only for an eight-storey building, it is necessary to connect the modules in walls C and V regarding tilting.

Table F.7: Tension forces in fasteners for the different buildings.

	Tension force in fasteners (kN)
4-storey	0
6-storey	0
8-storey	5

F.3 Anchoring of top modules against up-lifting

The necessary anchoring (F in figure F.6) between the uppermost modules and the modules beneath due to the wind load (Q in figure F.6) when it is acting on the long sides of the building are analysed. There will be a rotating moment from the wind load and the self-weight of the modules (G in figure F.6), together with the anchoring force, act as counteracting moment.

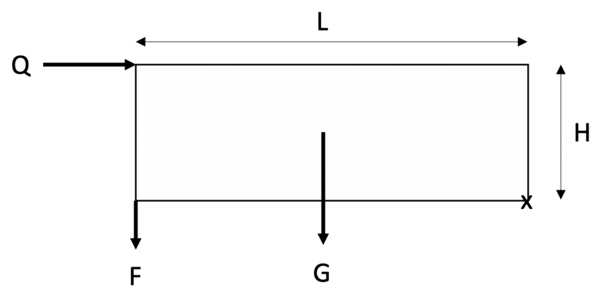


Figure F.6: Set-up of forces acting on the uppermost module.

The moment equilibrium for the top module is:

$$Q \cdot H = F \cdot L + G \cdot L/2$$

and F can be determined by:

$$F = \frac{Q \cdot H}{L} - \frac{G}{2}$$

Three modules in structure 1 on the top floor of an eight-storey building are analysed; the modules between walls A-B, D-E and F-G. The loads and the necessary anchoring are presented in table F.8. The force F is in all three cases negative, i.e. no anchoring of the top modules for an eight-storey building is required.

Table F.8: The anchoring force of the top modules of an eight-storey building.

	A-B	**D-E**	**F-G**
G (kN)	33.51	78.58	97.90
L (m)	5.17	8.67	8.67
Q (kN)	18.15	25.67	19.16
F (kN)	-6.22	-30.40	-42.32

How the values for the module between wall D and E in table F.8 are compiled is presented below.

The weight of the module between walls D and E is 7858 kg according to table A.1 and the self-weight as a load is then:

$$G = 7858 \cdot 10 \cdot 10^{-3} = 78.58 \ kN$$

Q is the wind load acting on the module together with the wind load acting on the roof, values are taken from table B.19 and B.20, the design wind load Q is:

$$Q = (w_{e,pressure} \cdot \frac{h}{2} + q_{roof}) \cdot 1.5 \cdot w_{module} = (0.85 \cdot \frac{3}{2} + 3.07) \cdot 1.5 \cdot 3.94 = 25.67 \ kN$$

Where the height of influence of the load acting on the module is $h/2 = 1.5$ m and the width of the module is 3.94 m.

The length of the module is 8.67 m, the height is 3 m and the anchoring force F can then be calculated:

$$F = \frac{Q \cdot H}{L} - \frac{G}{2} = \frac{25.67 \cdot 3}{8.67} - \frac{78.58}{2} = -30.40\ kN$$