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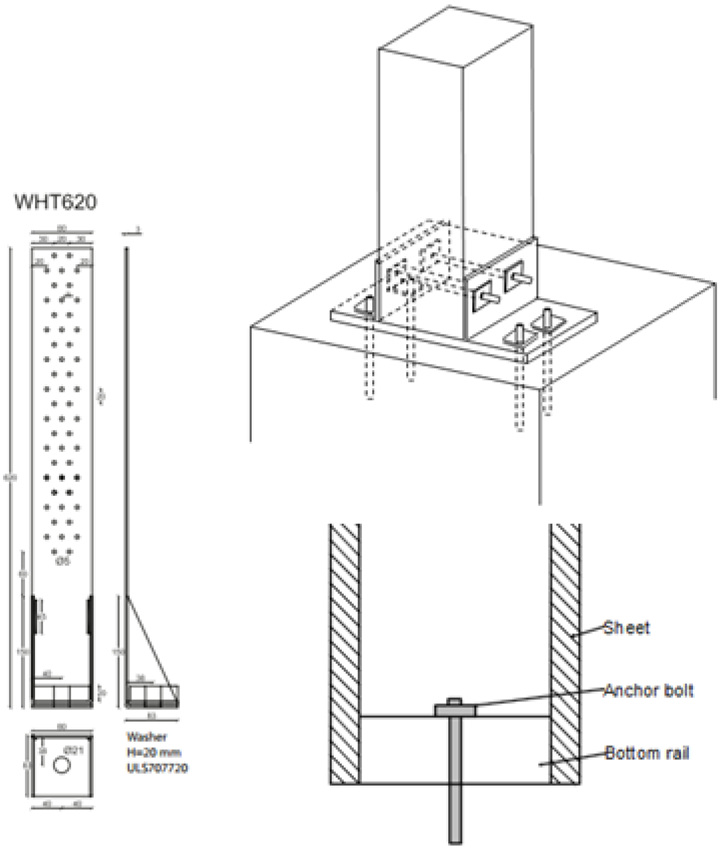

Multi-storey houses in timber

Stability and anchoring systems

Master’s Thesis in the Master’s Program Structural Engineering and Building Technology

DENNIS FERRI

SOFIA LAM

Department of Civil and Environmental Engineering

Division of Structural Engineering

Steel and Timber Structures

CHALMERS UNIVERSITY OF TECHNOLOGY

Gothenburg, Sweden 2015

Master’s Thesis 2015:66

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Cover:
Examples of anchorage systems looked into in this master thesis. To the left: hold downs, up to the left: Post Base with Steel Bracket and down to the left: Anchorage with anchor bolts.

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ABSTRACT

In the year of 1994 Swedish Boverket introduced new regulations concerning multi-storey buildings in timber which allowed buildings with more than two stories to be erected. In conjunction with the rising awareness of environmental issues and the strive towards a more sustainable society, construction of multi-storey houses in timber have increased. This since the material is considered to be a more environmental friendly alternative than traditional building materials such as steel and concrete. Characterizing for multi-storey timber buildings is the low self-weight which can cause stability problems and entails large uplifting forces in the stabilizing elements of the structure. This is a well-known problem for taller timber constructions which places high demands on anchorages to be able to resist the uplifting forces caused by lateral loads. The higher a timber structure is the higher forces the building has to be able to withstand.

The aim of this thesis was to design two buildings with the same floor plan but with different number of stories, one with four stories and one with seven stories. Stability calculations were made and critical uplifting forces were compared for both of the buildings. The calculations showed that the self-weight for both buildings were sufficient to resist tilting. The uplifting forces acting on the shear walls for the seven storey building that needs to be anchored resulted in more than three times larger tensile forces compared with the building of four floors. In addition to this, suggestions on solutions for anchoring systems such as glued-in rods, slotted-in steel plates and hold-downs have been presented for the seven storey building.

Key words: post and beam, balloon framing, stud walls, platform framing, shear walls, tilting, anchorage, glued-in rods, slotted-in steel plates, hold-down

Flervåningshus i Trä
Stabilitet och förankringssystem

Examensarbete inom masterprogrammet rogram Structural Engineering and Building Technology

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SAMMANFATTNING

År 1994 införde Boverket nya regler gällande flervåningshus i trä där byggnader med fler än två våningar fick uppföras. I samband med den ökande miljömedvetenheten och strävan mot ett mer hållbart samhälle har byggandet av flervåningshus i trä ökat. Detta då materialet anses vara ett miljövänligare alternativ än de traditionella byggnadsmaterialen stål och betong. Något som är kännetecknande för flervåningshus i trä är den låga egentyngden som kan skapa stabilitetsproblem och medföra stora dragkrafter i byggnadens stabiliserande element. Detta är ett välkänt problem för träkonstruktioner vilket ställer höga krav på förankringar som måste kunna ta upp stora krafter i drag på grund av horisontala laster. Desto högre en träbyggnad är desto större krafter måste byggnaden kunna stå emot.

Målet med detta examensarbete var att designa två byggnader med samma planlösning men olika våningsantal, en byggnad med fyra våningar och en med sju våningar. Stabilitetsberäkningar utfördes och kritiska dragkrafter jämfördes för de båda byggnaderna. Beräkningarna visade att egentyngden för de båda konstruktionerna var tillräcklig för att motverka stjälpning. Dragkrafterna för de stabiliserande väggarna som måste förankras på sjuvåningshuset visade sig bli nästan tre gånger högre än dragkrafterna på huset med fyra våningar. Utöver detta har förslag på förankringslösningar som inlimmade stavar, dymlingsförband med inslitsade stålplåtar, och vinkelbeslag presenterats för sjuvåningsbyggnaden.

Nyckelord: pelare- och balksystem, ballongmetod, regelväggar, ’plattform-metod’, skjuvväggar, stjälpning, förankring, inlimmade stavar, dymlingsförband med inslitsade stålplåtar, vinkelbeslag

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Preface

The master thesis was carried out between January and June 2015 at the Department of Civil and Environmental Engineering, Structural Engineering, at Chalmers University of Technology, Sweden.

From the beginning, the purpose of this master thesis was to do a cost and structural analysis for two equally high buildings but with different structural systems. The purpose was later changed to comparing two buildings with different heights but with the same structural system and floor plans regarding stability and uplifting forces caused by lateral loads.

We would like to thank our supervisor Professor Robert Kliger and our examiner, Assistant professor Reza Haghani for their continuous support during the thesis. Their help throughout the project is tremendously appreciated.

Göteborg June 2015

Dennis Ferri and Sofia Lam

Notations

Roman upper case letters

A	Area
B	Width
L	Length
E	Modulus of elasticity
F_{tk}	Characteristic pull-out strength
F_{td}	Design pull-out strength
G	Shear modulus
G_k	Characteristic load
M	Moment
M_{rRk}	Characteristic fastener yield moment
R	Resistance force
R_{ax}	Anchorage capacity
V	Self-weight

Roman lower case letters

d	Diameter
$f_{v.k}$	Shear strength parameter
f_y	Yield limit
h	Height
l_a	Anchoring length
l_{eff}	Effective length
n	Number of dowels
n_s	Number of plates
q	Wind load
q_p	Peak velocity pressure
t	Thickness
w	Wind pressure

Greek lower case letters

α	Inclination angle
γ_M	Partial factor

ν	Poisson ratio
ρ	Density
τ	Shear

Symbols

//	Parallel
$\perp$	Perpendicular

1 Introduction

1.1 Background

Since year 1994 when new regulations from Boverket allowed more than two storey timber building to be constructed, the number of multi-storey timber structures has increased. Today, construction companies that are building multi-storey timber structures usually construct four or five floor buildings, but the possibility of building taller is demanded. One of the main issues with constructing tall buildings is due to problem with stability and anchoring of large uplifting forces.

Timber is a light material compared to more traditional building materials such as steel and concrete. The low self-weight of timber structures can cause stability problems and entail large uplifting forces in the stabilizing elements of the structure. The taller a building is more difficulties concerning stability and anchoring arises.

To resist larger uplifting forces that is mainly caused by wind loads anchorages with high capacity are required. The anchorages have to be able to transfer vertical and horizontal forces to the foundation slab in order to prevent overturning of the building. Different types of anchorage systems are available for lower multi-storey buildings but for taller multi-storey building these anchorage system might not be sufficient to transfer the larger loads to the foundation slab.

1.2 Aim

The aim of this report was to study two multi-storey buildings with the same floor plan but different heights and to check the stability and compare the uplifting forces of the shear walls of the buildings and to present possible anchorage systems for the taller building.

1.3 Objectives

The objectives were to study different constructions methods, post and beam with balloon framing and stud walls with platform framing, for buildings in timber and to design two reference buildings, one with four stories and the other with seven stories. In addition, the objectives were to check tilting of the building, to compare the uplifting forces of the shear walls and to suggest anchoring systems for the seven storey building.

1.4 Method

First, a literature study was made considering different construction methods for buildings with 4-5 floors, which is a common floor height today for multi-storey timber buildings. The focus was on balloon framing constructed with post and beam system and platform framing built with stud walls. The focus was also on how anchorage can be solved for these systems.

Two buildings based on one of the studied structural systems was designed in the software Revit, one building with four floors and the other with seven floors. These (reference) buildings were checked with regards to stability. In addition, hand calculations of the uplifting forces the shear walls were subjected to were made and suggestions of solutions for different anchoring systems were provided.

1.5 Limitations

This thesis project was focused on one timber structural system, multi-storey houses constructed with stud walls. The designs of the building only included the framing and floor plans with shear walls. The anchoring systems that were studied were glued-in rods, hold-downs and slotted-in plates. The calculations regarding the stability and slotted-in steel plates for anchorage were made according to Eurocode 5. Since the design code does not provide any solutions for glued-in rods, calculations were performed in regards to recommendations from previous carried out tests with this anchorage systems. The stability checks of the design buildings were only checked against tilting and uplifting forces in the shear walls caused by wind load. Earthquake loads were not included.

2 Timber and Wood-based Products as a Construction Material

2.1 What is timber?

Wood is the organic matter that is attained from trees and timber is wood used as a building material. Timber (or lumber) is a natural building material and has been used for thousands of years. Sawn timber is larger pieces of wood that are cut from logs in various shapes and dimensions. These members can contain natural abnormalities, such as knots or spiral grains that are seen as defects in engineering. Wood is known to be a hygroscopic and anisotropic material (Crocetti et al. 2011).

Since wood is an organic material some types of animals and plants digest parts of the wood, which can lead to a lower strength and stiffness of the material. Two categories of agents (biotic and abiotic) cause deterioration and destruction of wood. Biotic deterioration is a biological attack and can be caused by for example fungi or insects. Abiotic agents are non-biological, which include the sun, wind, water, chemicals and fire.

2.2 Advantages with constructing with timber

The natural material is rather light which means that less material for foundation is needed. This leads to lower costs and savings of foundation material, compared when constructing with higher density buildings, e.g. buildings made of concrete (Swedish Wood 2012b). In addition, the strength to weight ratio is higher and favorable compared with traditional materials (Friquin 2010).

Furthermore, timber is a material suitable for prefabrication. By using prefabricated elements less material is wasted, the construction cost is lower and erection process is faster since the work on-site required is minimized. Furthermore, the disassembly of a construction made out of timber is easy and can be reused or recycled as biofuel or used as another building material. By producing timber material in factories the results are of high quality since the elements are produced in a controlled environment, i.e. the production is weather independent. Another advantage from an architectural point of view is that flexible solutions and innovative designs are possible. It is also easy to make modifications on site compared with traditional materials (Fire Protection Engineeering 2014; Friquin 2010). Moreover, timber constructions have proven to be less affected by seismic loads than concrete or masonry constructions (Swedish Wood 2012a).

2.3 Sustainability benefits

The past decades the rising awareness of sustainability has arisen among people, companies, governments etc. Also in the building sector this has been noticeable where construction companies strive towards constructing green and sustainable buildings. Not only optimization of environmental impacts is sufficient to achieve sustainable buildings but also economic and social aspects are of importance. When taking all these aspects into consideration timber has been proven to be a suitable alternative to achieve sustainable constructions (Fire Protection Engineeering 2014).

The sustainable benefits from using timber material are many. The effect on the environment is low from production to demolition compared with traditional building materials such as steel and concrete. This is due to timber being a renewable construction material, has the ability to absorb carbon dioxide and store it in their woody tissue resulting in a reducing of greenhouse emissions. The carbon dioxide also remains fixed in timber products over a time and by continuously replacing harvested trees with new plants, the same amount of carbon dioxide stays in the atmosphere. Even when it comes to energy, timber has an advantage compared to more common building materials in three different energy categories – manufacture, transport and maintenance. Manufacturing timber demands little external energy due to most of the energy used for timber production in Sweden is coming from biofuels. Transportation is made easier and is more fuel efficient due to timber being a light material. When it comes to maintenance timber elements can be repaired or switched with uncomplicated methods and to reasonable prices.

2.4 Physical characteristics of timber

As mentioned before, wood is a hydroscopic material which means that it is easily affected by the moisture content in the surrounding air. When discussing moisture content in wood, an important term is fibre saturation point (FSP). It indicates the moisture content where the cell walls are saturated with moisture but the cavities in the wood are empty. This point varies depending on the wood species but usually it takes place at 25 – 32 % moisture content where a lower point causes shrinkage and a higher point swelling (Burström 2010).

The amount of shrinkage or swelling depends on which direction that is studied, i.e. wood is an anisotropic material. Due to swelling and shrinkage wood can get geometrical imperfection which in some cases makes the wood pieces inappropriate to use. These distortions are twist, spring (also known as crook), cup and bow (Crocetti et al. 2011), Figure 2.1.

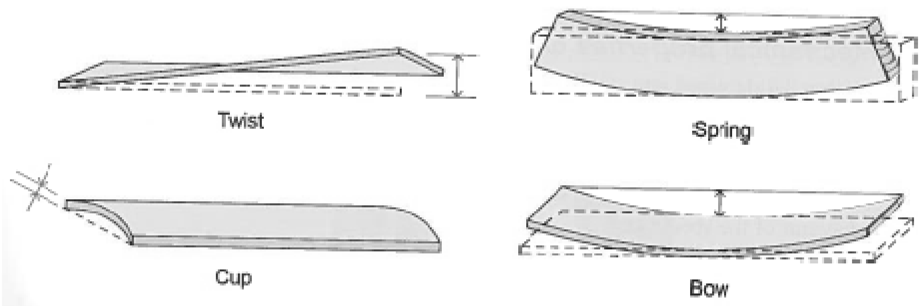

Figure 2.1 Distortion of wood (Design of timber structures 2011).

2.5 Mechanical properties of timber

Wood is an anisotropic material with different properties in different directions. The three principle axes are longitudinal, radial and tangential. The longitudinal direction is parallel to the fibres, the radial and tangential is perpendicular to the grain with the difference that the radial direction is normal to the grain and the tangential is tangent to the grain (Green et al. 1999). Figure 2.2 shows the main axes directions.

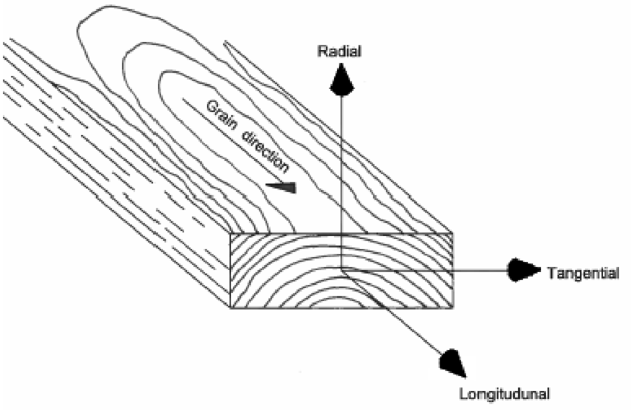

Figure 2.2   Principle directions of wood.

The main loading stresses of wood is in tension, compression and in shear (Record 2004). When testing specimen there is a difference between smaller clear wood samples and larger timber pieces, where the former one is disregarded from natural characteristics such as knots where in timber samples they are not. Clear wood specimen is when loaded parallel to the fibres strongest in tension with a stress-strain relationship to appear to be linear until a brittle failure occurs. However, when loaded in tension perpendicular to the grain the strength properties are very weak (Airey et al. 2010; Kreith & Goswami 2005; Crocetti et al. 2011).

When loaded in compression the strength is high parallel to the fibres but not as strong as in tension due to different behavior of wood in compression. At a high compression load the cell structure deforms and fails as a result of buckling of the fibres and will obtain a plasticizing behavior (Crocetti et al. 2011).

There are three modes for shear to occur, if the difference between the radial and tangential direction is disregarded; shear parallel to the grains, shear perpendicular to the grains and rolling shear (Kreith & Goswami 2005). The highest strength obtained is planes parallel to the grain direction whereas for shear perpendicular to the grain, i.e. rolling shear, is approximately half the strength parallel to the grain. Shear in tangential-longitudinal (τ_{TL}) and radial-longitudinal (τ_{RL}) are most likely where shear forces occur in timber structures. The lower value of these is used in Eurocode since it is hard to distinguish between the two directions (Crocetti et al. 2011).

Table 2-1 shows example of strength properties (in MPa) for different types of timber products. Values are taken from Eurocode 5 (BSI 2014).

Table 2-1 Strength properties of some timber products (MPa).

	Solid timber		Glulam	LVL	
	C40	C22	GL20c	Kerto S (t=21-90mm)	Kerto Q (t=21-24mm)
Tension // to grain	24	13	15	35	17.6
Tension ⊥ to grain	0.4	0.4	0.5	0.8	6
Compression // to grain	26	20	18.5	35	19
Compression ⊥ to grain	2.9	2.4	2.5	1.8-6	1.8-9
Shear	4	3.8	3.5	2.3-4.1	1.3-4.5

As stated before a clear specimen of wood in tension parallel to the fibres is the strongest. However, as can be seen in the table above, the compression parallel to the grain is slightly higher than tension parallel to the grain for solid timber. The reason for this is partly because of natural characteristics such as knots that induces higher concentrated stresses around these and reduces the strength where in tension it is affected the most (Crocetti et al. 2011). Glulam, which consist of lams or pieces of lumber that are glued together with adhesives, have also higher compression strength parallel to the fibres but lower in tension. The same for the Laminated Veener Lumber which consists of a number of veener sheets glued together (Chapter 2.8) in the same direction (Kerto S) or some layers in different directions (Kerto Q) (Crocetti et al. 2011).

The mechanical properties of wood and timber are influenced by a number of factors; moisture, duration of load, long term deformations, temperature and size (volume). If the moisture is below the fiber saturation point the strength and stiffness will increase whereas if it is above FSP there is not a noticeable difference. The duration of load (DOL) or loading time, mostly affect the bending strength in timber. Tests have shown that the strength decreases more if long term load is applied compared with short time. An elastic time-dependent deformation of a member caused by loading is known as creep. The environment such as temperature and moisture also affects the creep level. The deformation can be split into three stages, elastic deformation, delayed elastic deformation and viscous deformation. The first mentioned will occur instantly after applied load. After a constant load is applied the deformation will increase which is both a delayed elastic and viscous deformation. Delayed elastic deformation will after unloading go back to its initial stage while viscous deformation is non-reversible, i.e. permanent.

2.6 Stiffness of timber

All materials are elastic in one way or another. The less elastic a material is the stiffer it is. Stiffness is measured in elasticity module, also known as E-module, which per definition is the relation between stress and strain. The higher the E-modulus is, the higher the stiffness is (Carbontrikes 2008). Below, in Table 2-2, E-modulus for solid timber, glued laminated timber and LVL in different strength classes is shown.

Table 2-2 E-modulus for different timber materials.

Stiffness of Solid timber [MPa]									
	C40	C35	C30	C27	C24	C22	C20	C18	C16
Elastic modulus // to grain, $E_{0.mean}$	14000	13000	12000	11500	11000	10000	9500	9000	8000

Stiffness of Glued Laminated timber [MPa]							
	GL20c	GL22c	GL24c	GL26c	GL28c	GL30c	GL32c
Elastic modulus, $E_{0,g,mean}$	10400	10400	11000	12000	12500	13000	13500

Stiffness of Laminated Veneer Lumber (LVL) [MPa]			
	Kerto S	Kerto Q (21-24 mm)	Kerto Q (27-69 mm)
Elastic modulus // to grain, $E_{0.mean}$	13800	10000	10500

As the table shows, timber is very stiff parallel to the grain. In comparison with more traditional materials, timbers stiffness is only one thirtieth of steels. Despite that, timber is relatively good regarding fluctuations due to its high stiffness. More research although needed in order to improve the vibration resistance in timber constructions.

2.7 Engineered wood products

Engineered wood products are mainly composites of wood (sawn timber, veneers, strands, chips or fibres) that are remanufactured to structural members. The components are bonded together with an adhesive. The production of engineered wood products allows for smaller pieces of timber to be used which is an advantage compared to normal sawn timber which has a limitation in dimensions due to the size of the trees and the process in factories. (Crocetti et al. 2011).

Some of the advantages with engineered wood products are that they are man-made and can therefore be designed in a variety of thickness, sizes and grades in order to meet the customers' needs (Covering Floor News 2013). They are optimized to reach their maximum strength and stiffness resulting in better wood product than typical products like solid timber (naturally:wood 2015). Engineering wood products are also efficient during the manufacturing process leading to little waste (APA 2015).

Even though engineering wood products are optimized it is important to know that the E-modulus can today, not be made higher by any manufacturing process. Therefore the same problems regarding stiffness, deflection and vibrations remains as for solid timber. To overcome these difficulties the engineered wood product has to be interleaved with other building materials such as reinforced concrete, steel or FRP (Fibre Reinforced Polymer), which have higher stiffness properties.

2.8 Laminated Veener Lumber (LVL)

Developed in the 1970s, Laminated Veneer Lumber (LVL) has become a popular timber product used in many respects within timber constructions. Manufactured by attaching multiple layers of rotary peeled or sliced thin wood veneers with adhesives under heat and pressure, a high-strength engineered wood product is made. LVL is used for permanent structural application such as beams, purlins, trusses and formwork. Timber members in LVL can also be made long to almost any length where the only limitation being the transportation to the construction site (Wood Solutions 2013).

LVL has a high stiffness because the veneers are placed in a way so that the grains are oriented in the same direction. The orientation contributes to making LVL straighter and more uniform compared to solid timber also giving it almost the same orthotropic properties, more known as mechanical properties against different axes. LVL is more resistant to shrinkage and warping and can withstand larger loads and span longer distances than solid timber (Wood Solutions 2013).

LVL is more familiar under the name Kerto which are divided into four types – Kerto-S, Kerto-Q, Kerto-T and Kerto-Ripa all with different properties and applications.

Typically for Kerto-S and Kerto-T is that the grains run longitudinally through all the layers while 1/5 of the veneers of Kerto-Q are glued cross wised. Kerto-Ripa is a combination of Kerto-Q panels and Kerto-S ribs perfectly suited as floor and roof members. Kerto-S is more suitable as a material for timber beams especially when long spans with minimal deflection are required and Kerto-Q when high compression strength is needed. Kerto-T has similar properties as Kerto-S but lighter and therefore better fit for use as wall studs, load- and non-load-bearing in external and internal walls (MetsäWood 2014).

2.8.1 Cross-Laminated Timber (CLT)

Invented in the early 90s in Europe, CLT has been gaining popularity since then, becoming a strong competitor to classical building materials like steel, masonry and concrete. This is due to CLT being a great complement to today’s existing light frame and heavy timber options which has resulted in CLT becoming more common in construction of multi-storey buildings in timber (Mohammad et al. 2011).

CLT is made out of dried lumber panels that are glued together under pressure with the grains of the panels running perpendicular to the neighboring panels’ grains (Mohammad et al. 2011). This placement of the panels makes CLT a stiffer product that allows the loads to transfer on all sides of the material, also keeping its strength and shape (American Wood Council 2013). The most common number of layers in CLT is three to seven layers but more layers can be added upon request in some cases (Mohammad et al. 2011).

The area of usage for CLT is often in prefabricated walls and floor panels offering the possibility to build floor slabs crossing long spans and walls long enough for a single story. This is due to high in-plane and out-of-plane stiffness properties in both directions that gives CLT it stability properties, much similar to a reinforced concrete slab where a two-way action is achieved (Mohammad et al. 2011).

3 Challenges with Constructing in Timber

There are many great advantages with using timber as a construction material as mentioned in previous chapters. However, there are some challenges with constructing with timber such as fire, stability and sound and vibration, which are presented below.

3.1 Fire

A challenge regarding timber constructions is fire. It is known that wood burns but what is less acknowledged is that it does so in a predicted way and relatively slow. Due to many previous fire accidents very strict regulations concerning this issue have been decided. The thicker a wooden piece is, the longer it can resist fire. Some engineering products with larger cross-sections such as glulam have a high resistance against fire and additionally the glue used that keeps the wooden pieces together can endure fire rather well (Crocetti et al. 2011).

The wood will start to burn immediately when exposed to flames but once the outer part of the wood has been burn it turns into charcoal which has a function to protect the inner part of the wood member against heat, i.e. charring effect. If timber structures are compared with steel structures, the latter mentioned does not burn. However, when a critical temperature is reached the steel structure will collapse unexpectedly due to softening and melting of the steel. One reason to why timber is not chosen in many cases as load bearing members is due to lack of documentations and codes, while for example concrete and steel are well documented (Friquin 2010).

In Eurocode 5 (design of timber structures) fire is treated and requirements that have been set up are based on standardized tests and classifications. The two main areas concern fire technology properties for construction products and fire resistance for load bearing structures. Eurocode proposes some examples of calculations to fulfill the requirements but these are however not complete. SP Handbook includes calculations from Eurocode and alternative ways to fulfill the regulations in the design codes and important considerations to take when designing timber buildings (SP Trä 2012).

With the risk of fire in multi-storey timber structures automatic sprinkler systems are suggested to be installed in order to prevent fire from occurring and spreading and simplify the extinguish of fire. There are other solutions, e.g. fire doors that are recommended in Eurocode but sprinkler are one of the most effective ones (Buchanan et al. 2014).

3.2 Stability

Timber constructions that are rather light cause stability problems, particularly the horizontal forces needs to be taken care of. For multi-storey buildings, the wind pressure increases with the height and the uplifting forces on the wind side is strong. A common solution for multi-storey buildings in timber is to construct the ground floor in concrete which has a higher density and then connect the structure with the concrete (Crocetti et al. 2011; Swedish Wood 2012b). To prevent horizontal load from deforming the structure, diagonal bracings or shear walls can be added. An additional way to achieve stability is by making rigid joints between elements that prevents any angular changes. For the uplifting forces especially in the shear walls the design of anchorage to the foundation needs to be made carefully (Crocetti et al. 2011).

3.3 Acoustic

A major issue regarding timber constructions is acoustics due to the light density. The design codes recommendations and regulations are more suitable for heavier constructions (VINNOVA & Formas 2009). Being exposed to noise (unwanted sound) can lead to hearing loss or even psychological long-term effects such as stress. For light-weight buildings many problems concern direct transmission, i.e. airborne transmission and impact sound, and flanking transmission (Canadian Centre for occuational Health and Safety 2008; Scottish Building Standards 2013), see Figure 3.1. As is known, the forces in a structure take the easiest and fastest path in a structural system. The same goes for sound, it finds the weakest connection.

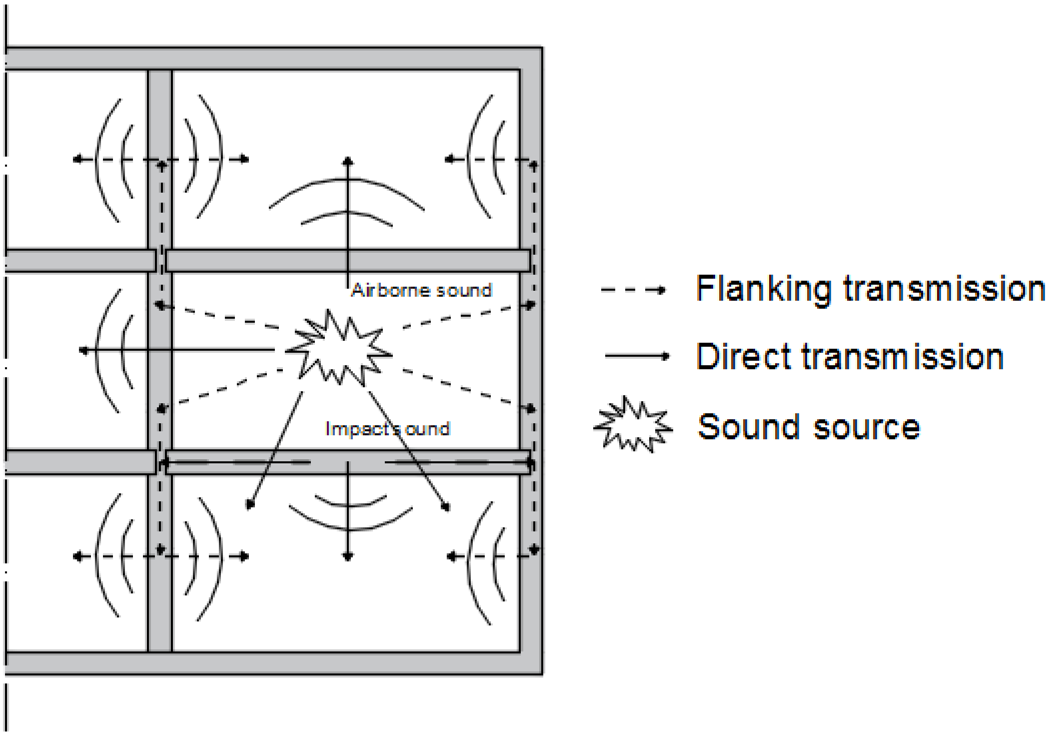

Figure 3.1 Sound transmission path in a building (based on The Scottish Government 2008).

3.3.1 Airborne sound transmission

Sound is waves (or energy) caused by vibrations in a medium (e.g. air or water). Airborne sound transmissions are pressure waves (e.g. voices, radio or television sounds) transferring through air from one space to another. The energy created from the sound source starts to set surfaces in the room (e.g. walls and floors) in vibration and the sound is spread to other effected building areas, see Figure 3.1. The level of noise depends on the properties of the dividing walls and the floor and neighboring constructions where some or most sound can be reflected or absorbed. An important factor is the weight of the construction. The sound isolation is better the higher mass the construction has. This is especially concerning lower frequencies, which is why it is a bigger problem in light weigh constructions. For beam and post structural systems which have many cavities, insulation can be incorporated to improve the sound (and thermal) isolation (Adelaide City Council n.d.; Scottish Building Standards 2013).

3.3.2 Impact sound transmission and vibrations

Impact sound and vibration is the most common problem in light weight structures of today. In this case the sound travels from a sound source (e.g. footsteps) through a building member (e.g. floor) it is in contact with, i.e. the sound source is in direct contact with a medium where the noise is spread through. The vibrations are then transferred to the structural parts of the building causing the neighboring room below to hear the noise due to radiation coming from the vibrations, see Figure 3.1. The floor and the floor finish properties have a great influence on how the sound will be transmitted and therefore should be carefully considered. (Adelaide City Council n.d.; Scottish Building Standards 2013).

3.3.3 Flanking transmission

Flanking transmission is often the most complex problem in light weight structures. It can be defined as noise transfer through openings around building members rather than through a building element, see Figure 3.2. Example of ways flanking transmissions occur are from the floor to the load bearing walls, through structural joints that are connected poorly and through the floor itself and its floor joist space (Acoustical Surfaces Inc. 2014). Some solutions for flanking transmission to have a low impact as possible; resilient layer on the floor and elastic isolators separating floors and walls (Ågren et al. 2012).

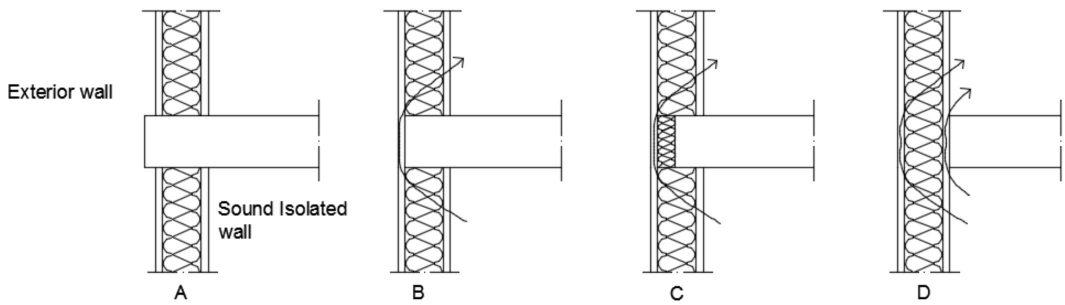

Figure 3.2 Example of flanking transmission through building members. A provides the optimal sound insulation while D provides the worst (Träguiden n.d).

4 Structural Systems

To ensure the stability of the building, different aspects have to be considered - both global and local. Essential checks have to be made concerning tilting and gliding, force distribution between the floors and between the building and foundation.

This chapter includes three types of methods for constructing with timber; balloon framing, platform framing and modules. Focus was on balloon and platform framing where two different structural systems are used, post and beam and stud walls. In addition to the construction systems, some commonly anchoring systems were studied.

4.1 Tilting and gliding

Tilting and gliding is checked in the global perspective and is caused by horizontal loads, which are mainly caused by wind but can also be created by leaning vertical elements. These loads need to be addressed in the form of tensile and shear stresses between the base plate and under the foundation. To ensure that the building is safe against tilting, the self-weight of the building and the concrete foundation must counteract the moment caused by lateral loading. The stabilizing moment due to the self-weight of the building should be greater than the tilting moment ($Ge > Hh$), see Figure 4.1. Gliding is checked by making sure that the shear stresses between the concrete foundation and the underground is smaller than the dimensioning strength of the material used as foundation. Vertical loads in the form of self-weight counteract the tilting moment caused by the horizontal loads. This is because the vertical loads are increasing the contact pressure and thereby also the friction against the foundation. When calculating tilting and gliding the vertical loads should be taken as a favorable effect in load combinations according to Eurocode (Girhammar et al.

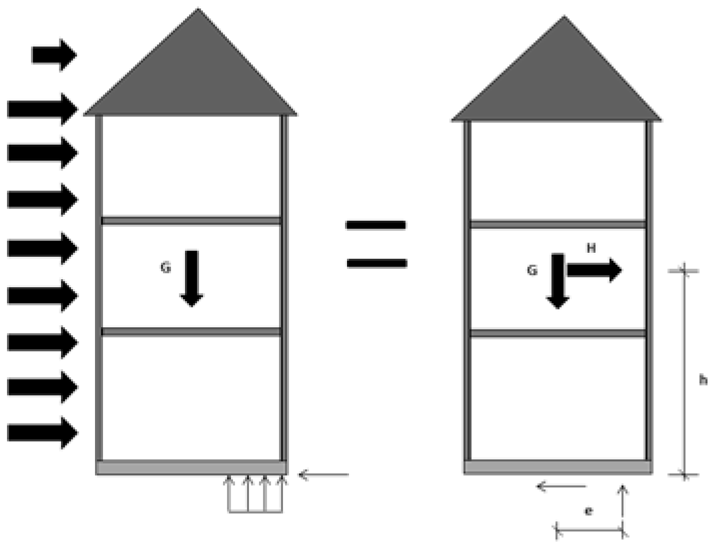

Figure 4.1 Stability regarding tilting and gliding for the whole building in a global perspective.

2010).

4.2 Force distribution in the building and its floors

The horizontal loads acting on the building are also acting on each floor creating forces at a local plane. The walls perpendicular to the wind direction on each floor, in both windward side and leeside, transfer the horizontal forces from the wind to the floor structure. When the horizontal loads act on the floors they are considered to be line loads along the floor edges. Due to inclination, the fictitious horizontal forces are added to the line loads, see Figure 4.2. The line loads acting on the floors are transferred by horizontal shear forces at the edges of the floor to underlying walls parallel to the wind direction, which in turn are transferring the horizontal forces to the foundation. These walls are called shear walls and are rigid and therefore capable of transferring horizontal forces from overlying roof or floors to the foundation in a plane parallel to the roof and floors. When being loaded the shear walls are in this way exposed to shear but also to bending, see Figure 4.3. The floors that are transferring shear stresses to underlying shear walls can be either placed on top of the wall or hanged on the walls, see Figure 4.4 (Källsner & Trätek 2009).

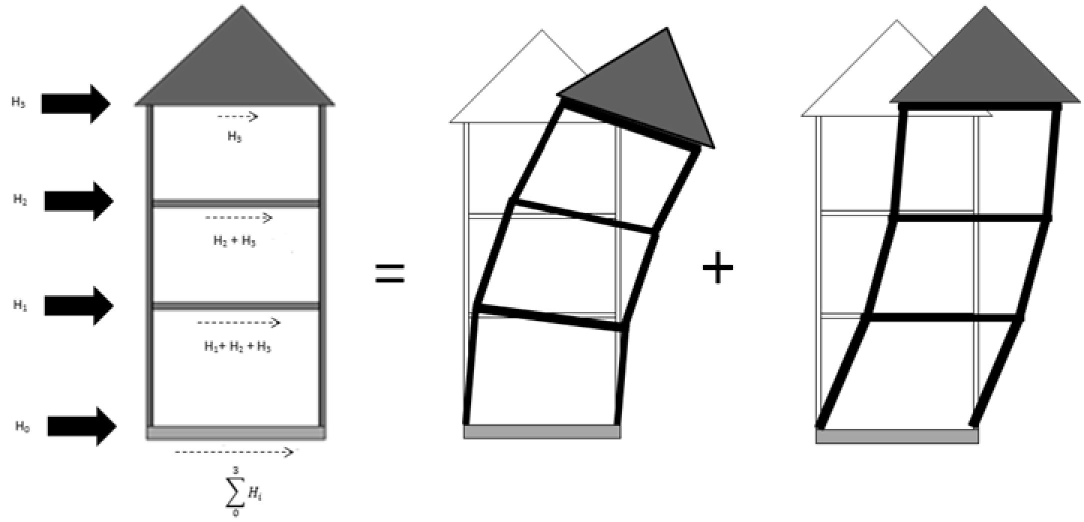

Figure 4.2 Stability regarding bending and shear deformation.

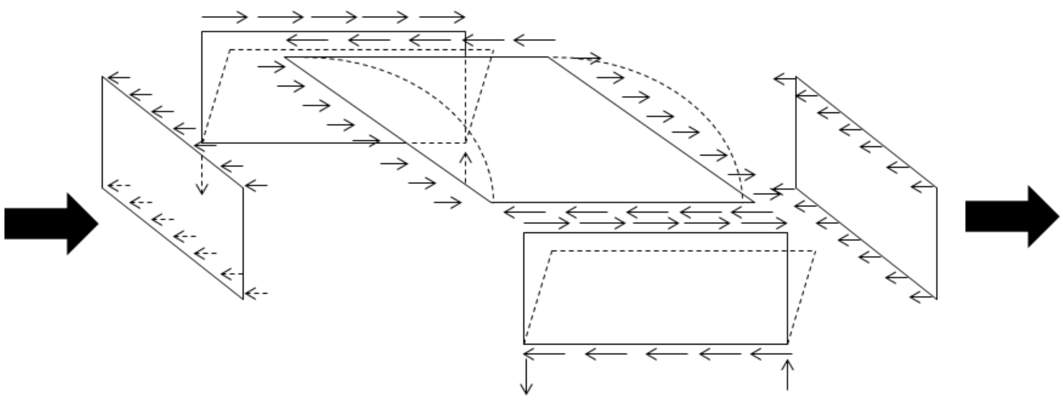

Figure 4.3 Transfer of horizontal loads through the wall panels. The dashed lines are showing the deformations.

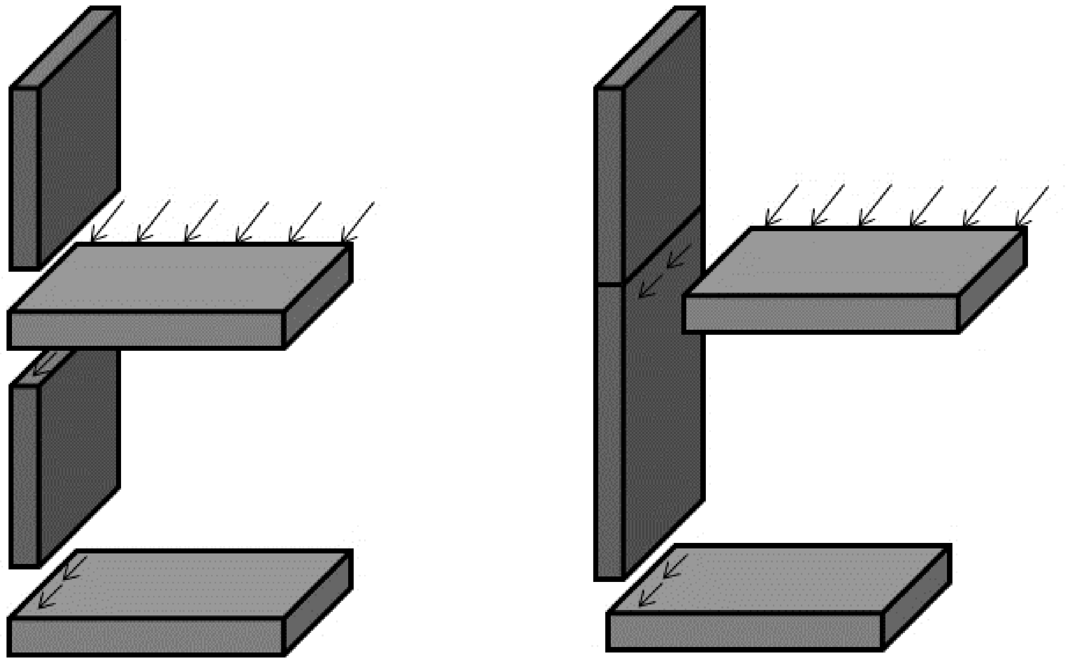

Figure 4.4 Horizontal forces are acting on the floors and being transferred to the floor below or the foundation.

Figure 4.4 above shows how the horizontal forces are transferred between the floors and the walls and Figure 4.5 shows in detail how a single wall takes the load and generating pressure and uplifting force. The configuration of the connections in the nodes between the floors and the walls are essential to ensure the assumed way the loads are acting. Except that the horizontal forces are generating shear forces, uplifting forces and compression forces are also generated in the wall elements. The uplifting forces can be fully or partially anchored (Källsner & Trätek 2009).

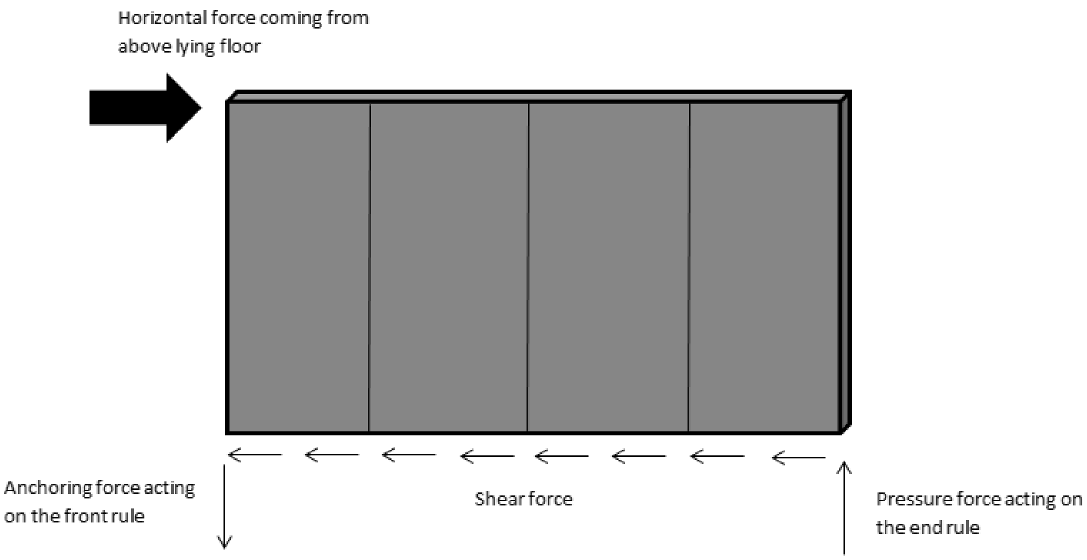

Figure 4.5 Force distribution on a simple wall.

To calculate the distribution of forces in the stud walls and stabilizing walls the method based on elastic theory is used, meaning that the deformations caused by the outer forces are reversible. The conditions for the walls, to be able to perform the calculation, are that the nodes between the walls and floors are considered to be joined and anchored to the substructure. It is very important that the anchorage force by the wall edge is transferred through an anchorage to the foundation while the walls or studs are considered to be stiff (Källsner & Trätek 2009).

4.3 Force distribution on the foundation

The foundation is placed between the building and the ground and is taking care of the forces brought down to the foundation by the horizontal and vertical stabilizing elements in the building. The top of the foundation is the part that resists the horizontal- and vertical forces while the bottom side of the foundation resists with the outer compression forces against the ground, see Figure 4.6. The load distribution on the concrete foundation should be dimensioned against moment and shear stresses (Källsner & Trätek 2009).

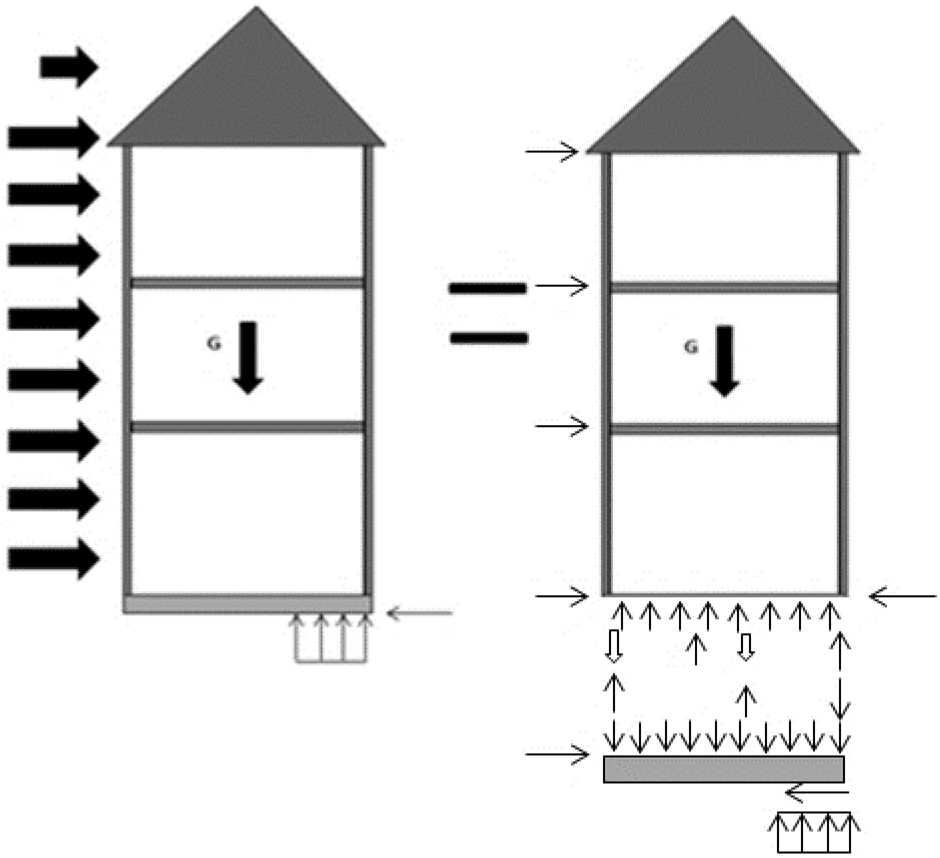

Figure 4.6 Schematic sketches over the force distribution in the concrete foundation slab.

The foundation slab must be prevented from moving at all times in order to transfer all the loads from the overlying structure to the ground. If stabilization of the slab cannot be guaranteed, installation of piles under the slab is required. Depending on the type on structure being erected, the soil conditions, load capacity and different view of the surroundings, there are several methods of piling using different materials as piles (Stål & Wedel 1984).

The most commonly used piles methods on the market are: lace-mounted pole, friction piles, cohesive piles and Franki piles. All methods are designed for different purpose and soils, and the piles made of different material such as steel, concrete or timber can be applied for all piling methods (Stål & Wedel 1984).

When piles are being installed the most common way is to knock or vibrate them down into the ground. Piles can also be pressed down to the ground by hydraulic power. In some cases concrete piles can even be cast on site. Predrilled holes are then made in the ground which later are filled with concrete to become piles (Stål & Wedel 1984).

Type of piles that are used to keep houses stable differs from building to building. For smaller houses steel plates are optimal but for larger and heavier buildings concrete piles are preferable. Even though multi-storey houses are relatively light structures compare to structures made out of steel or concrete, concrete piles are particularly better to resist uplifting forces since they can be piled down to great depths. This is essential for multi-storey timber houses due to their light weight where the pile resistance has to be able to counteract the uplifting force (Stål & Wedel 1984).

4.4 Loads

The loads acting on a building are calculated according to Eurocode 1. Self-weight, imposed load, wind and snow loads, which are causing tilting and gliding, are considered and combined after given rules in Eurocode, resulting in designing load combination which are applied in the design calculations (Källsner & Trätek 2009).

4.4.1 Wind and snow load

The determination of the wind load starts by finding out the location of the building. This is to determine the reference mean velocity pressure and to obtain the terrain category. When this is achieved the pressure coefficients on the external walls and roof can be determined where the pressure differ in different part of the building, which in turn depend on the dimensions. Likewise the wind pressure, or wind load, is determined on the windward and leeside which also differ depending on the height of the building.

Determining the snow load starts with determining the characteristic snow value based on the geographical location of the building. The characteristic snow value is then multiplied with the shape coefficient; taking into account the roof exposure and design/geometry obtain the design snow load value (Eurocode1 2013).

4.4.2 Self-weight and imposed load

The self-weight, also called dead weight, includes all loads that are relatively constant over time; walls, floors, roof etc. In short, all elements that are immovable in the building, are contributing to the self-weight.

The self-weight contributes to the stability of the building. The heavier a building is the more stable it is, but concerning the costs that follows with higher self-weight making a building very heavy is not the optimal solution for complete stabilization.

Imposed loads, also called live loads, are defined as temporary or moving loads that act during a short time in or on the building after its erection. The imposed loads include all variable loads and their probability of happening at the same time. Examples of imposed loads are people and movable objects such as furniture (Eurocode1 2013).

4.4.3 Anchorage (General)

All loads acting on the building generated by the wind and snow load, self-weight and imposed loads have to be transferred to the foundation slab and to the ground in order to keep the building steady. To make this possible the connections between the walls and the floors have to be strong enough to withstand both horizontal and vertical forces. The same applies to the anchorages between the foundation slab and the structure above it. The only challange is that the anchorage might have to be able to resist moment as well. This sets high demands on the anchorages which have to be stiff enough to keep the building safe and steady by resisting all possible forces acting on it.

4.4.4 Load combinations

When more than one load is acting on a building or a part of a building a load combination is necessary. This is to ensure the safety of the structure at different possible scenarios while being loaded to the maximum. Load combinations consist of one or several dead weights (self-weight) and live loads (imposed load), which are combined after the criteria favorable and unfavorable. In the load combination the load selected as the main is the one providing the greatest load effect (Eurocode1 2013).

4.4.5 Plastic design method

Eurocode proposes two methods of designing wall diaphragms called Method A and B, which are according to an elastic model. Another way to perform calculations are with a plastic method developed by Källsner.

What separates the two methods apart is that the plastic method gives the structural engineer greater possibilities of choosing how the stabilizing system should be designed. This is achieved by making it possible to change the force flow in the structure in order to maximize the utilization in different construction materials allowing higher tensile/uplifting forces. When designing according to the plastic method larger displacements are therefore allowed which should be checked for afterwards.

The method of plasticity includes two methods: one general and one simplified. The simplified method is designed so that the designing horizontal load resistance always is less or equal with the general method. Since the general method always gives higher or same horizontal load resistance as the simplified method it is more common and of greater interest to use the general method when applying the plastic method in structural design.

The conditions for applying plastic analysis are based on horizontal stabilization where the joints between walls and studs/columns have plastic properties. It is essential that there is a plastic relationship between the forces and displacements. To avoid brittle fractures, cracks or punching shear failure should be avoided (Källsner & Trätek 2009).

4.5 Balloon framing

This construction method belongs to the earlier methods within the area of timber systems. The method originates from the beginning of 1800 century when faster and more economical erections of timber houses were requested. It is believed that the first timber building erected with a balloon frame was a warehouse in Chicago in

1832 by George Washington Snow, this according to architectural historian Paul E. Sprague (Johnson 2007).

What is characteristic with the balloon method is that the framing members are constructed studs or columns running continuously from the foundation to the top of the building. The continuously running studs or columns form a large shell which can be used as weather protection when executing work inside the structure. This requires long timber elements for the upbringing. Back in the days this limited the number of stories to two because timber elements could not be made as long as wanted. Today there is knowledge on how to connect timber elements and make them longer resulting in constructions taller than two stories.

In a balloon structure the vertical loads are transferred to the foundation by the load bearing studs or columns. To keep the system stable against lateral loads, such as wind, the studs or columns are covered with outer sheathing. This results in fewer number of needed horizontal framing members, which also is distinctive for balloon framing (Johnson 2007). The horizontal members are jointed to the sides of the studs creating a floor (Tonks 2004).

A major advantage the balloon method had when it was gaining popularity as construction method for timber structures, was that due to its design it only used up one third of all the timber needed at erection compared to more common traditional jointed frames at that time. Because less timber was used, balloon framing was much faster than traditional framing methods and therefore less manual labor was needed. These attributes for balloon framing resulted in approximately 40% lower costs (Johnson 2007) and since the system required less material it was also lighter, light as a balloon, hence the name; Balloon framing (Understand Building Construction 2015).

Even though balloon framing had a great advantage considering costs, the framing method almost died out. One reason, also mentioned above, is the length of the lumber that could not be longer then the height of the tree (Understand Building Construction 2015). If using engineering wood products these can theoretically be made as long as possible but are restricted by the length of the means of transport. Another reason is fire. A balloon frame catches on fire much faster than a building designed with for example the platform method. This is due to the openings between the floors and the studs or columns are not blocking the fire. Instead it lets the fire to spread along the balloon-frame wall studs to the top. Horizontally the fire spreads through the floor joist bays (Fire Engineering 2014).

Today, balloon framing is mainly applied in construction of smaller houses. The method has not had its breakthrough for being applied in multi-storey houses, but despite that, construction methods similar or based on the balloon method have arisen.

One construction method, which has been developed by a Swedish company, is to let stabilizing walls go from the foundation up to the roof as one element. When the stabilizing walls are attached to the foundation, columns and beams are mounted on between them creating a stabilized "box" making it possible for the next step to create a roof and put floors. A post and beam system is here built with the balloon method.

4.5.1 Post and beam system

Beam and post system is probably one of the oldest building systems in the world. Mostly used in concrete and steel structures, but have been used in greater extent in timber structures the past decades (Tlustochowicz 2011).

A beam and post system is a flexible system giving great possibilities of variation, especially for multi-storey buildings. The system is used when large and open surface areas are requested inside the building, residential or non-residential, or when the facades are requested to have large openings. The system consists of columns, beams and floors making the erection of the system fast due to few member and joints (Canadian Wood Council n.d.). The parts can either be prefabricated or manufactured on site (Träguiden 2014).

In a post and beam system it is the columns that are transferring the vertical loads to the foundation while the beams are taking care of the horizontal loads. When designing the vertical timber elements it is important that the parts (posts/columns), which are transferring vertical forces to the foundation, have the same deformation properties so that the entire structure does not deform unevenly. It is also essential that timber elements that are built into the system or replacing old ones does not deform disproportionately in the vertical direction due to load or change in the moisture content (Träguiden 2014).

For horizontal loads, the post- and beam system has challenging problems with withstanding lateral load such as wind. For that reason bracing components are needed to keep the system steady which can be made by adding struts, wallboards, studs and/or trussing to the system (American Institute of Timber Construction 2012). The horizontal loads can then be transferred to the ground through the bracing elements which are placed in staircases or in the facades in order to not ruin the floor plan (Träguiden 2014).

When assembling the post- and beam system together different joints made out of metals can be used; metal connector plates, light metal connectors, bolts, screws, lag screws, nails, staples etc. (Canadian Wood Council n.d.). Even timber connectors can be used but are preferred in smaller timber constructions rather than in multi-storey buildings. This is because wood connections have not been tested in laboratory conditions in greater extent especially for use for multi-storey timber construction. Instead steel connections are more preferable to fulfill design codes (Timberpeg 2007).

4.5.1.1 Anchorage

Anchorage of a post- and beam system is normally made via connection to a concrete slab. The connections presented below are therefore designed to suit timber columns and concrete slabs and no other foundations such as timber or brick. There are different solutions on how to anchor post/columns to the concrete slab. Some are presented below:

Post Base with Steel Rod and Washer: One of the most popular ways to attach columns to concrete. A steel rod with threads fastened in the concrete. A hole is drilled in the column which later can be placed on top of the rod. The rod goes all the way up to a part of the column that has been sawn out. In the sawn out part the rod is bolted to a plate washer and later hidden with a plug. To separate the concrete and the timber and keeping water away, PVC or a steel plate are used, see Figure 4.7.

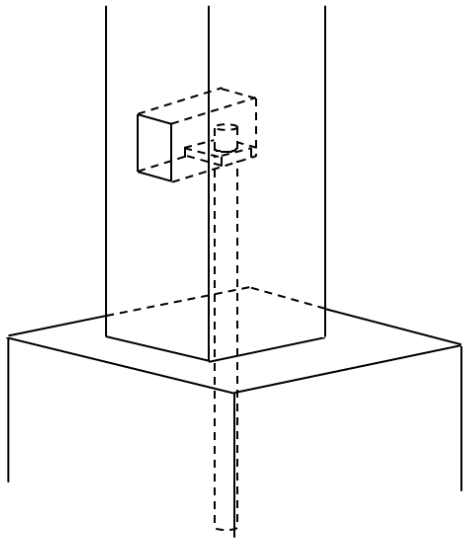

Figure 4.7 Post base with steel rod and washer

Post Base with Steel Boot: This connection is conventional and one of the simplest connections available. A so called boot is being used which the column stands on. The column is then bolted to the boot and the boot is bolted to the concrete foundation keeping the column steady, see Figure 4.8.

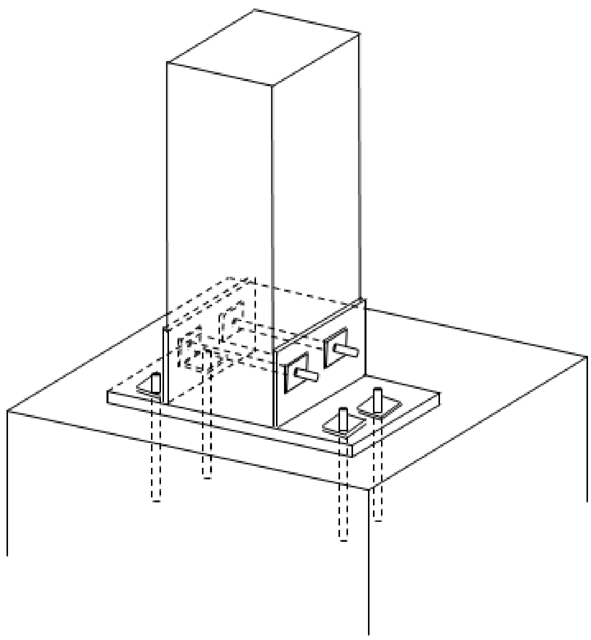

Figure 4.8 Post base with steel boot

Post Base with Steel Bracket: This connection is also placed on top of a steel boot but is partially concealed making it excellent to use when the column is placed adjacent to a wall. The steel boot or the steel bracket that is the right term, is fastened to the concrete foundation with bolts while the vertical part of the steel bracket is bolted to the column, see Figure 4.9.

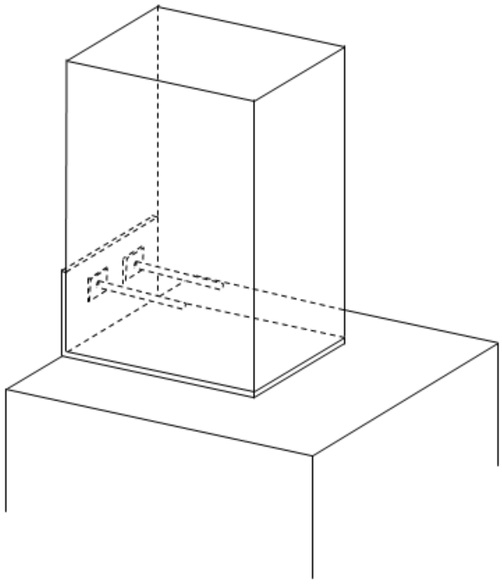

Figure 4.9 Post base with steel bracket

Post Base with Timber Linx Connector: The Timber Linx connector is a connection tube where a part of the connector is attached in the concrete foundation and the other part in the column, see Figure 4.10. The part of the connector which is in the timber column has a hole in the top. A pre-drilled hole is made in the timber column at the same level where the steel connector ends. In the pre-drilled timber hole and connector hole an expanding cross pin is inserted making the column stand steady (Vermont timber works 2015).

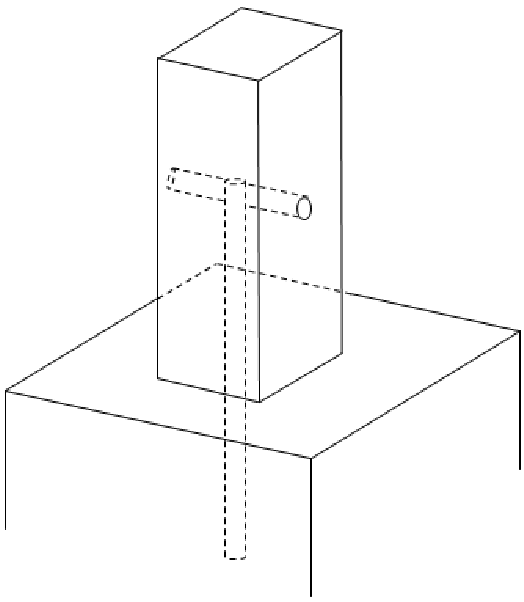

Figure 4.10 Post base with timber linx connector

4.6 Platform framing

Timber platform framing method originated in America 1833 (Crocetti et al. 2011) and is practised everywhere today. It was developed from balloon framing and is one of the most commonly used methods for constructing multi-storey houses (Structural Timber Association 2014).

This method implies that one storey is built at the time and can be of any type of structural system; columns and beams, stud walls, solid walls or a combination. After every floor is built, the floor is constructed on top of what will make the exterior walls. When the floor is fastened in place it will help to resist the lateral wind loads. The floor structure is then used as a platform where one storey high studs or exterior walls and interior walls are erected on and then the next level with floors and walls are built and this process continues to the desired number of levels. Finally the roof is erected on the walls of the top level (American Wood Council 2001), Figure 4.11.

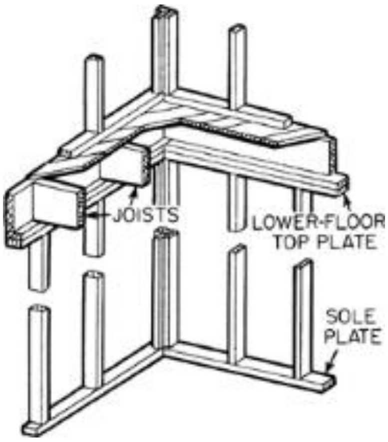

Figure 4.11 Platform frame (McGraw-Hill, 2003).

The timber members are shorter compared with when balloon framing is used which makes it easier to handle and transport since one floor height is built at a time. Furthermore, since platform framing method allows usage of timber engineering products, the sizes and dimensions of the members are not affected by the construction method but rather by transportation and erection issues.

Using platform framing method and constructing floor by floor is a relatively fast and easy erection method where the builders have a solid floor to work on. The timber members are smaller / shorter than if balloon framing is used which makes the handling of the material easier. Furthermore, less workforce is needed when using platform framing which can lower the construction costs. This method to construct does not require fire protection during construction since each floor creates a fire block (Structural Timber Association 2014).

A disadvantage with using platform framing method is vertical shrinkage in the horizontal oriented grains on the floor system which is caused by drying of the wood (DoItYourself 2015). In addition, weather protection during the construction time is important since the construction is exposed to the outdoor environment until the roof is placed.

4.6.1 Stud walls

A common way to construct multi-storey buildings in timber with the platform method is with vertical wall studs. The walls can be constructed to carry vertical loads or be non-load bearing. The spacing between the studs is usually 600 mm with a dimension of 140 mm x 38 mm for external walls. However, the dimensions vary depending on building type and the thickness of thermal insulation. For load bearing wall panels consisting of vertical studs the vertical forces from walls, floors and roofs are carried. Internal walls can in a building be constructed to resist some vertical and horizontal loads (Structural Timber Association 2014).

To resist lateral loads (racking resistance) such as wind, the wall studs are sheathed with panels and act as a continuous wall diaphragm. The vertical studs are then carrying axial and lateral wind loads. To connect the studs together, horizontal studs or top/bottom rails are used and usually have the same size as the vertical studs (Structural Timber Association 2014). The bottom rail or sill plate at the bottom of the wall is the part that is connected to the concrete foundation. In order to prevent movements from for example wind, settlement and earthquakes, the sole plate must be steadily anchored to the foundation. This is commonly executed with tie straps and anchor bolts (House Plans 2013).

To transfer vertical and horizontal loads around openings, doors and windows, lintels, cripple studs and opening studs are used. See Figure 4.12.

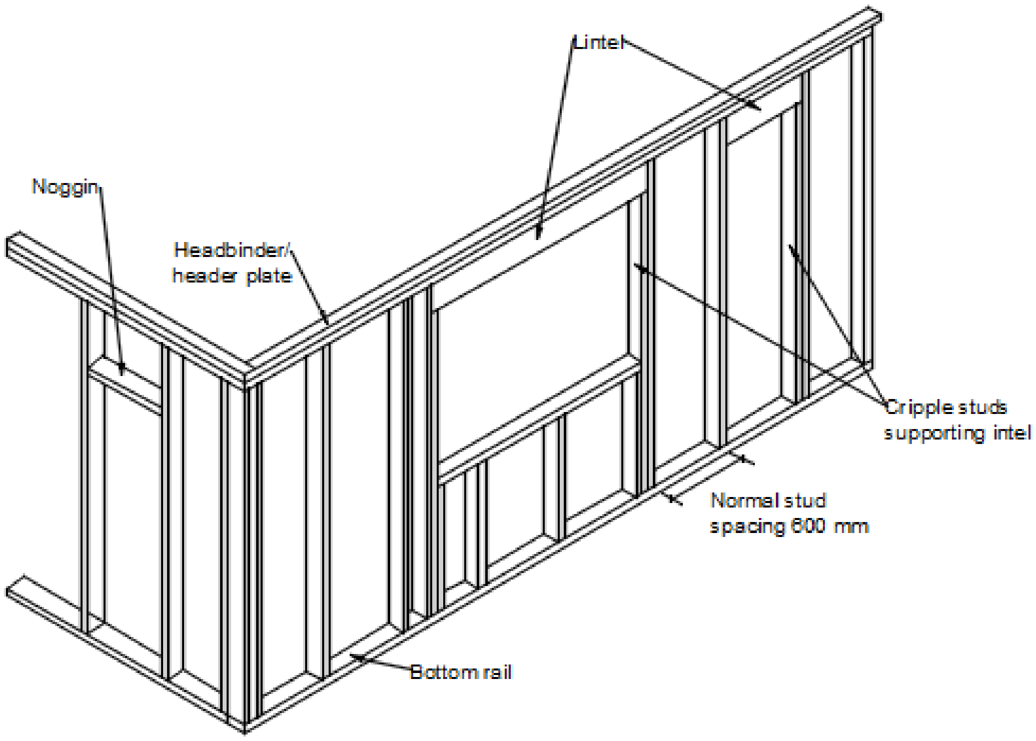

Figure 4.12 Stud wall and its components.

In the Nordic countries the light timber frames are usually prefabricated in a factory while in America the frames are produced on-site (Crocetti et al. 2011). The possibility to produce the framing members in a factory reduces the total construction time and costs and the quality improve due to a production in a more controlled environment (Martinsons n.d.).

Due to the frames being light, vibrations and deflection problems might occur. To prevent this acoustic insulation is used in the walls, floor slabs and roofs. In addition, an alternative or combined way is by using engineering wood material such as glulam, CLT or LVL with higher density for the joists to help reduce the acoustic problems (Crocetti et al. 2011).

4.6.1.1 Anchorage

Shear walls are subjected to uplifting forces and shear from the wind, see Figure 4.13. In addition to these, shear forces are acting between the timber members.

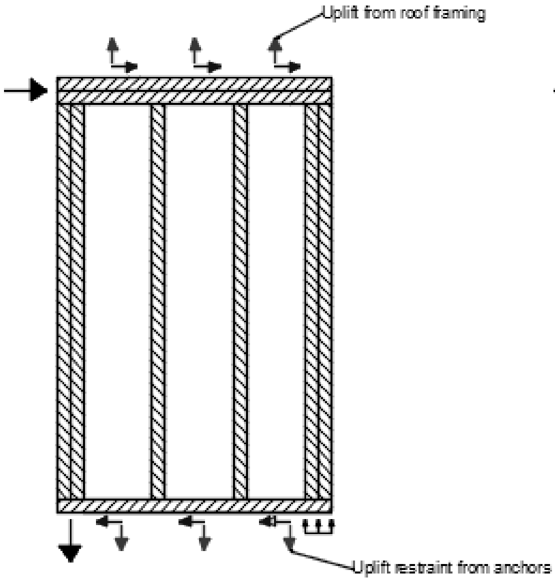

Figure 4.13 Resisting forces from wind in a shear wall (based on Vilasineekul 2009).

To resist the uplifting forces of the shear walls causing overturning of the building, the walls need to be anchored. Commonly there are three sorts of anchorage systems for stud shear walls; embedded hold-downs, hold-downs with threaded anchor and threaded rods with bearing plates (civil + structural ENGINEER 2011). See Figure 4.14.

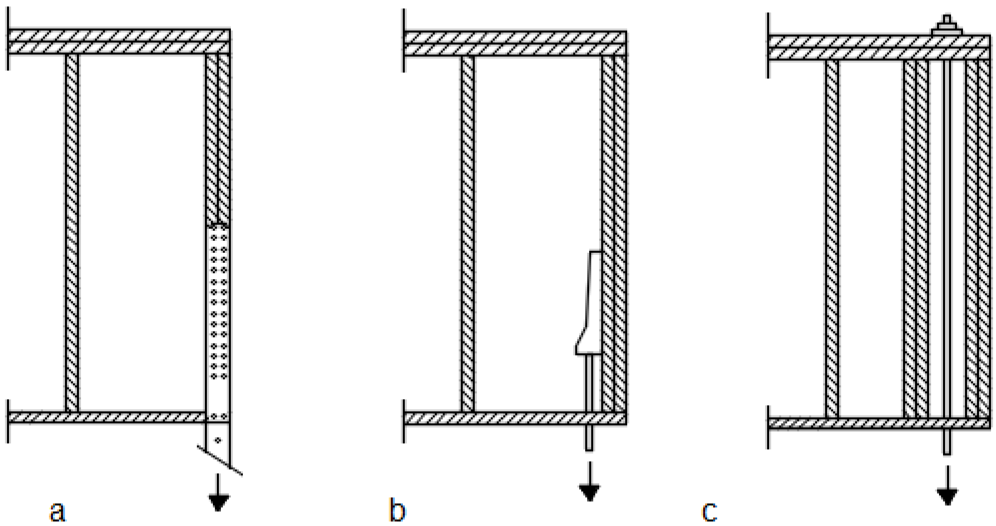

Figure 4.14 Commonly used anchorage systems for shear stud walls (based on Vilasineekul 2009)

Hold-downs is a steel device which are mounted at the end of a shear wall and provides resistance against uplifting forces and overturning moments. Along the timber post the hold down is fastened with screws and/or nails and bolts (Homebuilders' Guide 1999).

The embedded hold-down is usually nailed to the framing and partially embedded into the concrete, see Figure 4.14a. Hold-downs with threaded anchor are placed at the end of the wall and tie the shear wall to the foundation, see Figure 4.14b. Figure 4.14c shows threaded rods through a shear wall and fastened with a bearing plate above. This system is usually used in multi-storey buildings (civil + structural ENGINEER 2011).

The shear walls in a timber frame building are subjected to uplifting forces and shear forces, see Figure 4.13. Anchor bolts are designed to resist horizontal shear. They usually do not transfer any vertical loads to the foundation compared with hold-downs that tie the vertical end stud to the foundation. The dimensions of hold-downs are larger due to larger concentrated forces. Figure 4.15 shows a section of anchorage with anchor bolt. This type of connection can cause deformation in the wood due to compression perpendicular to the grains (Girhammar et al. 2010).

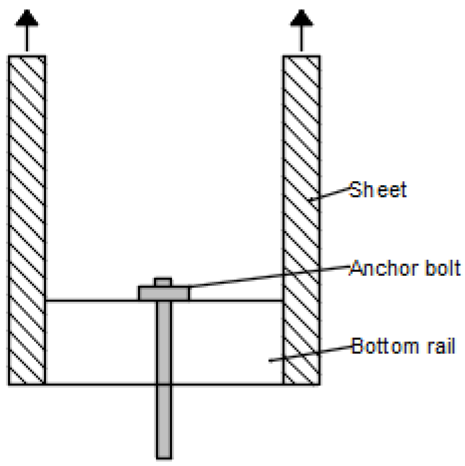

Figure 4.15   Anchorage with anchor bolts (based on Girhammar et al 2010)

4.7 Modular systems

Multi-storey timber framed module houses or volumetric elements are prefabricated buildings. Each volume box has six sides where the floors and the ceilings are hung onto the vertical load-carrying walls. The interior parts such as claddings, services and equipment for example kitchen furniture, are as well mounted. Before transporting the modules to the site, they are weather protected. On the site the modules are assembled next to and stacked on each other with the help of a dub. Due to the light timber framed construction, regulations regarding acoustics is fulfilled with polyurethane tape or Xylodyne which is applied on top of the studs. Polyurethane tape and Xylodyne are expensive and are therefore only applied on the studs and not along the rim. There is also a possibility to use solid wood which will provide stiffer stabilizing walls instead of light frame systems (Crocetti et al. 2011).

A module can be designed and constructed as a single or several rooms and as well an entire apartment. Since the houses are assembled in factory the construction time is shortened on-site. From the factory the volume elements are transported to the site and lifted to place with cranes and then fastened. The on-site process is fast compared with the platform framing (Setra 2013). These types of houses are mostly student and senior houses. Since modules consists of larger members, the restrictions of the dimensions of the volumetric elements are often limited to 4.15 m in width, 13 m in length and 3.10 m in height due to manufacturing and transport possibilities (Crocetti et al. 2011).

4.7.1 Anchorage

The most common way as anchoring method for modular systems is to take advantage of their self-weight. Nail plates are used to put together the module elements which are counteracting the uplifting forces created by the wind forces. The entire system is anchored to the foundation using steel brackets where bolts or screws are used (Giang & Moroz 2013).

5 Connection Systems and Anchorage

Anchoring a multi-storey timber building to the foundation has to be executed in order to meet the requirement of safety and stability. It is complicated since the elements being anchored have to be able to transfer both horizontal and vertical loads to the foundation. In some cases anchored elements are fixed and also have to be able to resist moments.

Different types of fasteners can be used to anchor multi-storey houses in timber to the foundation. An anchoring system that is gaining more popularity in the construction market is glued-in rods. Two of the most common anchorage systems used today are slotted-in steel plates and hold-downs, which are all described more in detail below.

5.1 Glued-in rods

Glued-in rods, see Figure 5.1, are joints that have been developed since 1980. Even though improvements have been made, there is still no standard on how to perform calculations for this type of connection in Eurocode. However, there are guidelines that are applied in various countries but these guidelines have great differences.

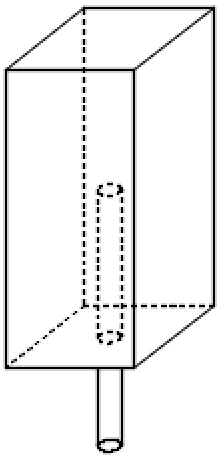

Figure 5.1 Timber element with glued in rod.

Glued-in rods are hybrid joints consisting of timber, rods and adhesive. This type of joint is appropriate for both new constructions and for strengthening and has therefor become more popular. Due to the rods being surrounded by timber it provides good fire resistance and has an aesthetic appealing look. When used as reinforcement the glued-in rods have been proven to prevent cracks in high stressed areas perpendicular to the fibres and in shear, e.g. curved glulam beams. Bonded-in rods (usually threaded at the edges of the rod) allow for the forces to be transferred efficiently through the load bearing members in the center parts of the cross-section (Tlustochowicz et al. 2011).

For joints that have to withstand larger forces glued-in rods are commonly used, especially bonded-in rods connecting glulam or LVL members. The joints have (if designed well) an efficient load bearing capacity. The rods can be fixed parallel or perpendicular to the fibre direction and provide high strength and stiffness properties. Glued-in rods can be used as an anchorage between timber and a concrete foundation for example, but as well for a timber-timber connection or timber-steel (Tlustochowicz et al. 2011).

The advantage with using a steel rod is the possible failure mode of ductility which means that the failure mode for the connection preferably should be yielding of the steel rods. The rods are usually made of steel but with recent technology the rods can be made of FRP (fibre reinforced polymers) dowels or hardwood. If steel rods are designed as the failure mode i.e. weakest link, a uniform distribution of forces can be assumed (Tlustochowicz et al. 2011).

The adhesive for the joint is an important part. Adhesive joints have been used for a long time in for example finger joints. The pull-out strength is affected by the type of adhesive used in the joint. Pull-out tests that have been carried out have resulted in a higher strength for adhesives based on fibre reinforced phenol-resorcinol (PRF) following by polyurethane (PUR) and epoxy (EPX). What should be noted is that the method used for constructing the joint is dependent on the type of adhesives and its properties. For example if the diameter of the hole is bigger than the rod the adhesive that is used have to have a viscosity property to be able to fill the gaps. To avoid brittle failure the bonded line with adhesives should not be the failure mode. The manufacturing and production process should be controlled to assure high quality. This is considered when making choices concerning geometrical properties and method of producing the joint. Today, in countries that apply glued-in rods, the most common adhesives are 2-component PUR and EPX (Tlustochowicz et al. 2011).

Furthermore, State-of-the-art review on timber connections with glued-in steel rods by Tlustochowicz et al., a classification system concerning the mechanical behavior of joints with glued-in rods is proposed. Important factors that are affecting the performance of the joints are the geometry, the material and the loading and boundary conditions. See Figure 5.2 for more detailed proposal (Tlustochowicz et al. 2011).

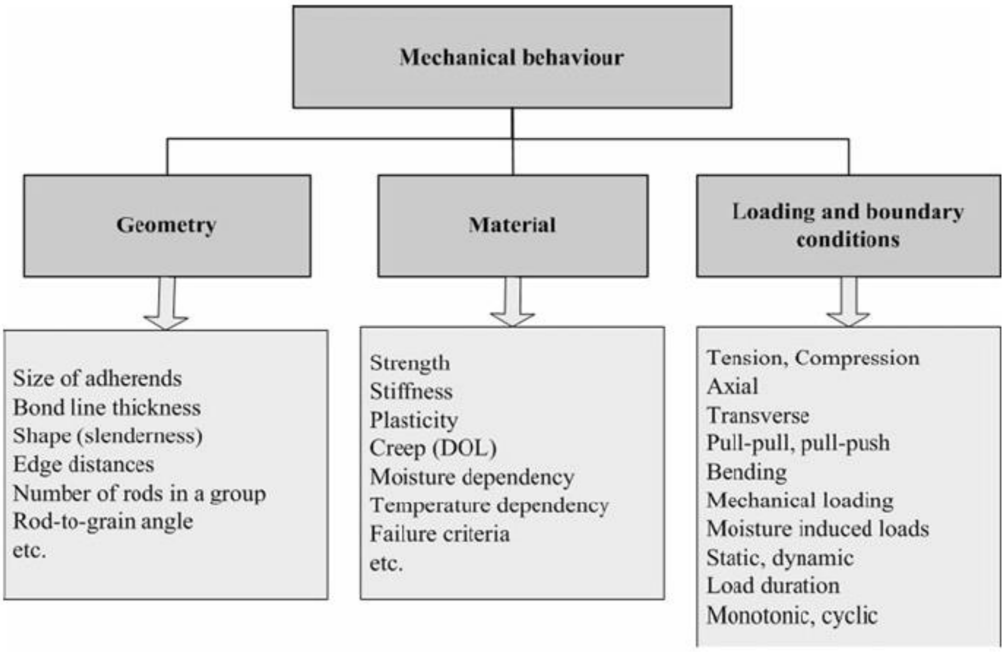

Figure 5.2 Classification chart for glued-in rod connections (State-of-the-are review of glued-in steel rods, Tlustochowicz 2011).

There are five failure modes for this type of joint, see Figure 5.3; shear failure along the rod (a), tensile failure (b), group tear out (c), splitting failure (d) and yielding of the rod (e). Glued-in rod tests that have been carried out made of single rod joints, show that a small edge distance can cause splitting failure, shear block failure or weaker pull-out strength.

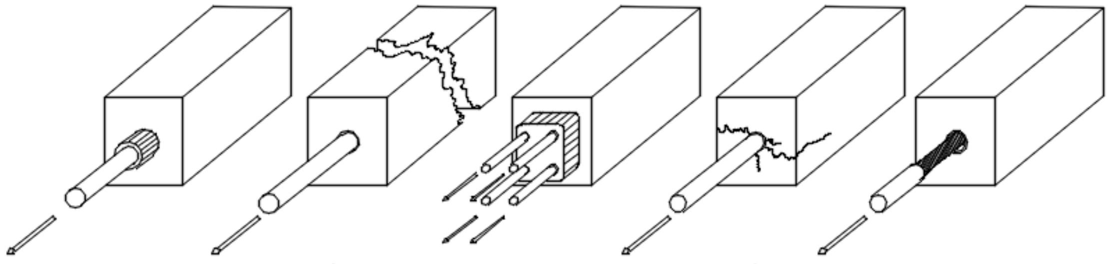

Figure 5.3 Failure modes for glued-in rods (Based on Tlustochwicz 2011).

Even though studies have shown a strong correlation between the bond line thickness and the performance of glued-in rods, a relationship is hard to determine due to the tests showing different behavior in e.g. ductility. Other factors and its effect on glued-in rods such as density of the wood, moisture content, temperature, load duration and loading in angle to the grain have not been determined and further research are needed (Hunger et al. 2013; Tlustochowicz et al. 2011). In addition, due to the differences between the guidelines for glued-in rods, an agreement is needed for a unified design approach.

5.1.1 Design

As mentioned before there are no approved calculations or models concerning glued-in rod connections in Eurocode. However, many projects and tests that have been carried out which have led to varying proposals of distances between the rods and minimum edge distances parallel to the grains. See Table 5-1 for recommended distances.

Table 5-1 Comparison between different standards concerning edge distance and distances between rods parallel to the grain (Tlustochwicz 2011).

Rods parallel to the grain	prEN1995:2001	DIN 1052:2004-08	STEP1	French Professional Guide
Distance between rods	4*d*	5*d*	2*d*	3*d*
Edge distance	2.5*d*	2.5*d*	1.5*d*	2.5*d*

prEN 1995:2001 (a developing European standard) suggests a distance between the rods of *4d* where *d* is the diameter of the rod. In other countries it varies between 2*d* for STEP 1 (Netherlands) to 5*d* for DIN 1052:2004-08 (Germany). The proposed edge distances are more similar, 2.5*d* (Tlustochowicz et al. 2011).

Determining the pull-out strength is as well different for the standards and an agreement has not yet been concluded. However, some suggestions of formulas have been summerized in the report State-of-the-are review of glued-in steel rods by Tlustochowicz (2011) and in Comparison of design rules for glued-in rods and design rule proposal for implementation in European standards by International council for research and innovation in building and construction (Hunger et al. 2013;

Tlustochowicz et al. 2011). The pull-out strength can be summarized in the equation (5.1) below.

$$R_{ax.k} = \pi \times d \times l \times f_{v.k} \qquad (5.1)$$

where:
$R_{ax.k}$ =Characteristic pull-out strength
d = diameter of the rod
l = anchorage length
$f_{v.k}$ = shear strength parameter

5.2 Slotted-in steel plates

Steel plates are often used as connectors in timber structures. Especially when heavy and large structures have to be mounted onto a concrete slab and when timber elements have to be jointed together. This has mainly to do with their efficiency of carrying high loads in the joints but also due to steel plates being easy to install and maintain. Steel plates are in contrast a bad solution when it comes to being fire resistant. In case of fire, steel becomes very hot which leads to loss in strength. The steel has therefore to be protected against fire and heat, either by fire protecting paint or by building the steel into the timber.

Embedding the steel plates in the timber is the most preferable solution when considering fire resistance and appearance. The steel plates are installed after making slots in the timber member and are then pre-drilled and so is the timber member surrounding it. The installation is completed with dowels being inserted in the pre-drilled holes (Crocetti et al. 2011). Examples of two different slotted-in steel plates can be studied in Figure 6.4.

For fastening timber columns to a concrete foundation slotted-in steel plates are preferable due to steel being easy to fasten to concrete. From a sustainable point of view these types of connections are durable due to their capability to keep water away preventing the connection to weaken.

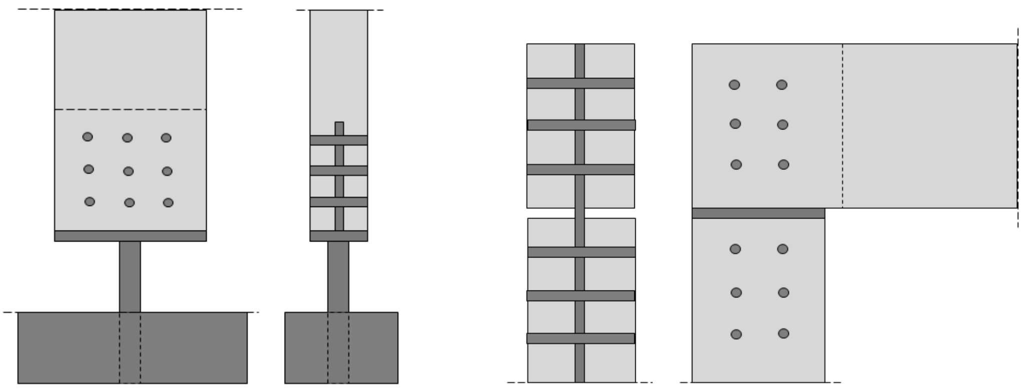

Figure 5.4 Slotted in steel plates where (a) shows a slotted-in steel plate to a concrete foundation and (b) between timber elements

An important thing to have in mind is the thickness of the steel plate that has to be thick enough to be able to withstand pressure from the dowels without starting to yield (Associated Glued Laminated Timber 2014).

When designing slotted-in steel plates the strength of the dowels used as connectors are determined on the basis of theory of plasticity. The design regulations can be found in Eurocode. The theory of plasticity predicts the strength in an accurate way when the load is applied in the grain direction, but not if the load instead is applied in the transverse direction. In that case plastic failure may occur which is most likely to happen in the connections/anchorages. To make sure that ductile behavior is achieved in the timber small dowel diameters are recommended. Another condition to obtain ductile behavior is to prescribe minimum spacing between the dowels and the distance to a dowel and edge. Something to be aware of is when determining the minimum spacing is that no distinction is made between different timber materials (CIB 2008).

Slotted-in steel plates can be designed with more than just one plate. Two or three plates are also common which of course result in different failure modes. For a single slotted-in steel plate there are three failure modes while for two and three slotted-in steel plates there are six each. The different failure modes can be seen Figure 6.5 where failure occur either in the timber, dowel or both due to tensile force. The failure modes of slotted-in steel plates are derived according to the theory of plasticity (Sawata et al. 2006).

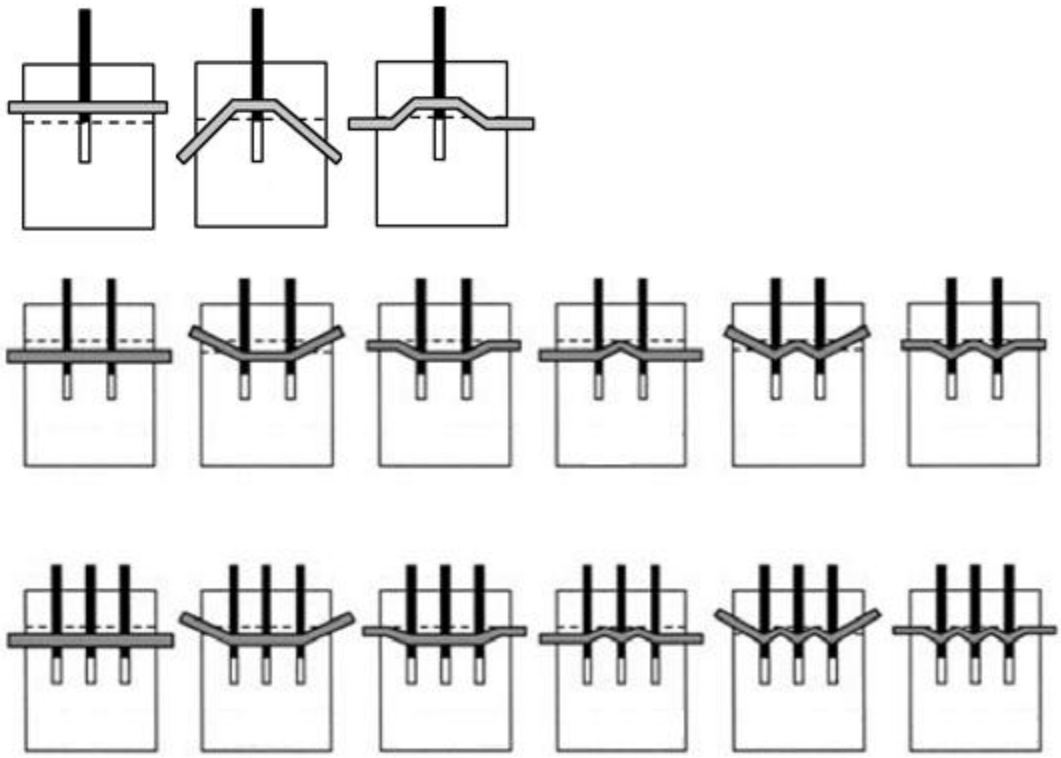

Figure 6.5Failure modes for one-, two- and the slotted-in steel plates (Sawata et al. 2006) and (Crocetti, R., Johansson, M., Johnsson, H., Kliger, R., Mårtensson, A., Norlin, B., Pousette, A., Thelandersson 2011).

5.3 Hold-downs

Hold-downs connect the wooden structures that are subjected to uplifting forces to the concrete foundation. It consists of an angle bracket with threaded bars and are fastened with screws and nails on the wooden part, see Figure 5.6, (Simpson Strong Tie 2015; Rothoblaas n.d.).

Figure 5.6 Hold-down (Simpson Strong Tie 2015).

In the Master's thesis 'Multi-storey timber frame building - Modelling the racking stiffness of timber-frame shear-walls' by Hoekstra (2012), tests that have been carried out with hold-down connections where the results showed that the connections can provide high stiffness (Hoekstra 2012). Hold-downs are proven to be able to resist high tensile stresses with the use of high-strength steel. The installations of the connection is easy due to fastening with screws and nails on the wooden part and bars with nuts going down in the concrete foundation (Rothoblaas n.d.). The hold-downs can be directly bought from a manufacturer or designed specifically for a certain project. The washers that are used are usually designed for ULS, which gives an increase in strength.

6 Reference building

In this Master's thesis two reference buildings, one with four floors and the other with seven floors, were designed based on given information from a Swedish company specializing in timber constructions. The company contributed with drawings and information (e.g. self-weight of material and type of material) from previous projects. Today, the company mostly constructs multi-storey buildings with four floors. In the company, anchoring of the shear walls is usually performed with nails in the sole plate where the exterior walls are made of glulam. For the stabilizing inner walls the anchorage systems used consist of U-steel profiles. The connections are calculated with a plastic method. The highest anchorage forces that have been needed for their constructions are approximately 80 kN and the large tensile forces are anchored with tension rods that are connected to welded steel plates in the foundation. These forces were not compared with the designed buildings in this thesis.

Reference building 1 and 2, see Figure 6.1 and Figure 6.2, were constructed with the platform method. The floor plans with stabilizing walls were the same for the two buildings but the height differed. The dimensions of the reference buildings were 16 x 24 x 11.4 (B x L x H) m for the four storey building and 16 x 24 x 20.1 m for the taller one. Each floor was 2.4 m high (inner height).

Every floor was carried by load bearing walls and columns, which in turn carried another floor on top. The floors mainly consisted of beams made of LVL (Laminated Veneer Lumber) and other components were gypsum and air gaps in order to fulfill the requirements of a timber floor. In addition, steel beams IPE360 were integrated in each floor in order to make the building more stable which has a higher density than timber and able to carry larger loads. The interior shear walls, see Figure 6.3, were made of CLT and were 250 mm thick. The shear walls in the corners consisted of studs and panels with a total thickness of 395 mm. The staircase and elevator shaft were made of CLT.

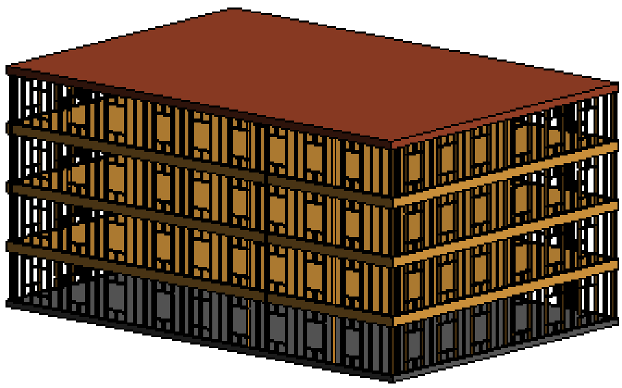

Figure 6.1 Reference building 1 with four stories.

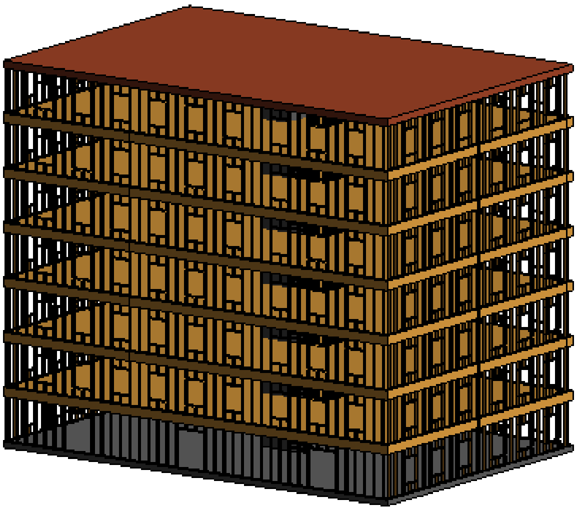

Figure 6.2 Reference building 2 with seven stories.

The floor plans for each floor were the same for both buildings. Figure 6.3 shows the floor plan and placement of shear walls and columns.

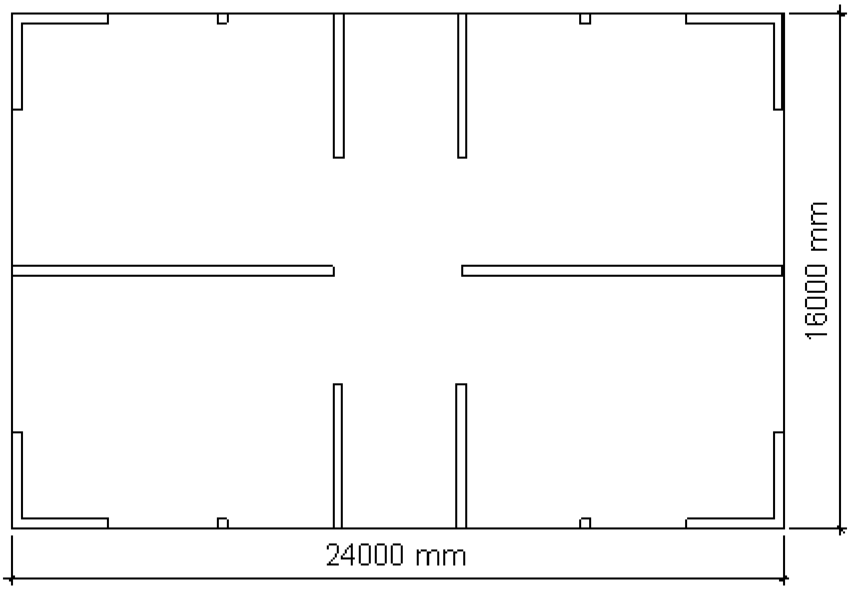

Figure 6.3 Floor plan and placement of shear walls and columns.

6.1 Stability calculations

Stability calculations were made for both reference buildings. The controls that were made were tilting and overturning of the building and uplifting of the shear walls due to wind load. The buildings were then compared with each other in regards to tensile forces of the shear walls that needed to be anchored.

6.1.1 Tilting

The self-weight was considered as a favourable load since the weight of the building helps to resist the wind loads that can cause tilting of the building. When calculating the resisting moment, the self-weight of the foundation was included due to the large contribution to resist tilting. The tilting moment will be the largest when there is no snow load or imposed load and was therefore not included in the calculations of loads acting in the vertical direction. Furthermore, due to unintended inclination the tilting moment will be larger and naturally the higher a building is the larger effect the wind load will have.

The stability calculations, see Appendix A and B, for both reference buildings shows that the self-weight of the buildings were large enough to resist the tilting moments when the wind was coming from both directions (on the long respectively the short façade). Figure 6.4 and 6.5 show the magnitude of the compression forces that were needed due to the much larger self-weight compared with the wind load.

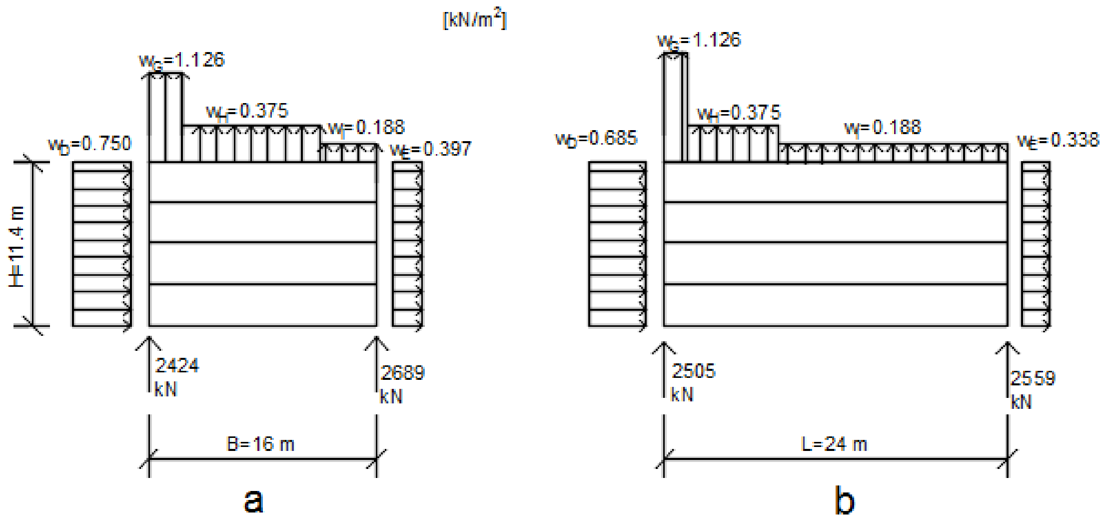

Figure 6.4 a) Wind coming on the long facade b) Wind coming on the short façade

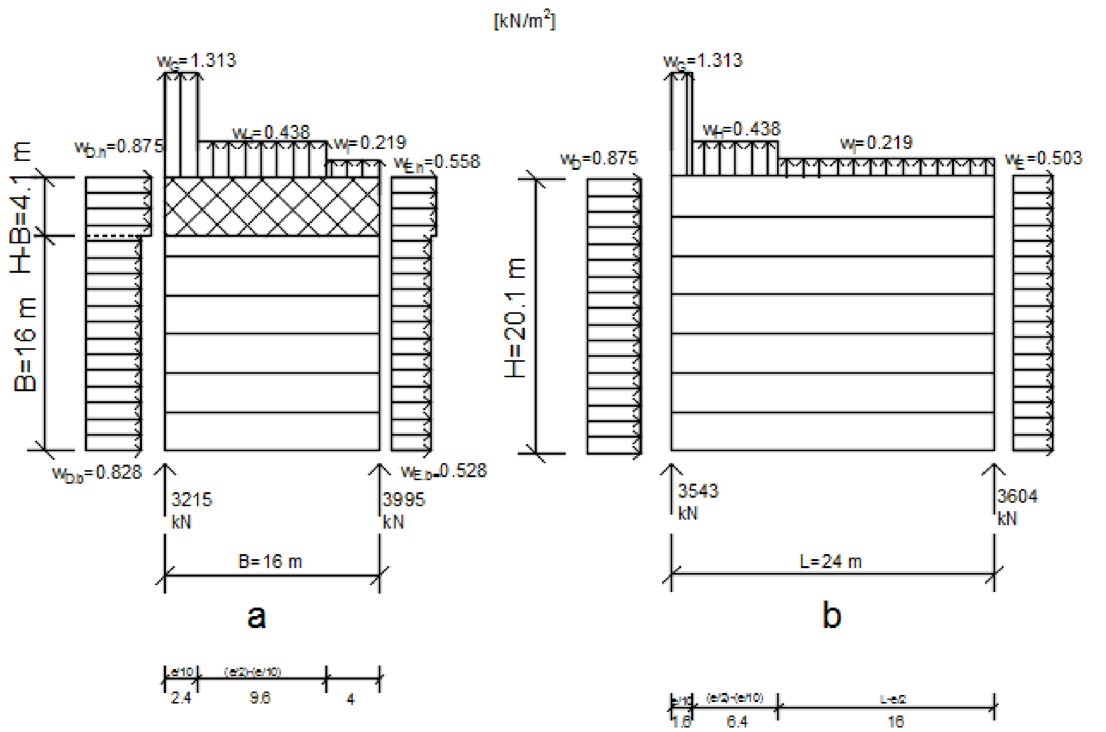

Figure 6.5 a) Wind coming on the long facade b) Wind coming on the short facade

6.1.2 Uplifting forces on shear walls

As mentioned before the shear walls are subjected to uplifting forces caused by the wind. The tensile forces that need to be anchored are larger the taller the building is. The anchorage for the uplifting force for multi-storey houses in timber can therefore be very large, leading to difficulties to execute and expensive costs.

Two stabilizing shear walls (1 and 2) of approximately 10 m in length were designed to resist the wind load when coming on the short façade. See Figure 6.6 for the placement (and names) of the shear walls. When the wind was coming on the long façade, four shorter walls (3-6) with a length of approximately 4.5 m resisted the wind load. Shear walls 1-6 were made of CLT. In addition to these walls two shear walls in each corner were placed (7-14) which was especially needed for the building with seven stories. These exterior shear walls were made of studs with OSB-panels. Walls that were placed horizontally in the floor plan in Figure 6.6 were resisting the wind load coming on the short façade direction. The walls that were placed vertically resisted the wind load coming on the longer façade.

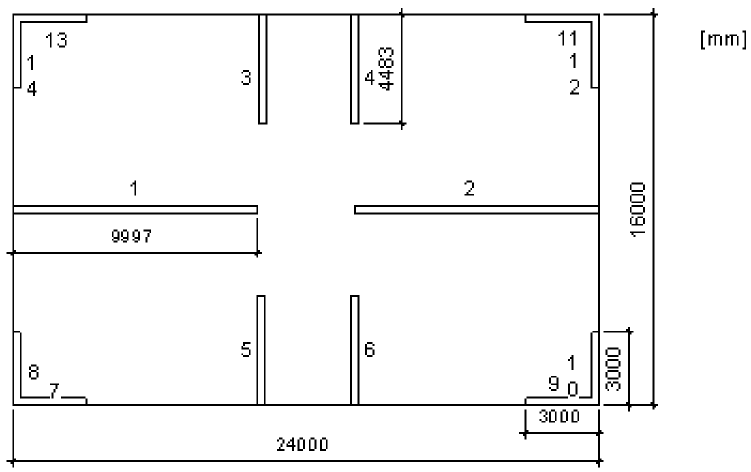

Figure 6.6 Floor plan of shear walls.

The placement of the shear walls gave a symmetric floor plan, see Figure 6.7. With a vertically dividing line the same load was resisted by shear wall 1 and wall 2, wall 7, 9, 11 and 13 equally carried the same load. Looking at the horizontally dividing line, wall 3, 4, 5 and 6 carried the same load and wall 8, 10, 12 and 14 in the same way. Due to the symmetry, calculations were therefore only made for shear wall 5, 8, 1 and 13.

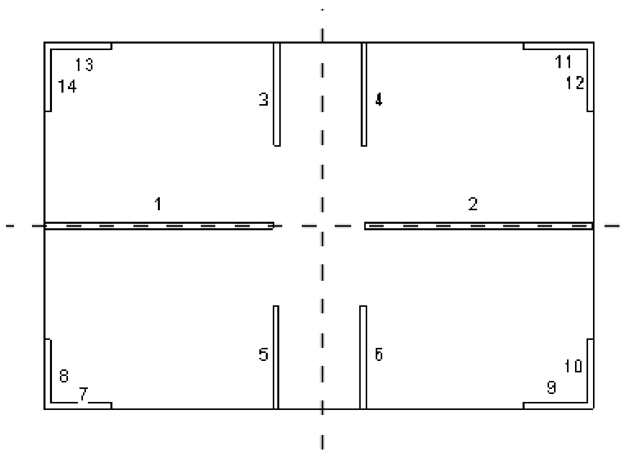

Figure 6.7 Floor plan with symmetry lines.

When calculating the uplifting forces the critical place was on the bottom floor, this is where the uplifting force was the largest. To find the uplifting forces the shear walls were considered as supports on a beam where the wind load was a distributed load over the beam. Figure 6.8 shows a section of the long façade with stabilizing walls (thicker lines) considered as a beam when the wind was coming on the long side of the reference buildings. Figure 6.9 shows the same section but on the shorter sides of the buildings.

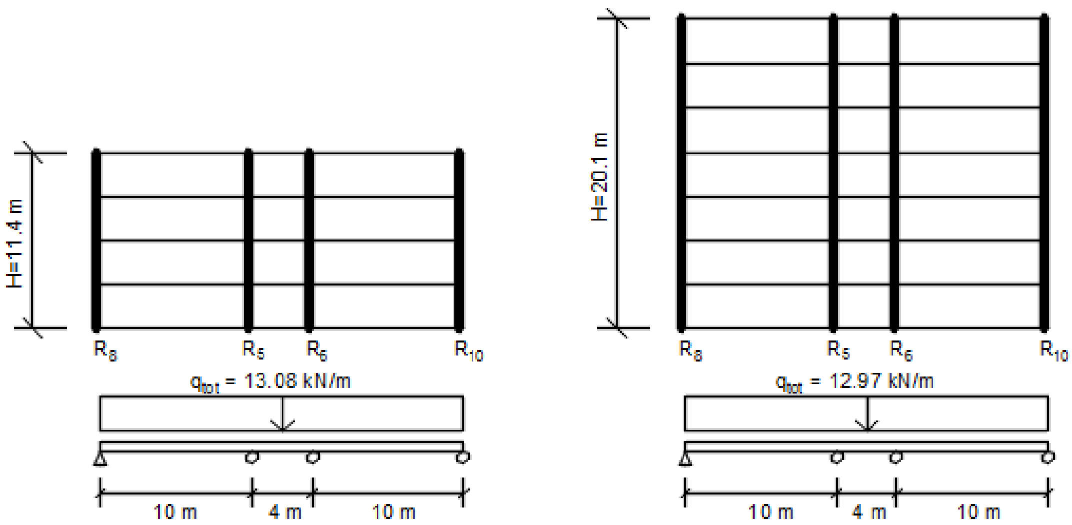

Figure 6.8 Section of building on the longer side with stabilizing walls

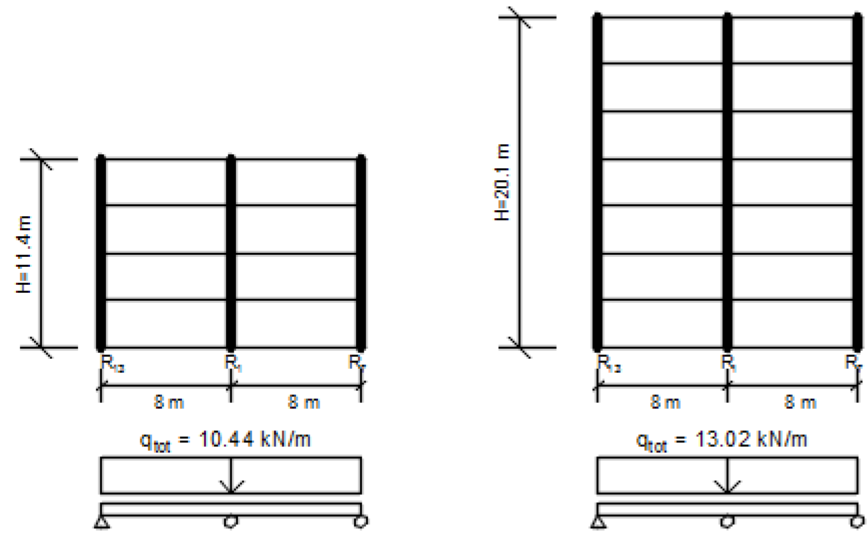

Figure 6.9 Section of building on the shorter side with stabilizing walls

The affected area the wind load was acting on the buildings with four respectively seven floors are shown in Figure 6.10. When the wind was coming on the long façade calculations for the tensile forces on the shear walls were only made for wall 5 (a and b) and wall 8 (c and d) due to symmetry. When the wind was coming on the shorter façade the affected area is shown for wall 1 (e and f) and for wall 13 (g and h).

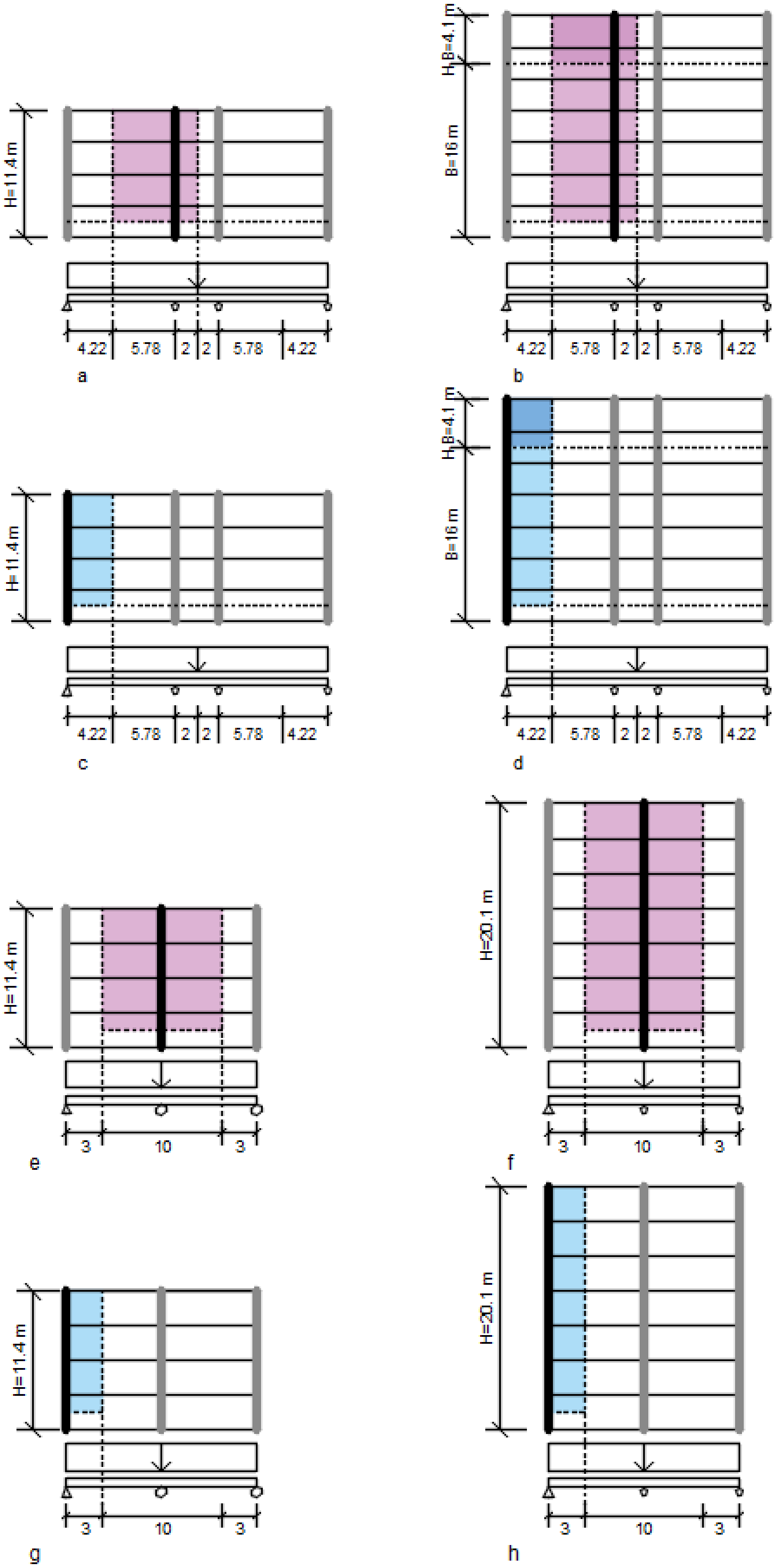

Figure 6.10 Affected area by wind load for each shear wall.

To find the tensile and compression forces of each shear wall, moment equations were used. See Appendix C and D. When studying the shear walls, the tension forces were the most difficult and expensive to anchor compared to the compression force acting on the other end of the wall. Wall 5 had an effective length of 7.78 m, see Figure 6.10a and b. In Figure 6.11 the forces needed to anchor shear wall 5 (and 3, 4 and 6) are presented for reference building with four and seven floors when the wind was coming on the long façade. It can be seen that the tensile force that needs to be anchored was 96 kN for the four storey building and 311 kN for the seven storey, almost 3.3 times as big for a seven storey building than a four storey building.

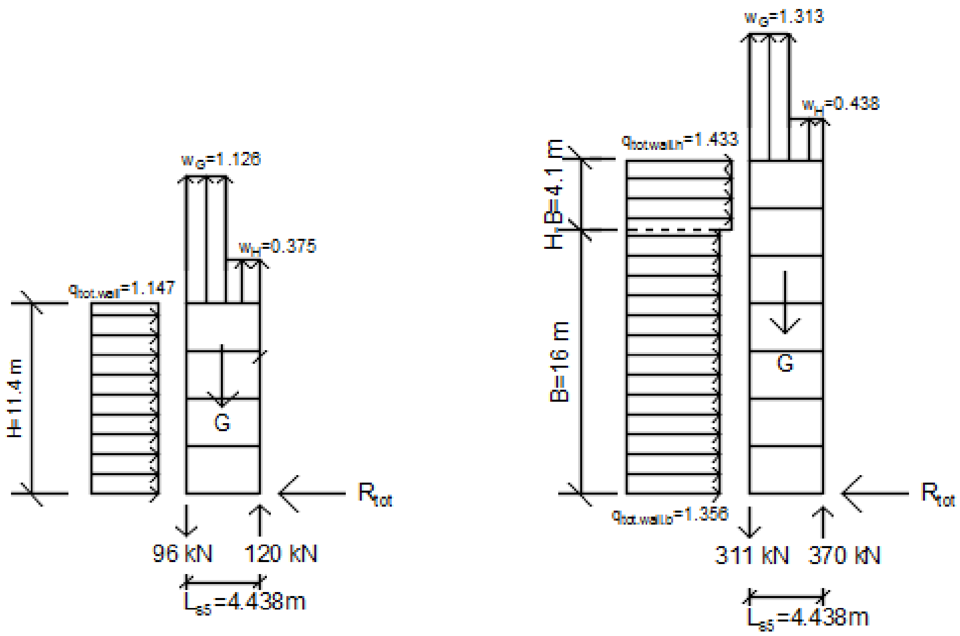

Figure 6.11 Elevations of shear wall 5.

When the wind was coming on the short facade the tensile forces that needed to be anchored on wall 1 with an effective length of 10 m were 39 kN for a four storey building and 133 kN for the building with seven floors. See Figure 6.12.

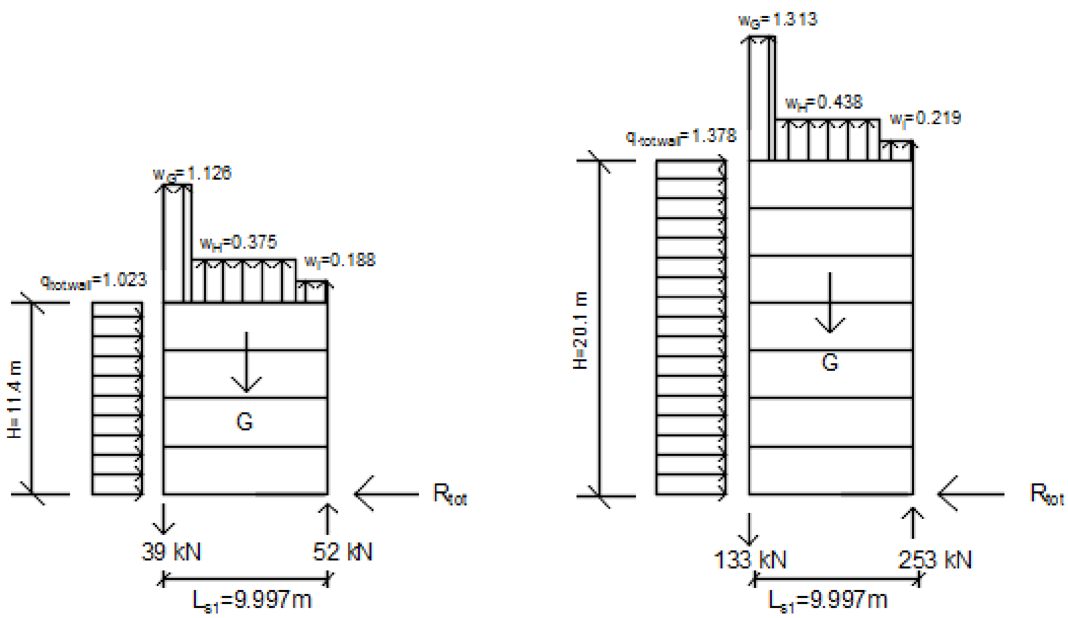

Figure 6.12 Elevations of shear wall 1.

When the wind was coming on the long façade the uplifting force for the shear walls in the corners for the four storey building, wall 8 was 106 kN and for the same wall but for seven stories 334 kN. In the wind direction of the shorter façade, the uplift force in wall 13 was 67 kN and 248 kN for the four respectively seven stories. The shear walls in the corners were made of OSB boards panels connected to both sides of the bottom rail. These helped to resist the wind loads and made the walls stiffer. However, the panels were subjected to uplifting forces as well and needed to be anchored but was however not performed in this thesis.

6.1.3 Anchorage

Solutions for anchorage system for the seven storey building are presented in the following sub-chapters. For the shear walls made of CLT, glued-in rods and slotted-in steel plates have been suggested. For the stud walls, hold-downs and threaded rods through the stud walls have been proposed.

6.1.3.1 Glued-in rods

To anchor the largest tensile force for walls made of CLT (wall 5), glued-in rods was an alternative to use due to its promising anchoring capacity. The uplifting force in wall 5 was 311 kN and checks were made concerning bonded line failure, pull-out strength and yielding of the steel. Due to the pull-out strength 7 rods with a diameter of 24 mm and an anchor length of 350 mm were required for this wall. 8 rods were used due to symmetry of two rows. See Figure 6.13 and 6.14 for the placement of the rods. The recommended edge distance that was used was 2.5 times the diameter of the rod. Between the minimum spacing between the rods, recommendations varied between 4-5 times the diameters of the rod. In this case the distance of *4d* = 96 mm was used (where d is the diameter of the rod) along the wall. Due to the 250 mm thickness of the wall the spacing between the rods/rows was 130 mm.

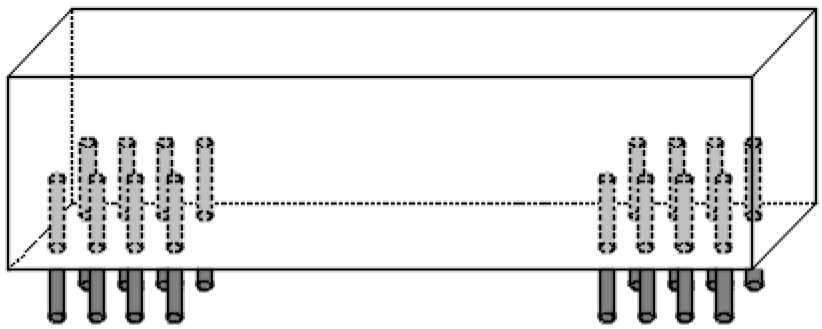

Figure 6.13 Wall with glued-in rods in 3D.

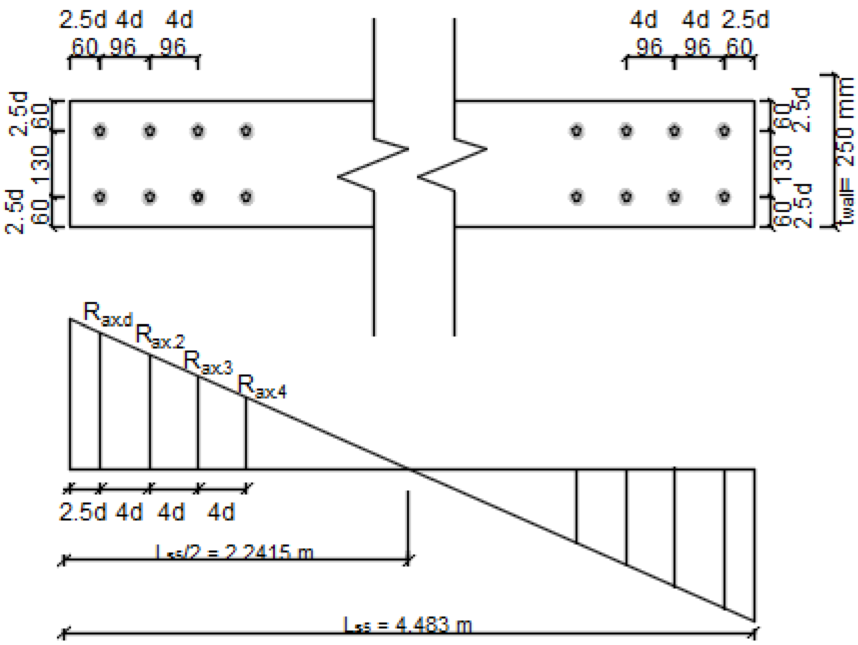

Figure 6.14 Wall 5 with glued-in rods seen from above and shear force diagram with linear force distribution.

In Chapter 5.1 it was stated that if the steel rods were designed as the failure mode i.e. weakest link, a uniform distribution of forces could be assumed. However, in this case, a linear distribution of the forces was assumed, see Figure 6.14.

The total resisting force of $R_{ax.i}$ for shear wall 5 turned out to be 425 kN when using glued-in rods which was larger than the uplifting force.

6.1.3.2 Slotted-in steel plates

Another way to anchor the shear walls of CLT was with slotted-in steel plates. The number of steel plates used as anchor varied, but the ones looked into in this project were slotted-in steel plates with: one, two or three plates. The resistance of the fasteners were designed according to the theory of plasticity. Different failure modes can occur where all of them can be found in Eurocode. The failure mode which was decisive in this design was the one when timber loses its solidness. In order to prevent this, 8 dowels were needed to be used when using one steel plate. For two and three steel plates 7 respectively 5 dowels were needed. Due to symmetry the number of dowels was increased to 8 and 6. To determine the distance between the dowels parallel and perpendicular to the grain and to the edges, recommendations from Eurocode were used where the outcome can be seen in Figure 6.15.

In Appendix F the calculations for anchoring the largest tensile force for wall 5 was found. The steel plate or plates had the dimensions 220 x 580 x 8 mm (b x h x t) with dowels of 20 mm in diameter. Figure 6.15 shows the number of dowels needed when slotted-in steel plates with one and two plates were used.

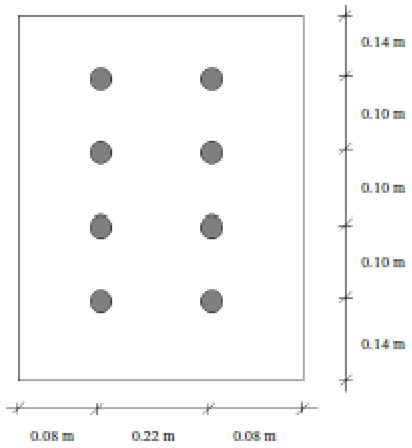

Figure 6.15 Slotted-in steel plate with 8 dowels.

6.1.3.3 Hold-downs

The shear walls in the corners were subjected to a critical uplifting force of 334 kN. Hold-down connection can provide high resistance against the tensile forces. In Appendix G, calculations were made using a hold-down that Rothoblaas is manufacturing. The steel quality was S355 and made of galvanic zinc with a thickness of 3 mm. The height of the chosen connector was 620 mm and the width was 80 mm. The washer plate at the bottom had a width of 80 mm and was anchored with a bolt with the diameter 20 mm (hole was 21 mm in diameter), See Figure 6.16. The hold-downs were fastened with anchor nails (ϕ 4.0 x 60) and special screws (ϕ 5.0 x 50).

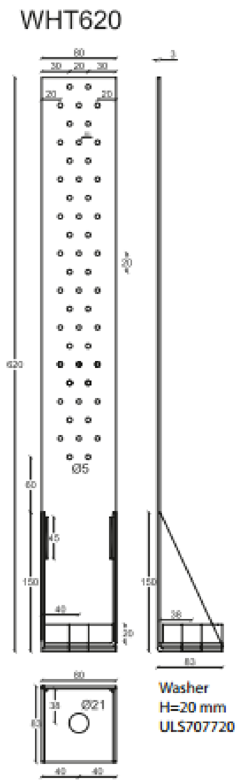

Figure 6.16 Hold-down from Rothoblaas.

To resist the uplifting force for shear wall 8 that was subjected to the highest wind load, 6 hold-downs were required. If the same type of connector was used for the most loaded CLT shear wall also 6 hold-downs were needed, see Appendix G.

6.1.3.4 Threaded rods through the stud wall with bearing plate

Another way to resist the uplifting force in the shear walls was with threaded rods that goes through the shear wall and fastened with a bearing plate above, see Figure 6.17. This solution is common for multi-storey buildings. Calculations that were made for the highest loaded shear wall showed that 3 rods were needed if threaded rods were used, see Appendix H.

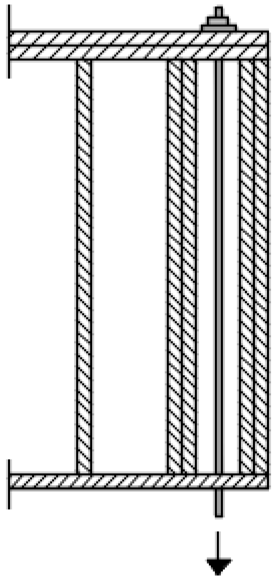

Figure 6.17 Section of anchorage of stud wall with a threaded rod.

7 Results

In Table 7-1 the tensile and compression forces acting on the reference buildings are shown. Figure 7-1 shows the placement of the walls.

Table 7-1 Summary of force on shear walls and anchorage

	Number of stories	
	4 stories	7 stories
Wall 5 (CLT), tension	96 kN	311 kN
Wall 5 (CLT), compression	120 kN	370 kN
Wall 8 (studs+panels), tension	106 kN	334 kN
Wall 1 (CLT), tension	39 kN	133 kN
Wall 1 (CLT), compression	52 kN	253 kN
Wall 13 (studs+panels), tension	67 kN	a248 kN

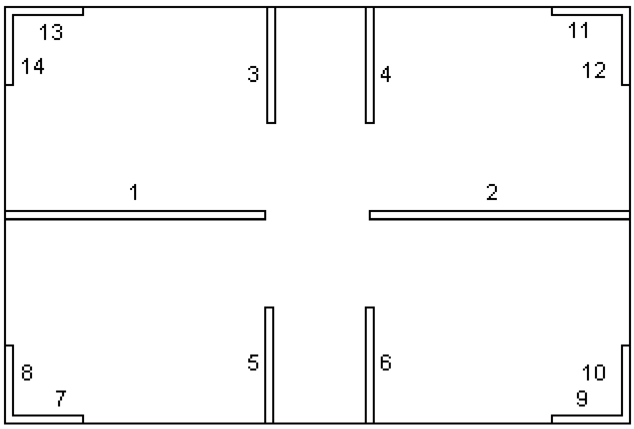

Figure 7.1 Placement of shear walls.

What can be seen in Table 7-1 is that the tension forces that needed to be anchored for the designed seven storey building were more than 3 times larger than for the four storey building. The anchoring system that could be used for the shear walls could be constructed with glued-in rods (wall 3-6) and slotted-in steel plates (wall 3-6), hold-downs (wall 7-14) or threaded rods going through the stud walls (wall 7-14).

7.1 Anchorage

7.1.1 Glued-in rods

Wall 3-6 needed to resist a load of more than 300 kN (when the wind was coming on the longer façade) and the number of rods needed was 7 due to the pull-out strength. If the failure mode had been at the bonded line of glue the number of rods required were 5. The chosen number of rods were 8 due to the possibility to have two full rows with 4 rods in each, see Figure 7.2.

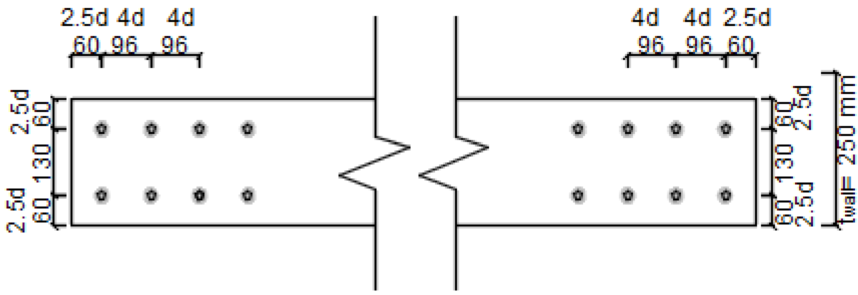

Figure 7.2 Stabilizing wall 5 with rods.

To avoid splitting and group tearing the distance between the rods and the rod to edge were taken according to prEn 1995:2001 which were 4*d* along the wall for the former and 2.5*d* for the latter. When looking at the thickness of the wall, the distance between the rows was 130 mm (larger than the recommended). See Figure 7.2.

7.1.2 Slotted-in steel plates

If slotted-in steel plates were to be used to withstand the uplifting force of shear wall 5, the connector would require 8 dowels with a steel plate that is 580 mm high, 220 mm in width and 8 mm thick. If the number of plates was increased to 2 or 3 plates, the required number of dowels needed were 7 respectively 5. Due to asymmetric number of dowels the dowels were adjusted to even numbers of 8 and 6.

7.1.3 Hold-down

When anchoring the stud walls a possible anchoring system to manage the uplifting forces was with hold-downs. Using connectors with dimensions 620 mm high and 80 mm wide, 6 of them were needed to anchor the shear wall (wall 8) that was subjected to the largest wind load.

7.1.4 Threaded rods through the stud wall with bearing plate

When anchoring the stud walls with threaded rods going through the stud walls with bearing plate, 3 rods with diameter of 24 mm was required to anchor the shear wall (wall 8) that was subjected to the largest wind load.

8 Discussion

The structural system being used as a reference building in this thesis is used by a market-leading company in Sweden specialized in timber construction. The company is known for their multi-storey timber houses which usually reach up to four floors.

The calculation method used in this project for the stability and tilting has been made according to Eurocode, an elastic model. As mentioned in previous chapters, using a plastic calculation method will allow for a reduction of the anchorage forces since one part of the wall is allowed to plasticize. The results of loads and uplifting forces can therefore vary depending on the method used for calculations. This can also explain the difference between the highest calculated tensile force, 100 kN, for the reference building with four stories in this project and the companie's highest tensile anchoring force of 80 kN.

One of the reasons to why higher buildings are not erected is because of the large anchorage forces that arise which demands alternative anchoring solutions. When comparing the results in Table 7-1, it becomes clear that there is a major difference in uplifting force between a four- and seven storey building. The uplifting forces that need to be anchored are much higher for the seven storey building than for the four storey building, approximately 3 times larger. These demands larger alternative anchorage which are often difficult and expensive to execute in today's situation.

In this project focus has been on three types of anchorage systems; glued-in rods, slotted-in steel plates and hold-downs (and threaded rods). Glued-in rods are as mentioned in the literature study not included in Eurocode but due to the ability of the anchoring system to resist large tensile forces it has become more popular. Previous projects and tests concerning glued-in rods have given various proposals on how to determine the pull-out capacity of a single rod. One issue with these tests is that in practice more than one rod is used. When analyzing a single rod, less parameters and failure modes are needed to be considered compared with using multiple rods. Of this reason, tests with multiple rods in a connection are usually not performed due to the complexity. The formulas to calculate the pull-out strengths that has been derived from single rod tests gives an uncertainty to use for multiple rods.

The shear capacity for slotted-in steel plates was determined to 40 kN, 47 kN and 71 kN depending on how many steel plates were used (one, two or three plates in this case). The capacity calculations for slotted-in steel plates are dependent on the thickness of the timber, steel plates and dowels, but also the spacing between the steel plates that are affecting the capacity. In this project all the possible outcomes regarding thicknesses and spacing have not been taken into account making it possible that the calculated capacity most likely could be higher.

The calculations performed for the studied anchorage systems show that they can be possible to execute. What is not treated in this project but is considered as an important factor, especially for construction companies, is the cost to perform these anchorages. Another valued factor is the method to execute the anchorages. For example, using glued-in rods requires manufacturing in factory since gluing on-site is forbidden (in Sweden). This will however certify the quality of the connection since it comes with the advantages of prefabrication. Hold-downs on the other hand are relatively easy to install but require large dimensions of the connections that are as well not as aesthetically appealing as glued-in rods or slotted-in steel plates.

Moreover, when designing buildings it is important to look at the entire structure and not todo sub-optimizations. For example what might seem to be the most optimal anchoring system for a structure might require a thicker concrete foundation than necessary or a complex reinforcement plan in the concrete. This has not been considered in the project.

8.1 Future studies

Complementary calculations for the shear walls with a plasticizing calculation model (for example Källsner) can be made to find out how much the anchorage capacity can be reduced. Additional options of anchoring systems should be investigated to find an optimal solution. Further studies should also be made concerning cost and execution of connections. For the glued-in rods, more tests and documentation are needed to analyse the behaviour of multiple rods in a connection.

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APPENDIX A: Reference building - 4 stories - Stability

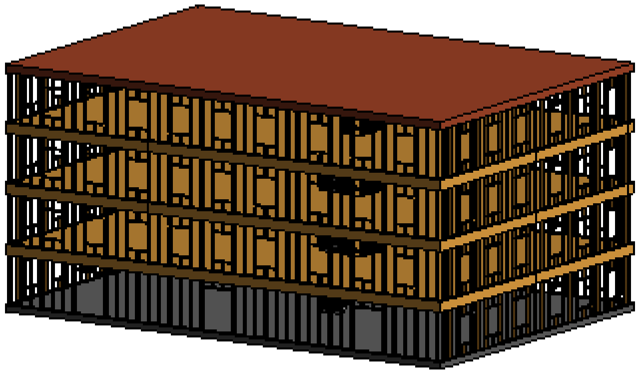


Figure A1: 3D model of reference building with 4 stories.

$B := 16m$	Width of building
$L := 24m$	Length of building
$h_{storey} := 2.4m$	Inner height of storey
$t_{floor} := 0.5m$	Thickness of floor slab
$t_{ext.wall} := 0.3m$	Thickness of exterior wall
$t_{foundation} := 0.3m$	Thickness of concrete foundation
$t_{shaft} := 0.25m$	Thickness of shaft / stabilizing walls
$t_{roof} := 0.3m$	Thickness of roof
$h_{tot} := 4h_{storey} + 3 \cdot t_{floor} + t_{roof} = 11.4\,m$	Total height of builing
$h_{floor} := h_{storey} + t_{floor} = 2.9\,m$	Height of storey and floor slab
$h_{floor7} := h_{storey} + t_{roof} = 2.7\,m$	Height of 7th storey with roof
$A_{shaft.stabilizing} := h_{storey} \cdot (2 \cdot 9.997m + 4 \cdot 4.483m) = 91.022\,m^2$	Area for shaft and stabilizing walls on each floor.
$\rho_{con} := 2400 \frac{kg}{m^3}$	Density of concrete
$\rho_{CLT.shaft} := 490 \frac{kg}{m^3}$	Density of solid timber

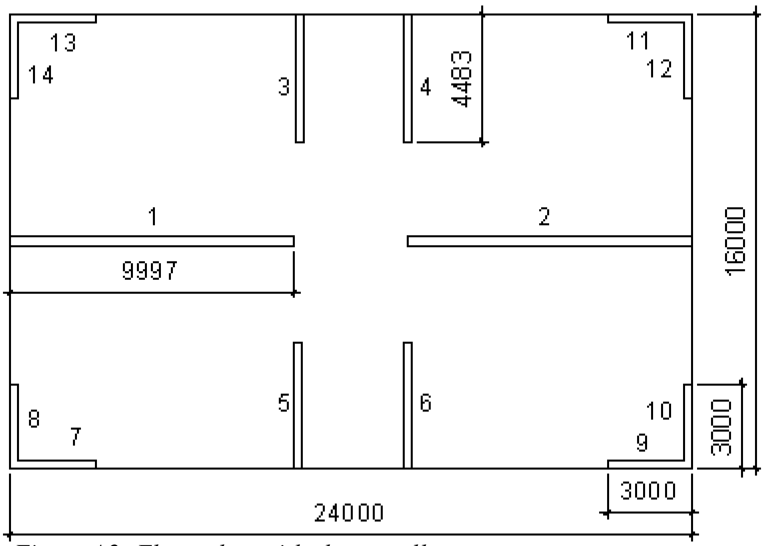


Figure A2: Floor plan with shear walls.

Permanent Characteristic Loads

$G_{k.roof} := 0.5\frac{kN}{m^2}$ $G_{k.floor} := 1.0\frac{kN}{m^2}$ $G_{k.foundation} := \rho_{con} \cdot g \cdot t_{foundation} = 7.061 \cdot \frac{kN}{m^2}$

$G_{k.ext.walls} := 0.8\frac{kN}{m^2}$ $G_{k.part.wall} := 2.1\frac{kN}{m}$ $G_{k.shaft} := \rho_{CLT.shaft} \cdot g \cdot t_{shaft} = 1.201 \cdot \frac{kN}{m^2}$

Wind coming on the long facade

Unintended Inclination

$\alpha_0 := 0.003$ Systematic part of inclination angle

$\alpha_d := 0.012$ Random part of inclination angle

$n := 2$ Number of load bearing walls

$$\alpha_{md} := \alpha_0 + \frac{\alpha_d}{\sqrt{n}} = 0.011$$

$\psi_0 := 0.7$

$\psi_1 := 0.5$

$\psi_2 := 0.3$

Self-weight as favorable

$$V_{d4} := 0.9 \cdot \left(G_{k.roof} \cdot B \cdot L\right) = 172.8 \cdot kN$$

$$V_{d3} := 0.9 \cdot \left[\begin{array}{l} G_{k.shaft} \cdot A_{shaft.stabilizing} + G_{k.ext.walls} \cdot h_{storey} \cdot (2L + 2 \cdot B) \; ... \\ + G_{k.floor} \cdot B \cdot L + G_{k.part.wall} \cdot 56m \end{array}\right] = 688.092 \cdot kN$$

$$V_{d2} := 0.9 \cdot \left[\begin{array}{l} G_{k.shaft} \cdot A_{shaft.stabilizing} + G_{k.ext.walls} \cdot h_{storey} \cdot (2L + 2 \cdot B) \; ... \\ + G_{k.floor} \cdot B \cdot L + G_{k.part.wall} \cdot 56m \end{array}\right] = 688.092 \cdot kN$$

$$V_{d1} := 0.9 \cdot \left[\begin{array}{l} G_{k.shaft} \cdot A_{shaft.stabilizing} + G_{k.ext.walls} \cdot h_{storey} \cdot (2L + 2 \cdot B) \; ... \\ + G_{k.floor} \cdot B \cdot L + G_{k.part.wall} \cdot 56m \end{array}\right] = 688.092 \cdot kN$$

Horizontal forces (self-weight as favorable)

$$H_{d4} := V_{d4} \cdot \alpha_{md} = 1.985 \cdot kN$$

$$H_{d3} := V_{d3} \cdot \alpha_{md} = 7.903 \cdot kN$$

$$H_{d2} := V_{d2} \cdot \alpha_{md} = 7.903 \cdot kN$$

$$H_{d1} := V_{d1} \cdot \alpha_{md} = 7.903 \cdot kN$$

Horisontal loads

$c_e := 2.4$ (Terrain category II)

$\rho_{air} := 1.25 \frac{kg}{m^3}$

$\upsilon_b := 25 \frac{m}{s}$

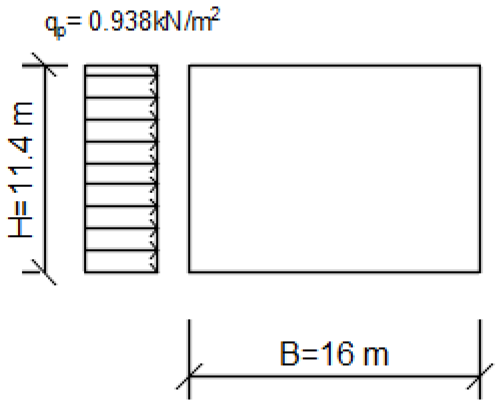
$q_b := \frac{1}{2} \cdot \rho_{air} \cdot \upsilon_b^{\ 2} = 0.391 \cdot \frac{kN}{m^2}$

$q_p := c_e \cdot q_b = 0.938 \cdot \frac{kN}{m^2}$

Wind pressure on walls

$e := \min(L, 2 \cdot h_{tot}) = 22.8\,m$

$d := B = 16\,m$



$h \le b$

Figur A3: Elevation of the short facade when the wind is coming on the long side.

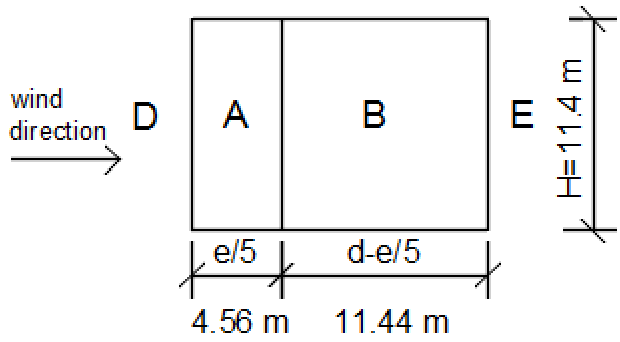


Figure A4: Elevation - short side

e ≥ d

$$a := \frac{h_{tot}}{d} = 0.713$$

Used to get C_{pe}-values (Eurocode)

Table A1: Wind pressure on walls.

Exterior Wall $C_{pe.10}$		q_p (kN/m^2)	
		0.938	
Zone	$C_{pe.10}$	w_e (h)	
A	-1.2	-1.126	suction
B	-0.8	-0.750	suction
C	-0.5	-0.469	suction
D	0.8	0.750	pressure
E	-0.42	-0.397	suction

$$w_D := 0.750 \frac{kN}{m^2} \qquad w_E := -0.397 \frac{kN}{m^2}$$

$$q_{tot.wall} := w_D - w_E = 1.147 \cdot \frac{kN}{m^2}$$

Wall D + E

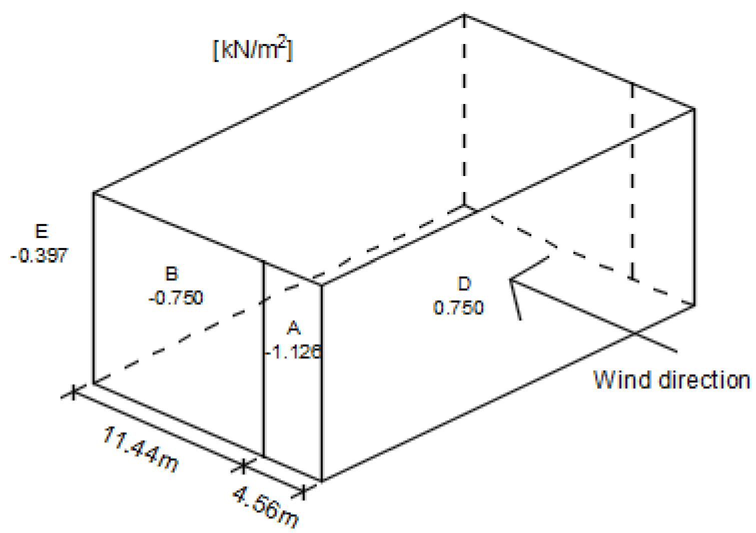


Figure A5: Wind pressure (kN/m2) on walls for the building.

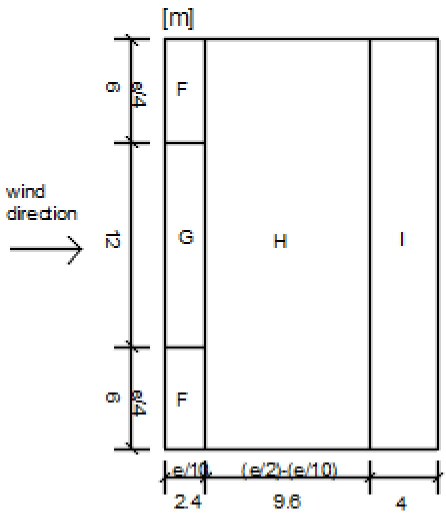


Figure A6: Roof plan with affected area by wind load.

Table A2: Wind pressure on roof.

Flat Roof $C_{pe.10}$		q_p (kN/m^2)	
		0.938	
Zone	$C_{pe.10}$	w_e	
F	-1.0	-0.938	suction
G	-1.2	-1.126	suction
H	-0.4	-0.375	suction
I	-0.2	-0.188	suction

$$w_G := -1.126 \frac{kN}{m^2}$$

$$w_H := -0.375 \frac{kN}{m^2}$$

$$w_I := -0.188 \frac{kN}{m^2}$$

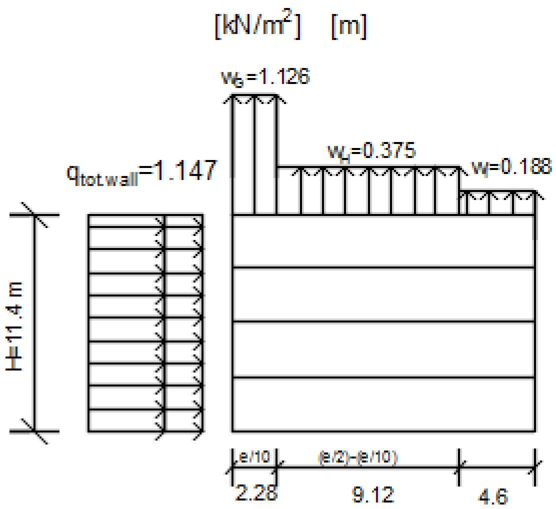


Figure A7: Elevation of short side with wind pressure coming on the long facade.

Tilting

$$e_{core} := \frac{B}{6} = 2.667\,m$$

Self-weight as favorable

Lever arm

$$x_G := \frac{B}{2} - \frac{e}{2\cdot 10} + e_{core} = 9.527\,m$$

$$x_H := \frac{B}{2} - \frac{e}{10} - 0.5\cdot\left(\frac{e}{2} - \frac{e}{10}\right) + e_{core} = 3.827\,m$$

$$x_I := \frac{B}{2} - e_{core} - 0.5\cdot\left(B - \frac{e}{2}\right) = 3.033\,m$$

$$\gamma_G := 0.9$$

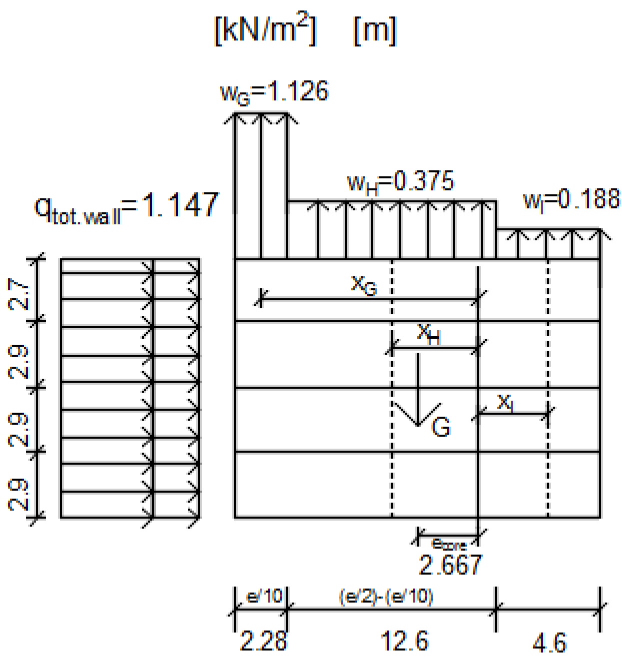


Figure A8: Lever arms between wind loads on roof and e_{core}.

$$M_{horisontal} := \gamma_G\cdot q_{tot.wall}\cdot L\cdot\left(h_{floor}\cdot 2.9m + h_{floor}\cdot 5.8m + h_{floor}\cdot 8.7m + h_{floor}\cdot 11.4m\right) \ldots = 2.229\times 10^3\cdot kN\cdot m$$
$$+ H_{d4}\cdot 11.4m + H_{d3}\cdot 8.7m + H_{d2}\cdot 5.8m + H_{d1}\cdot 2.9m$$

$$M_{vertical} := \gamma_G\cdot L\cdot\left[w_G\cdot\frac{e}{10}\cdot x_G + w_H\cdot\left(\frac{e}{2} - \frac{e}{10}\right)\cdot x_H - w_I\cdot\left(B - \frac{e}{2}\right)\cdot x_I\right] = -754.306\cdot kN\cdot m$$

$$M_{tilting} := M_{horisontal} + M_{vertical} = 1.475\times 10^3\cdot kN\cdot m$$

Resisting moment including self-weight of foundation

$$V_{tot} := V_{d4} + V_{d3} + V_{d2} + V_{d1} + G_{k.foundation}\cdot B\cdot L = 4.948\times 10^3\cdot kN$$

$$M_{resisting} := V_{tot}\cdot e_{core} = 1.32\times 10^4\cdot kN\cdot m$$

$$\frac{M_{tilting}}{M_{resisting}} = 0.112 \qquad <1 \qquad \text{OK}$$

<u>Wind coming on the long facade</u>

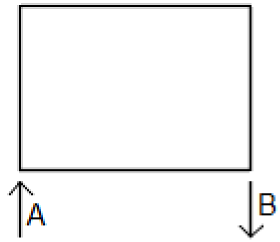


Figure A9: Moment calculations - Point A and B.

Moment around point B

$$R_A := \frac{-\left[q_{tot.wall}\cdot L\cdot h_{tot}\cdot\frac{h_{tot}}{2}\ \ldots \\ +w_G\cdot\frac{e}{10}\cdot L\cdot\left[\frac{e}{2\cdot 10}+\left(\frac{e}{2}-\frac{e}{10}\right)+\left(B-\frac{e}{2}\right)\right]\ \ldots \\ +\left[w_H\cdot\left(\frac{e}{2}-\frac{e}{10}\right)\cdot L\cdot\left[\left(B-\frac{e}{2}\right)+\frac{\frac{e}{2}-\frac{e}{10}}{2}\right]+w_I\cdot\left(B-\frac{e}{2}\right)\cdot L\cdot\frac{\left(B-\frac{e}{2}\right)}{2}-V_{tot}\cdot\frac{B}{2}\right]\right]}{B} = 2.470\times 10^3\cdot kN$$

$$R_{B1} := -\left[V_{tot}-w_G\cdot\frac{e}{10}\cdot L-w_H\cdot\left(\frac{e}{2}-\frac{e}{10}\right)\cdot L-w_I\cdot\left(B-\frac{e}{2}\right)\cdot L-R_A\right] = -2.643\times 10^3\cdot kN$$

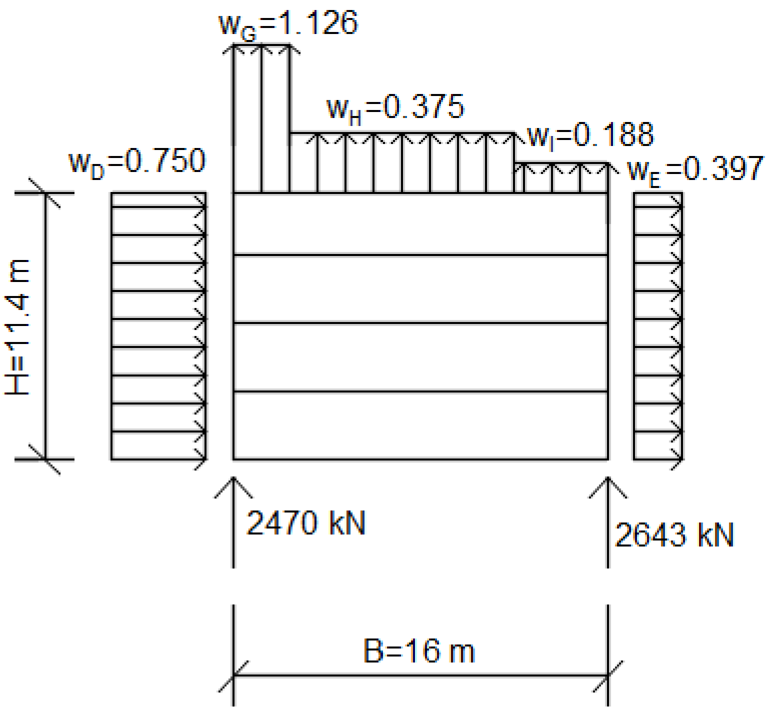


Figure A10: Section of the short facade of the building with wind load coming on long facade.

Wind coming on the short facade

Unintended Inclination

$\alpha_0 := 0.003$ Systematic part of inclination angle

$\alpha_d := 0.012$ Random part of inclination angle

$n := 4$ Number of load bearing walls

$$\alpha_{md} := \alpha_0 + \frac{\alpha_d}{\sqrt{n}} = 9 \times 10^{-3}$$

Horizontal forces (self weight as favorable)

$H_{d4} := V_{d4} \cdot \alpha_{md} = 1.555 \cdot kN$

$H_{d3} := V_{d3} \cdot \alpha_{md} = 6.193 \cdot kN$

$H_{d2} := V_{d2} \cdot \alpha_{md} = 6.193 \cdot kN$

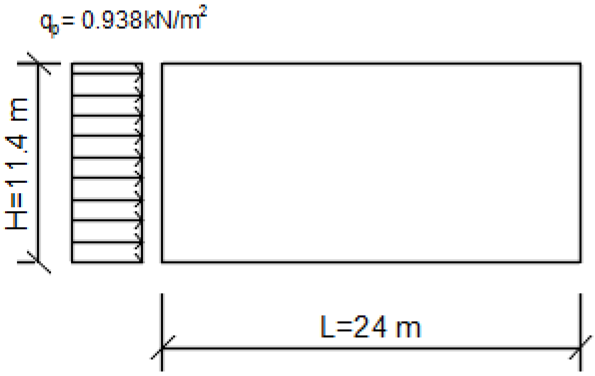
$H_{d1} := V_{d1} \cdot \alpha_{md} = 6.193 \cdot kN$

Horisontal loads
Wind pressure on walls

$q_p := c_e \cdot q_b = 0.938 \cdot \frac{kN}{m^2}$

$e := \min(B, 2 \cdot h_{tot}) = 16\,m$

$d := L = 24\,m$



$h < d$

Figure A11: Elevation of the long facade when the wind is coming on the short side.

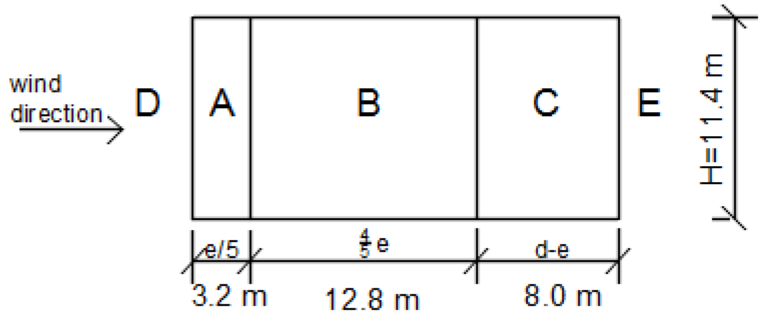


Figure A12: Elevation - long side.

e < d

$$a := \frac{h_{tot}}{d} = 0.475$$

Table A3: Wind pressure on walls.

Exterior Wall $C_{pe.10}$		q_p (kN/m^2)	
		0.938	
Zone	$C_{pe.10}$	w_e (h)	
A	-1.2	-1.126	suction
B	-0.8	-0.750	suction
C	-0.5	-0.469	suction
D	0.7	0.685	pressure
E	-0.36	-0.338	suction

$$w_D := 0.685 \frac{kN}{m^2}$$

$$w_E := -0.338 \frac{kN}{m^2}$$

$$q_{tot.wall} := w_D - w_E = 1.023 \cdot \frac{kN}{m^2}$$ Wall D + E

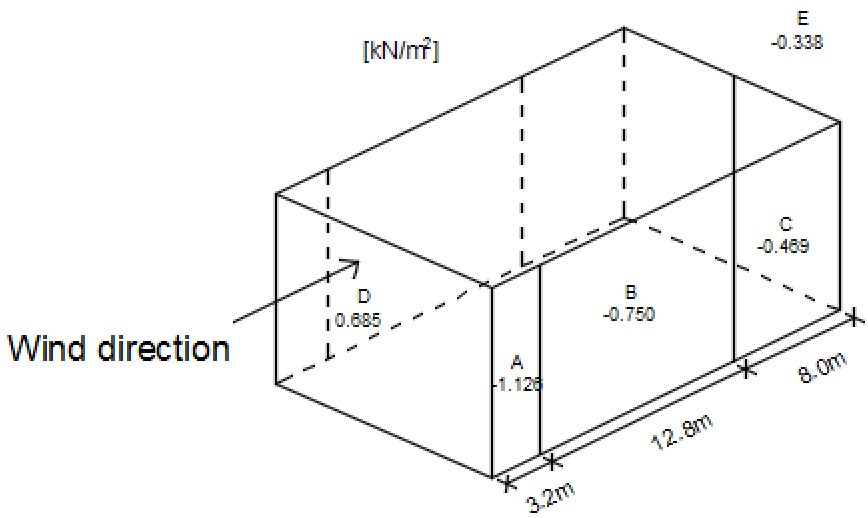


Figure A13: Wind pressure (kN/m²) on walls for the building.

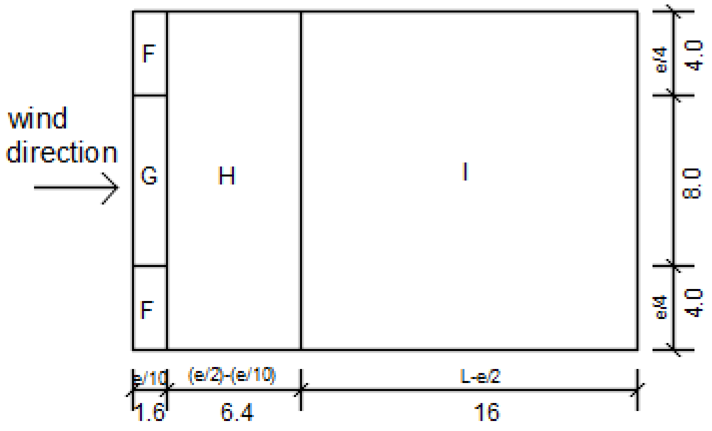


Figure A14: Elevation - long side.

Table A4: Wind pressure on roof.

Flat Roof $C_{pe.10}$		q_p (kN/m²)	
		0.938	
Zone	$C_{pe.10}$	w_e	
F	-1.0	-0.938	suction
G	-1.2	-1.126	suction
H	-0.4	-0.375	suction
I	-0.2	-0.188	suction

$$w_G := -1.126 \frac{kN}{m^2}$$

$$w_H := -0.375 \frac{kN}{m^2}$$

$$w_I := -0.188 \frac{kN}{m^2}$$

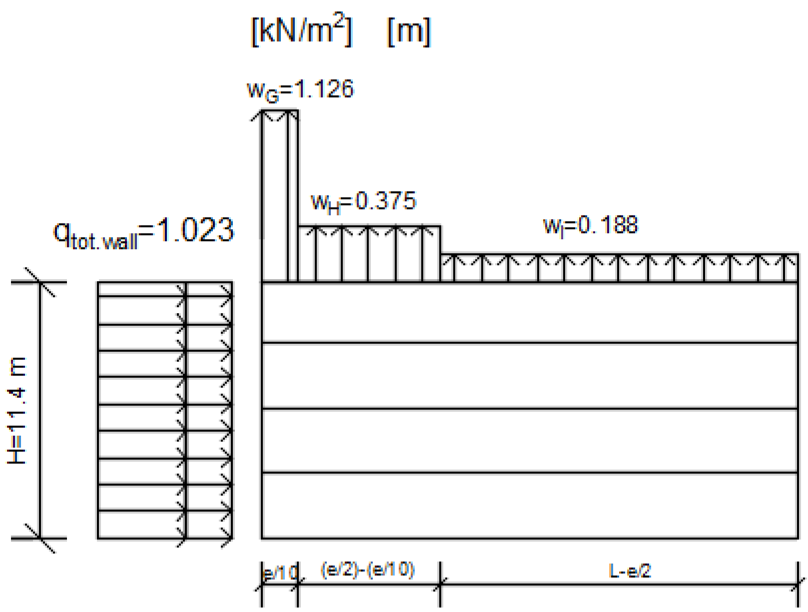


Figure A15: Section of the long facade of the building with wind load coming on short facade.

Tilting

$e_{core} := \frac{L}{6} = 4\,m$

Self-weight as favorable

Lever arm

$x_G := \frac{L}{2} - \frac{e}{2 \cdot 10} + e_{core} = 15.2\,m$

$x_H := \frac{L}{2} - \frac{e}{10} - 0.5 \cdot \left(\frac{e}{2} - \frac{e}{10}\right) + e_{core} = 11.2\,m$

$x_I := \frac{L}{2} - e_{core} - 0.5 \cdot \left(L - \frac{e}{2}\right) = 0\,m$

$\gamma_G := 0.9$

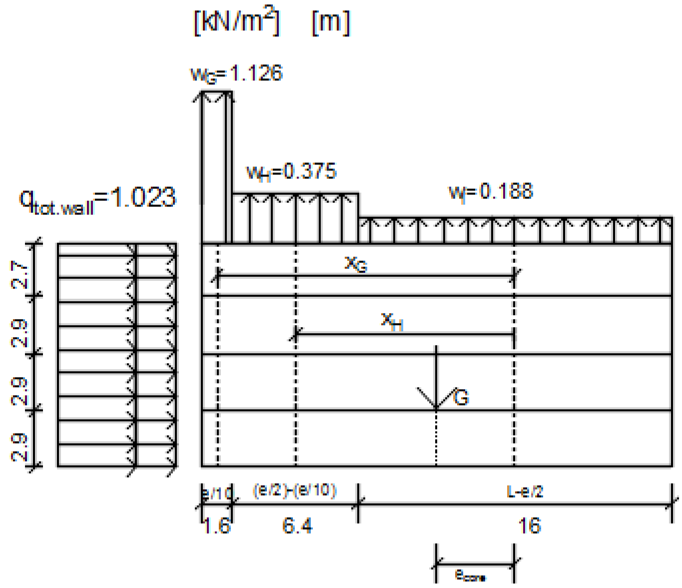


Figure A16: Lever arm between wind loads and e_{core}.

$$M_{horisontal} := \gamma_G \cdot q_{tot.wall} \cdot B \cdot \left(h_{floor} \cdot 2.9m + h_{floor} \cdot 5.8m + h_{floor} \cdot 8.7m + h_{floor} \cdot 11.4m\right) \ldots + H_{d4} \cdot 11.4m + H_{d3} \cdot 8.7m + H_{d2} \cdot 5.8m + H_{d1} \cdot 2.9m = 1.356 \times 10^3 \cdot kN \cdot m$$

$$M_{vertical} := \gamma_G \cdot B \cdot \left[w_G \cdot \frac{e}{10} \cdot x_G + w_H \cdot \left(\frac{e}{2} - \frac{e}{10}\right) \cdot x_H - w_I \cdot \left(L - \frac{e}{2}\right) \cdot x_I\right] = -781.406 \cdot kN \cdot m$$

$$M_{tilting} := M_{horisontal} + M_{vertical} = 574.428 \cdot kN \cdot m$$

Resisting moment including self-weight of foundation

$$V_{tot} := V_{d4} + V_{d3} + V_{d2} + V_{d1} + G_{k.foundation} \cdot B \cdot L = 4.948 \times 10^3 \cdot kN$$

$$M_{resisting1} := V_{tot} \cdot e_{core} = 1.979 \times 10^4 \cdot kN \cdot m$$

$\frac{M_{tilting}}{M_{resisting1}} = 0.029$ <1 OK

Moment around point B

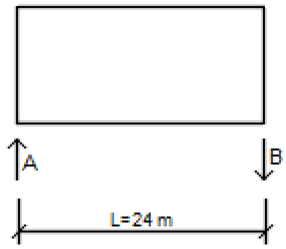


Figure A17: Moment calculations - Piont A and B.

$$R_{A1} := \frac{-\left[q_{tot.wall}\cdot h_{tot}\cdot B\cdot\frac{h_{tot}}{2} + w_G\cdot\frac{e}{10}\cdot B\cdot\left(L-\frac{e}{2\cdot 10}\right) \ldots + w_H\cdot\left(\frac{e}{2}-\frac{e}{10}\right)\cdot B\cdot\left(L-\frac{e}{10}-\frac{\frac{e}{2}-\frac{e}{10}}{2}\right) + w_I\cdot\left(L-\frac{e}{2}\right)\cdot B\cdot\left(\frac{L-\frac{e}{2}}{2}\right) - V_{tot}\cdot\frac{L}{2}\right]}{L} = 2.505\times 10^3\cdot kN$$

$$R_{B1} := -\left[V_{tot} - w_G\cdot\frac{e}{10}\cdot B - w_H\cdot\left(\frac{e}{2}-\frac{e}{10}\right)\cdot B - w_I\cdot\left(L-\frac{e}{2}\right)\cdot B - R_{A1}\right] = -2.559\times 10^3\cdot kN$$

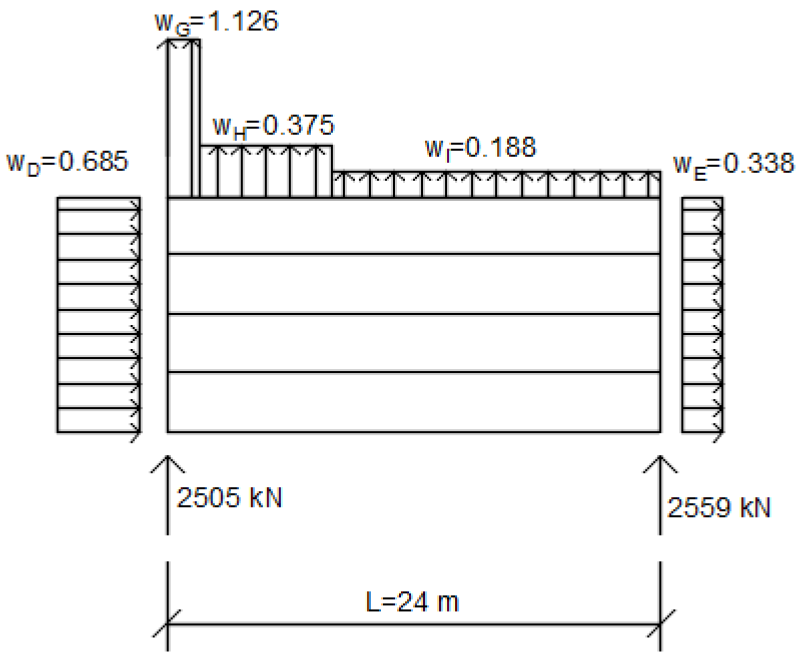


Figure A18: Section of the long facade of the building with wind load coming on short facade.

APPENDIX B: Reference building - 7 stories - Stability

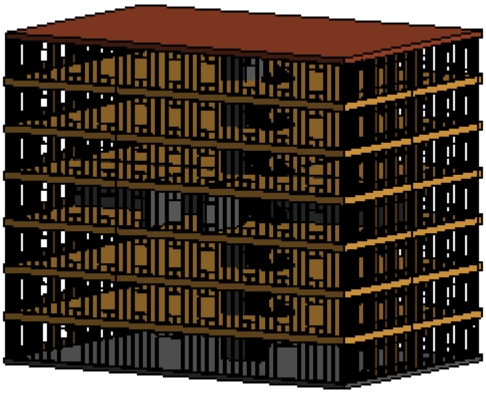
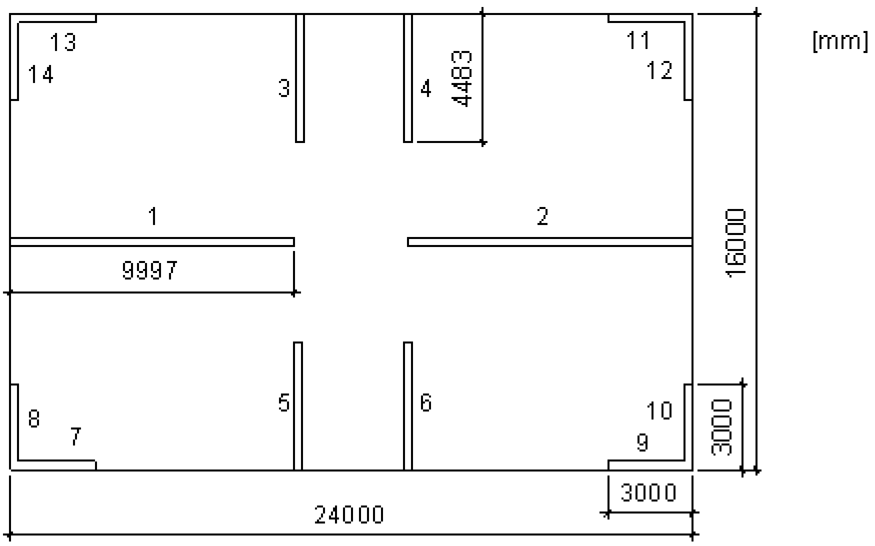


Figure B1: 3D model of reference building with 7 stories.

$B := 16m$	Width of building
$L := 24m$	Length of building
$h_{storey} := 2.4m$	Inner height of storey
$t_{floor} := 0.5m$	Thickness of floor slab
$t_{ext.wall} := 0.3m$	Thickness of exterior wall
$t_{foundation} := 0.3m$	Thickness of concrete foundation
$t_{shaft} := 0.25m$	Thickness of shaft / stabilizing walls
$t_{roof} := 0.3m$	Thickness of roof
$h_{tot} := 7h_{storey} + 6 \cdot t_{floor} + t_{roof} = 20.1\,m$	Total height of builing
$h_{floor} := h_{storey} + t_{floor} = 2.9\,m$	Height of storey and floor slab
$h_{floor7} := h_{storey} + t_{roof} = 2.7\,m$	Height of 7th storey with roof
$A_{shaft.stabilizing} := h_{storey} \cdot (2 \cdot 9.997m + 4 \cdot 4.483m) = 91.022\,m^2$	Area for shaft and stabilizing walls on each floor.
$\rho_{con} := 2400 \dfrac{kg}{m^3}$	Density of concrete
$\rho_{CLT.shaft} := 490 \dfrac{kg}{m^3}$	Density of solid timber



B2: Floor plan with shear walls.

Permanent Characteristic Loads

$G_{k.roof} := 0.5\frac{kN}{m^2}$ $G_{k.floor} := 1.0\frac{kN}{m^2}$ $G_{k.foundation} := \rho_{con} \cdot g \cdot t_{foundation} = 7.061 \cdot \frac{kN}{m^2}$

$G_{k.ext.walls} := 0.8\frac{kN}{m^2}$ $G_{k.part.wall} := 2.1\frac{kN}{m}$ $G_{k.shaft} := \rho_{CLT.shaft} \cdot g \cdot t_{shaft} = 1.201 \cdot \frac{kN}{m^2}$

Wind coming on the long facade

Unintended Inclination

$\alpha_0 := 0.003$ Systematic part of inclination angle

$\alpha_d := 0.012$ Random part of inclination angle

$n := 2$ Number of load bearing walls

$\psi_0 := 0.7$

$\psi_1 := 0.5$

$\psi_2 := 0.3$

$$\alpha_{md} := \alpha_0 + \frac{\alpha_d}{\sqrt{n}} = 0.011$$

Self-weight as favorable

$$V_{d7} := 0.9 \cdot \left(G_{k.roof} \cdot B \cdot L\right) = 172.8 \cdot kN$$

$$V_{d6} := 0.9 \cdot \begin{bmatrix} G_{k.shaft} \cdot A_{shaft.stabilizing} + G_{k.ext.walls} \cdot h_{storey} \cdot (2L + 2 \cdot B) \ ... \\ + G_{k.floor} \cdot B \cdot L + G_{k.part.wall} \cdot 56m \end{bmatrix} = 688.092 \cdot kN$$

$$V_{d5} := 0.9 \cdot \begin{bmatrix} G_{k.shaft} \cdot A_{shaft.stabilizing} + G_{k.ext.walls} \cdot h_{storey} \cdot (2L + 2 \cdot B) \ ... \\ + G_{k.floor} \cdot B \cdot L + G_{k.part.wall} \cdot 56m \end{bmatrix} = 688.092 \cdot kN$$

$$V_{d4} := 0.9 \cdot \begin{bmatrix} G_{k.shaft} \cdot A_{shaft.stabilizing} + G_{k.ext.walls} \cdot h_{storey} \cdot (2L + 2 \cdot B) \ ... \\ + G_{k.floor} \cdot B \cdot L + G_{k.part.wall} \cdot 56m \end{bmatrix} = 688.092 \cdot kN$$

$$V_{d3} := 0.9 \cdot \begin{bmatrix} G_{k.shaft} \cdot A_{shaft.stabilizing} + G_{k.ext.walls} \cdot h_{storey} \cdot (2L + 2 \cdot B) \ ... \\ + G_{k.floor} \cdot B \cdot L + G_{k.part.wall} \cdot 56m \end{bmatrix} = 688.092 \cdot kN$$

$$V_{d2} := 0.9 \cdot \begin{bmatrix} G_{k.shaft} \cdot A_{shaft.stabilizing} + G_{k.ext.walls} \cdot h_{storey} \cdot (2L + 2 \cdot B) \ ... \\ + G_{k.floor} \cdot B \cdot L + G_{k.part.wall} \cdot 56m \end{bmatrix} = 688.092 \cdot kN$$

$$V_{d1} := 0.9 \cdot \begin{bmatrix} G_{k.shaft} \cdot A_{shaft.stabilizing} + G_{k.ext.walls} \cdot h_{storey} \cdot (2L + 2 \cdot B) \ ... \\ + G_{k.floor} \cdot B \cdot L + G_{k.part.wall} \cdot 56m \end{bmatrix} = 688.092 \cdot kN$$

Horizontal forces (self weight as unfavorable)

$$H_{d7} := V_{d7} \cdot \alpha_{md} = 1.985 \cdot kN$$

$$H_{d6} := V_{d6} \cdot \alpha_{md} = 7.903 \cdot kN$$

$$H_{d5} := V_{d5} \cdot \alpha_{md} = 7.903 \cdot kN$$

$$H_{d4} := V_{d4} \cdot \alpha_{md} = 7.903 \cdot kN$$

$$H_{d3} := V_{d3} \cdot \alpha_{md} = 7.903 \cdot kN$$

$$H_{d2} := V_{d2} \cdot \alpha_{md} = 7.903 \cdot kN$$

$$H_{d1} := V_{d1} \cdot \alpha_{md} = 7.903 \cdot kN$$

Horisontal loads

$c_{e.b} := 2.65$ (Terrain category II)

$c_{e.h} := 2.8$

$\rho_{air} := 1.25 \frac{kg}{m^3}$

$\upsilon_b := 25 \frac{m}{s}$

$q_b := \frac{1}{2} \cdot \rho_{air} \cdot \upsilon_b^{\;2} = 0.391 \cdot \frac{kN}{m^2}$

$q_{p.b} := c_{e.b} \cdot q_b = 1.035 \cdot \frac{kN}{m^2}$ Peak velocity pressure on top part of wall

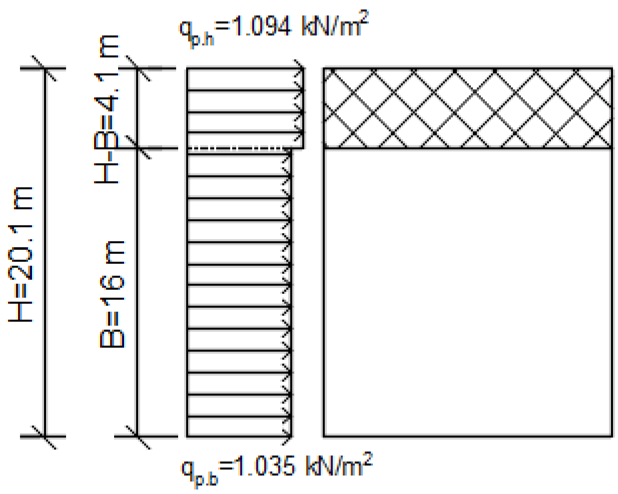
$q_{p.h} := c_{e.h} \cdot q_b = 1.094 \cdot \frac{kN}{m^2}$ Peak velocity pressure on bottom part of wall

Wind pressure on walls

$e := \min(L, 2 \cdot h_{tot}) = 24\,m$

$d := B = 16\,m$

$e \geq d$



b < h < 2b

Figure B3: Elevation of the short facade when the wind is coming on the long side.

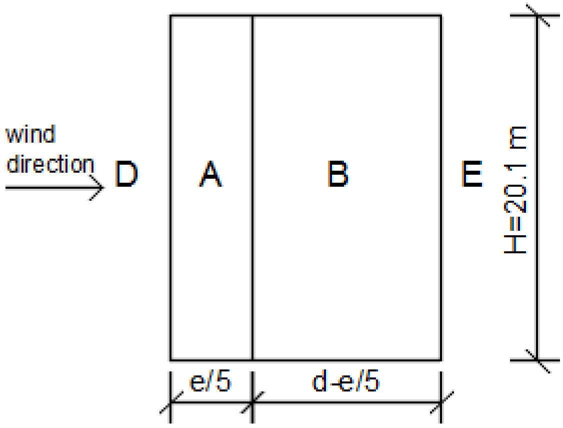


Figure B4: Elevation - short side.

$$a := \frac{h_{tot}}{d} = 1.256$$

Table B1: Wind pressure on walls.

Exterior Wall $C_{pe.10}$		$q_{p.h}$ (kN/m^2)	$q_{p.b}$ (kN/m^2)	
		1.094	1.035	
Zone	$C_{pe.10}$	w_e (h)	w_e(b)	
A	-1.2	-1.313	-1.242	suction
B	-0.8	-0.875	-0.828	suction
D	0.8	0.875	0.828	pressure
E	-0.51	-0.558	-0.528	suction

$w_{D.h} := 0.875 \frac{kN}{m^2}$ $\quad w_{D.b} := 0.828 \frac{kN}{m^2}$ $\quad w_{E.h} := -0.558 \frac{kN}{m^2}$ $\quad w_{E.b} := -0.528 \frac{kN}{m^2}$

$q_{tot.wall.h} := w_{D.h} - w_{E.h} = 1.433 \cdot \frac{kN}{m^2}$ Wall D top + E top

$q_{tot.wall.b} := w_{D.b} - w_{E.b} = 1.356 \cdot \frac{kN}{m^2}$ Wall D bottom + E bottom

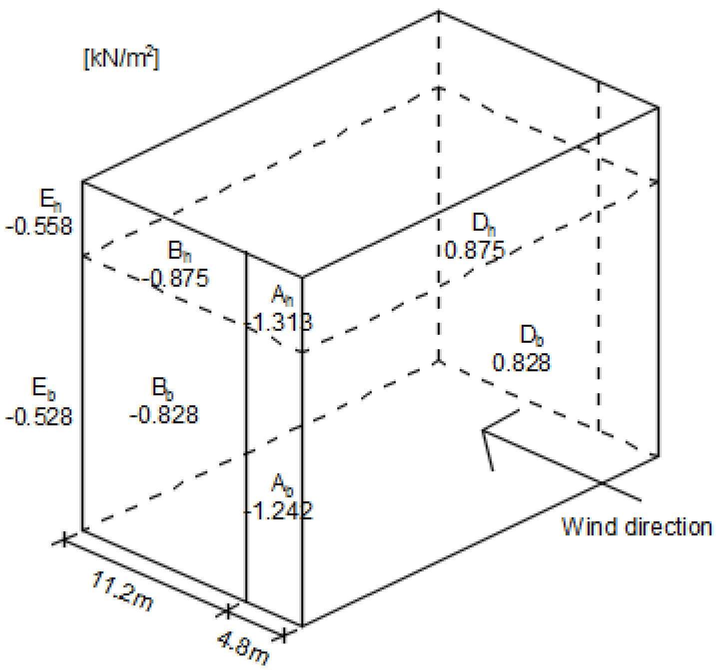


Figure B5: Wind pressure (kN/m2) on walls for the building.

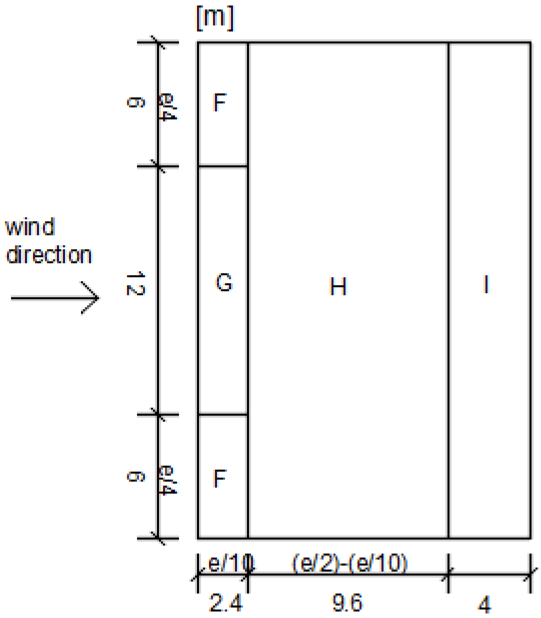


Figure B6: Roof plan with affcted area by wind load.

Table B2: Wind pressure on roof.

Flat Roof $C_{pe.10}$		$q_{p.h}$ (kN/m^2)	
		1.094	
Zone	$C_{pe.10}$	w_e	
F	-1.0	-1.094	suction
G	-1.2	-1.313	suction
H	-0.4	-0.438	suction
I	-0.2	-0.219	suction

$$w_G := -1.313 \frac{kN}{m^2}$$

$$w_H := -0.438 \frac{kN}{m^2}$$

$$w_I := -0.219 \frac{kN}{m^2}$$

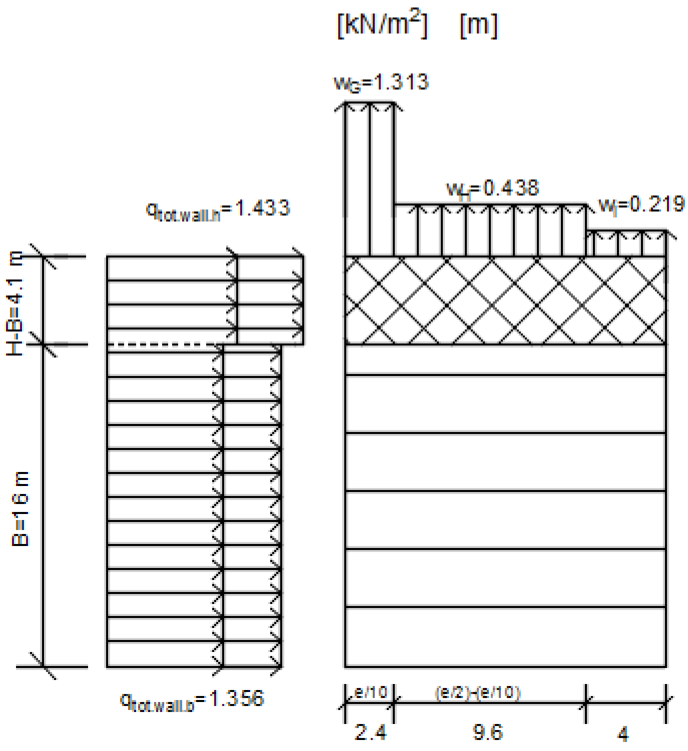


Figure B7: Elevation of short side with wind pressure coming on the long facade.

Tilting

$$e_{core} := \frac{B}{6} = 2.667\,m$$

Self-weight as unfavorable

Lever arm

$$x_G := \frac{B}{2} - \frac{e}{2 \cdot 10} + e_{core} = 9.467\,m$$

$$x_H := \frac{B}{2} - \frac{e}{10} - 0.5 \cdot \left(\frac{e}{2} - \frac{e}{10}\right) + e_{core} = 3.467\,m$$

$$x_I := \frac{B}{2} - e_{core} - 0.5 \cdot \left(B - \frac{e}{2}\right) = 3.333\,m$$

$$\gamma_G := 1.5$$

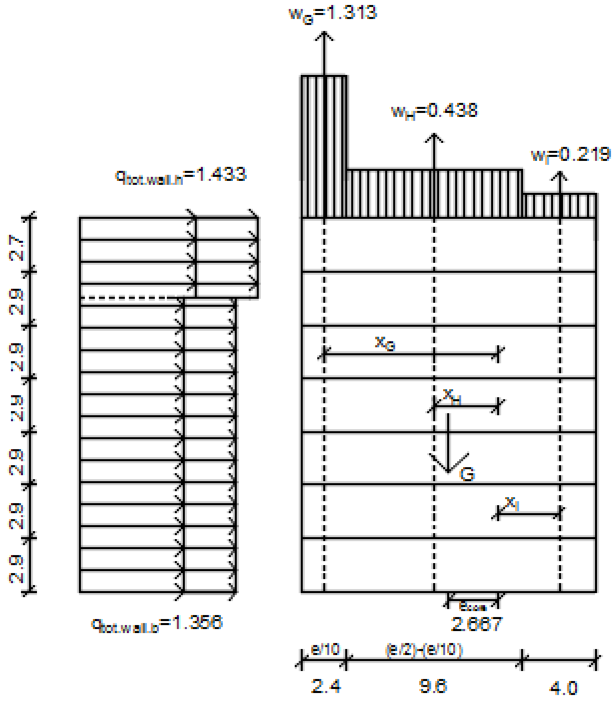


Figure B8: Lever arms between wind loads and e_{core}.

$$M_{horisontal} := \gamma_G \cdot q_{tot.wall.b} \cdot L \cdot \begin{pmatrix} h_{floor} \cdot 2.9m + h_{floor} \cdot 5.8m + h_{floor} \cdot 8.7m \ldots \\ + h_{floor} \cdot 11.6m + h_{floor} \cdot 14.5m \end{pmatrix} \ldots = 1.068 \times 10^4 \cdot kN \cdot m$$
$$+ \gamma_G \cdot q_{tot.wall.h} \cdot L \cdot \left(h_{floor} \cdot 17.4m + \frac{h_{floor7}}{2} \cdot 20.1m\right) \ldots$$
$$+ H_{d7} \cdot 20.1m + H_{d6} \cdot 17.4m + H_{d5} \cdot 14.5m + H_{d4} \cdot 11.6m \ldots$$
$$+ H_{d3} \cdot 8.7m + H_{d2} \cdot 5.8m + H_{d1} \cdot 2.9m$$

$$M_{vertical} := \gamma_G \cdot L \cdot \left[w_G \cdot \frac{e}{10} \cdot x_G + w_H \cdot \left(\frac{e}{2} - \frac{e}{10}\right) \cdot x_H - w_I \cdot \left(B - \frac{e}{2}\right) \cdot x_I\right] = -1.494 \times 10^3 \cdot kN \cdot m$$

$$M_{tilting} := M_{horisontal} + M_{vertical} = 9.189 \times 10^3 \cdot kN \cdot m$$

Resisting moment including self-weight of foundation

$$V_{tot} := V_{d7} + V_{d6} + V_{d5} + V_{d4} + V_{d3} + V_{d2} + V_{d1} + G_{k.foundation} \cdot B \cdot L = 7.013 \times 10^3 \cdot kN$$

$$M_{resisting} := V_{tot} \cdot e_{core} = 1.87 \times 10^4 \cdot kN \cdot m$$

$$\frac{M_{tilting}}{M_{resisting}} = 0.491 \qquad <1 \qquad \text{OK}$$

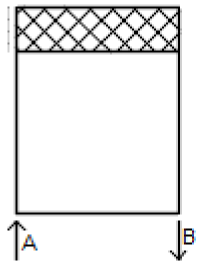


Figure B9: Moment calculations - Point A and B.

Wind coming on the long facade

Moment around point B

$$R_A := \frac{-\left[\begin{aligned}&q_{tot.wall.b}\cdot B\cdot L\cdot\frac{B}{2} + q_{tot.wall.h}\cdot\left(h_{tot}-B\right)\cdot L\cdot\left(B+\frac{h_{tot}-B}{2}\right)\,\ldots\\ &+w_G\cdot\frac{e}{10}\cdot L\cdot\left[\frac{e}{2\cdot 10}+4m+\left(\frac{e}{2}-\frac{e}{10}\right)\right]\,\ldots\\ &+\left[w_H\cdot\left(\frac{e}{2}-\frac{e}{10}\right)\cdot L\cdot\left(4m+\frac{\frac{e}{2}-\frac{e}{10}}{2}\right)+w_I\cdot 4m\cdot L\cdot\frac{4m}{2}-V_{tot}\cdot\frac{B}{2}\right]\end{aligned}\right]}{B} = 3.215\times 10^{3}\cdot kN$$

$$R_{B1} := -\left[V_{tot} - w_G\cdot\frac{e}{10}\cdot L - w_H\cdot\left(\frac{e}{2}-\frac{e}{10}\right)\cdot L - w_I\cdot 4m\cdot L - R_A\right] = -3.995\times 10^{3}\cdot kN$$

$$R_{B.check} := \frac{-\left[\begin{aligned}&q_{tot.wall.b}\cdot B\cdot L\cdot\frac{B}{2} + q_{tot.wall.h}\cdot\left(h_{tot}-B\right)\cdot L\cdot\left(B+\frac{h_{tot}-B}{2}\right)\,\ldots\\ &+-w_G\cdot\frac{e}{10}\cdot L\cdot\frac{e}{2\cdot 10} - w_H\cdot\left(\frac{e}{2}-\frac{e}{10}\right)\cdot L\cdot\left(\frac{e}{10}+\frac{\frac{e}{2}-\frac{e}{10}}{2}\right)\,\ldots\\ &+-w_I\cdot 4m\cdot L\cdot(B-2m) + V_{tot}\cdot\frac{B}{2}\end{aligned}\right]}{B} = -3.995\times 10^{3}\cdot kN$$

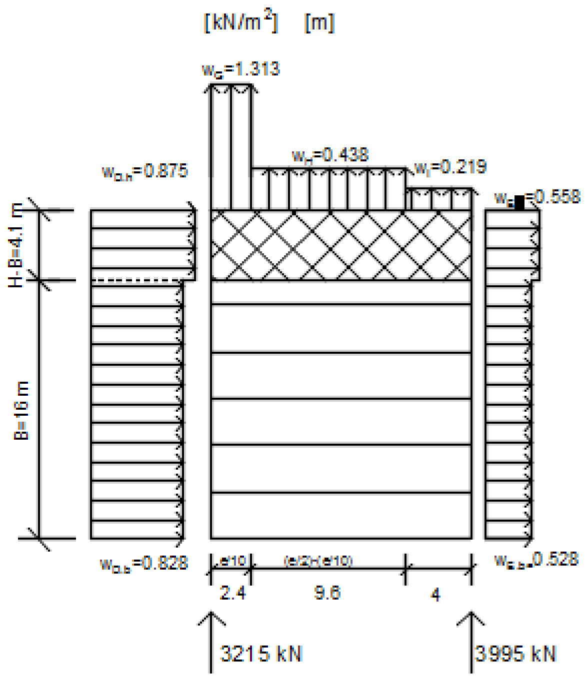


Figure B10: Wind loads acting on the building (wind coming on long facade).

Wind coming on the short facade

Unintended Inclination

$\alpha_0 := 0.003$ Systematic part of inclination angle

$\alpha_d := 0.012$ Random part of inclination angle

$n := 4$ Number of load bearing walls

$$\alpha_{md} := \alpha_0 + \frac{\alpha_d}{\sqrt{n}} = 9 \times 10^{-3}$$

Horizontal forces (self weight as unfavorable)

$H_{d7} := V_{d7} \cdot \alpha_{md} = 1.555 \cdot kN$

$H_{d6} := V_{d6} \cdot \alpha_{md} = 6.193 \cdot kN$

$H_{d5} := V_{d5} \cdot \alpha_{md} = 6.193 \cdot kN$

$H_{d4} := V_{d4} \cdot \alpha_{md} = 6.193 \cdot kN$

$H_{d3} := V_{d3} \cdot \alpha_{md} = 6.193 \cdot kN$

$H_{d2} := V_{d2} \cdot \alpha_{md} = 6.193 \cdot kN$

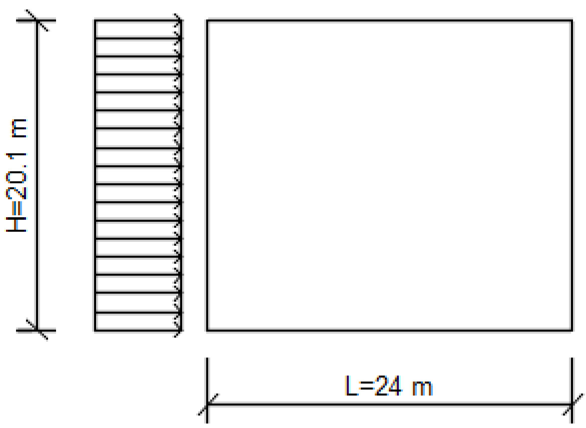
$H_{d1} := V_{d1} \cdot \alpha_{md} = 6.193 \cdot kN$

Horisontal loads
Wind pressure on walls

$q_{p.h} := c_{e.h} \cdot q_b = 1.094 \cdot \frac{kN}{m^2}$

$e := \min\left(B, 2 \cdot h_{tot}\right) = 16\, m$

$d := L = 24\, m$ $a := \frac{h_{tot}}{d} = 0.838$



$h < d$

Figure B11: Elevation of the long facade when the wind is coming on the short side.

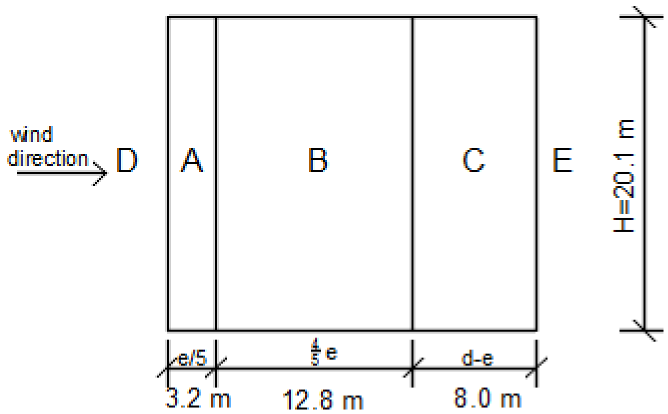


Figure B12: Elevation - long side.

e < d
Elevation - long side

Table B3: Wind pressure on walls.

Exterior Wall $C_{pe.10}$		$q_{p.h}$ (kN/m^2) 1.094	
Zone	$C_{pe.10}$	w_e (h)	
A	-1.2	-1.313	suction
B	-0.8	-0.875	suction
C	-0.5	-0.547	suction
D	0.8	0.875	pressure
E	-0.46	-0.503	suction

$$w_D := 0.875\,\frac{kN}{m^2}$$

$$w_E := -0.503\,\frac{kN}{m^2}$$

$$q_{tot.wall} := w_D - w_E = 1.378\cdot\frac{kN}{m^2}$$

Wall D + E

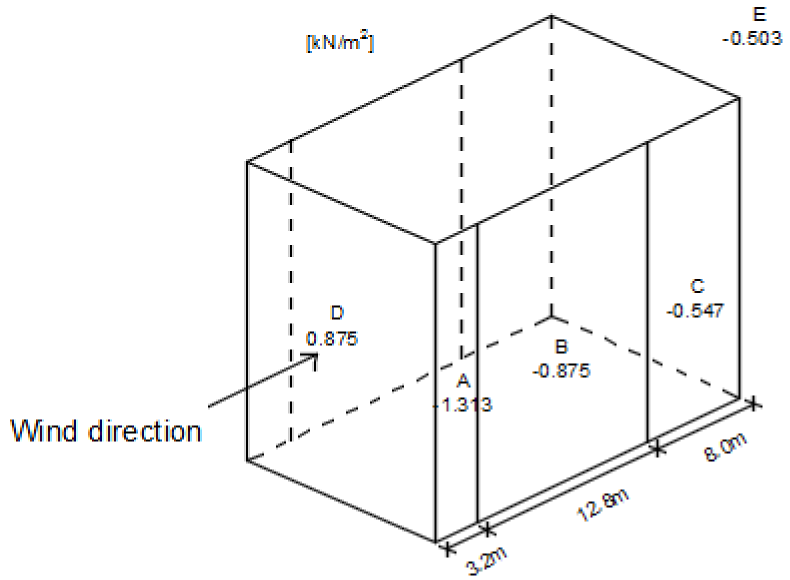


Figure B13: Wind pressure (kN/m2) on walls for the building.

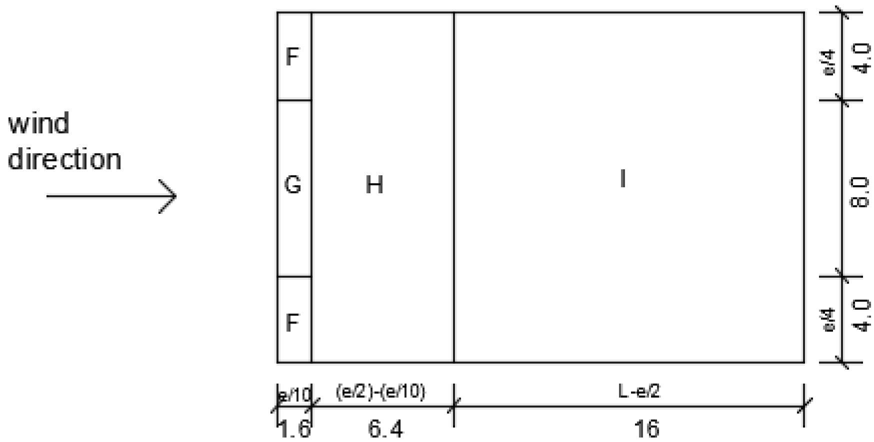


Figure B14: Elevation - long side

Table B4: WInd pressure on roof.

Flat Roof $C_{pe.10}$		$q_{p.h}$ (kN/m^2)	
		1.094	
Zone	$C_{pe.10}$	w_e	
F	-1.0	-1.094	suction
G	-1.2	-1.313	suction
H	-0.4	-0.438	suction
I	-0.2	-0.219	suction

$$w_G := -1.313 \frac{kN}{m^2}$$

$$w_H := -0.438 \frac{kN}{m^2}$$

$$w_I := -0.219 \frac{kN}{m^2}$$

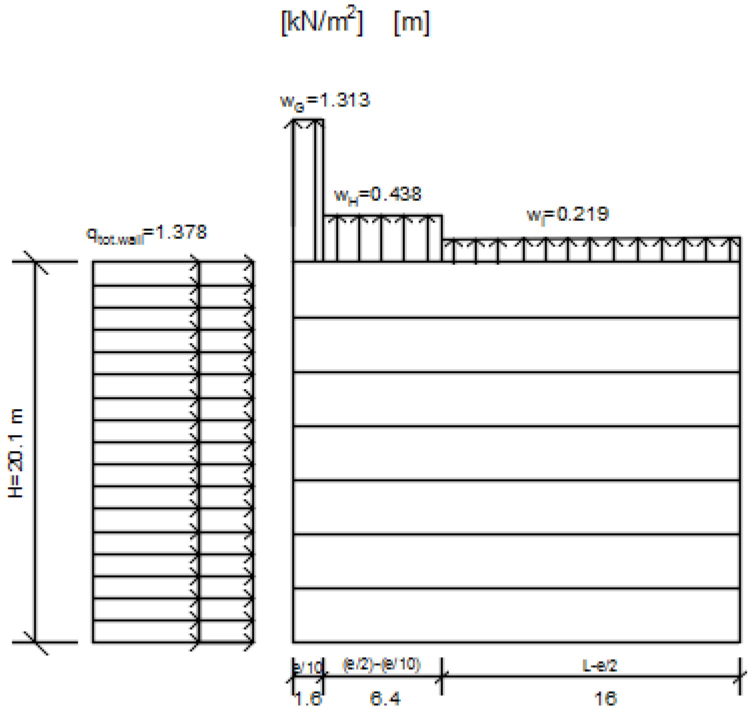


Figure B15: Elevation of long side with wind pressure coming on the short facade.

Tilting

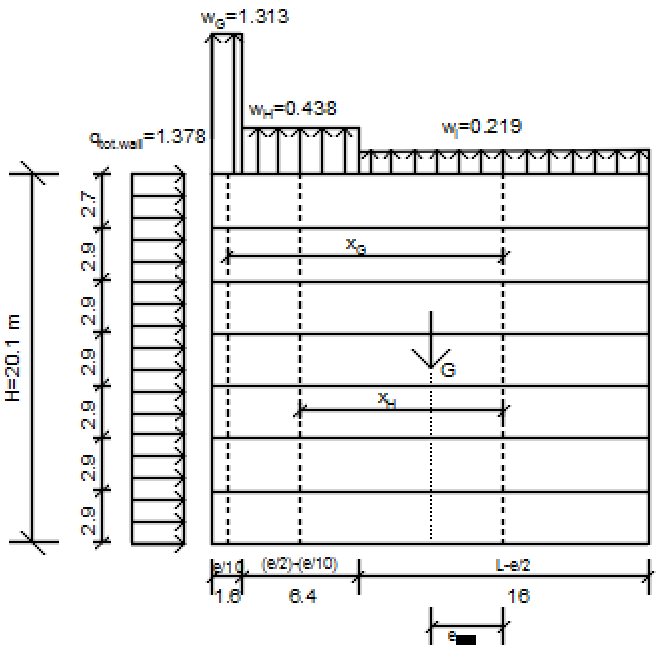


Figure B16: Lever arms between wind loads and e_{core}.

$e_{core} := \frac{L}{6} = 4\,m$

Self-weight as unfavorable

Lever arm

$x_G := \frac{L}{2} - \frac{e}{2 \cdot 10} + e_{core} = 15.2\,m$

$x_H := \frac{L}{2} - \frac{e}{10} - 0.5 \cdot \left(\frac{e}{2} - \frac{e}{10}\right) + e_{core} = 11.2\,m$

$x_I := \frac{L}{2} - e_{core} - 0.5 \cdot \left(L - \frac{e}{2}\right) = 0\,m$

$\gamma_G := 1.5$

$$M_{horisontal} := \gamma_G \cdot q_{tot.wall} \cdot B \cdot \begin{pmatrix} h_{floor} \cdot 2.9m + h_{floor} \cdot 5.8m + h_{floor} \cdot 8.7m \ldots \\ + h_{floor} \cdot 11.6m + h_{floor} \cdot 14.5m \ldots \\ + h_{floor} \cdot 17.4m + h_{floor7} \cdot 20.1m \end{pmatrix} \ldots = 8.044 \times 10^3 \cdot kN \cdot m$$
$$+ \begin{pmatrix} H_{d7} \cdot 20.1m + H_{d6} \cdot 17.4m + H_{d5} \cdot 14.5m + H_{d4} \cdot 11.6m + H_{d3} \cdot 8.7m \ldots \\ + H_{d2} \cdot 5.8m + H_{d1} \cdot 2.9m \end{pmatrix}$$

$$M_{vertical} := \gamma_G \cdot B \cdot \left[w_G \cdot \frac{e}{10} \cdot x_G + w_H \cdot \left(\frac{e}{2} - \frac{e}{10}\right) \cdot x_H - w_I \cdot \left(L - \frac{e}{2}\right) \cdot x_I \right] = -1.52 \times 10^3 \cdot kN \cdot m$$

$M_{tilting} := M_{horisontal} + M_{vertical} = 6.524 \times 10^3 \cdot kN \cdot m$

Resisting moment including self-weight of foundation

$V_{tot} := V_{d7} + V_{d6} + V_{d5} + V_{d4} + V_{d3} + V_{d2} + V_{d1} + G_{k.foundation} \cdot B \cdot L = 7.013 \times 10^3 \cdot kN$

$M_{resisting1} := V_{tot} \cdot e_{core} = 2.805 \times 10^4 \cdot kN \cdot m$

$\frac{M_{tilting}}{M_{resisting1}} = 0.233$ <1 OK

Moment around point B

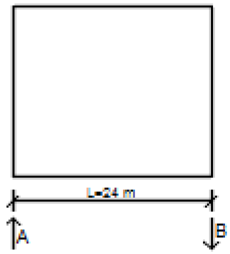


Figure B17: Moment calculations - Point A and B.

$$R_{A1} := \frac{-\left[q_{tot.wall}\cdot h_{tot}\cdot B\cdot\frac{h_{tot}}{2} + w_G\cdot\frac{e}{10}\cdot B\cdot\left(L-\frac{e}{2\cdot 10}\right) \ldots + w_H\cdot\left(\frac{e}{2}-\frac{e}{10}\right)\cdot B\cdot\left[\left(L-\frac{e}{10}\right)-\frac{\frac{e}{2}-\frac{e}{10}}{2}\right] + w_I\cdot\left(L-\frac{e}{2}\right)\cdot B\cdot\left(\frac{L-\frac{e}{2}}{2}\right) - V_{tot}\cdot\frac{L}{2}\right]}{L} = 3.408\times 10^{3}\cdot kN$$

$$R_{B1} := -\left[V_{tot} - w_G\cdot\frac{e}{10}\cdot B - w_H\cdot\left(\frac{e}{2}-\frac{e}{10}\right)\cdot B - w_I\cdot\left(L-\frac{e}{2}\right)\cdot B - R_{A1}\right] = -3.739\times 10^{3}\cdot kN$$

$$R_{B1.check} := \frac{-\left[w_D\cdot h_{tot}\cdot B\cdot\frac{h_{tot}}{2} - w_E\cdot h_{tot}\cdot B\cdot\frac{h_{tot}}{2} - w_G\cdot\frac{e}{10}\cdot B\cdot\frac{e}{2\cdot 10} \ldots + -w_H\cdot\left(\frac{e}{2}-\frac{e}{10}\right)\cdot B\cdot\left(\frac{e}{10}+\frac{\frac{e}{2}-\frac{e}{10}}{2}\right) - w_I\cdot\left(L-\frac{e}{2}\right)\cdot B\cdot\left[L-\frac{\left(L-\frac{e}{2}\right)}{2}\right] \ldots + V_{tot}\cdot\frac{L}{2}\right]}{L} = -3.739\times 10^{3}\cdot kN$$

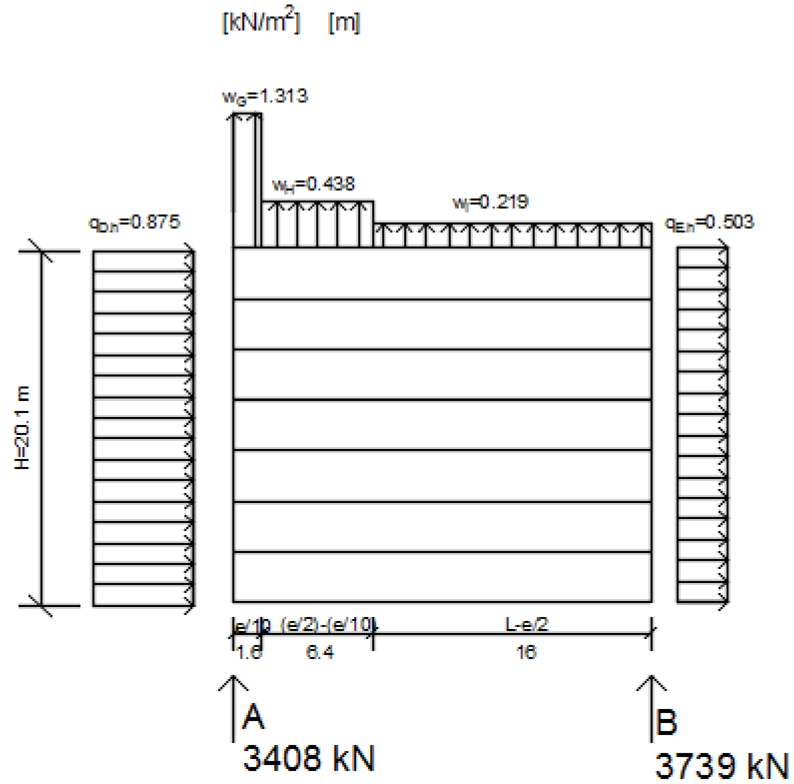
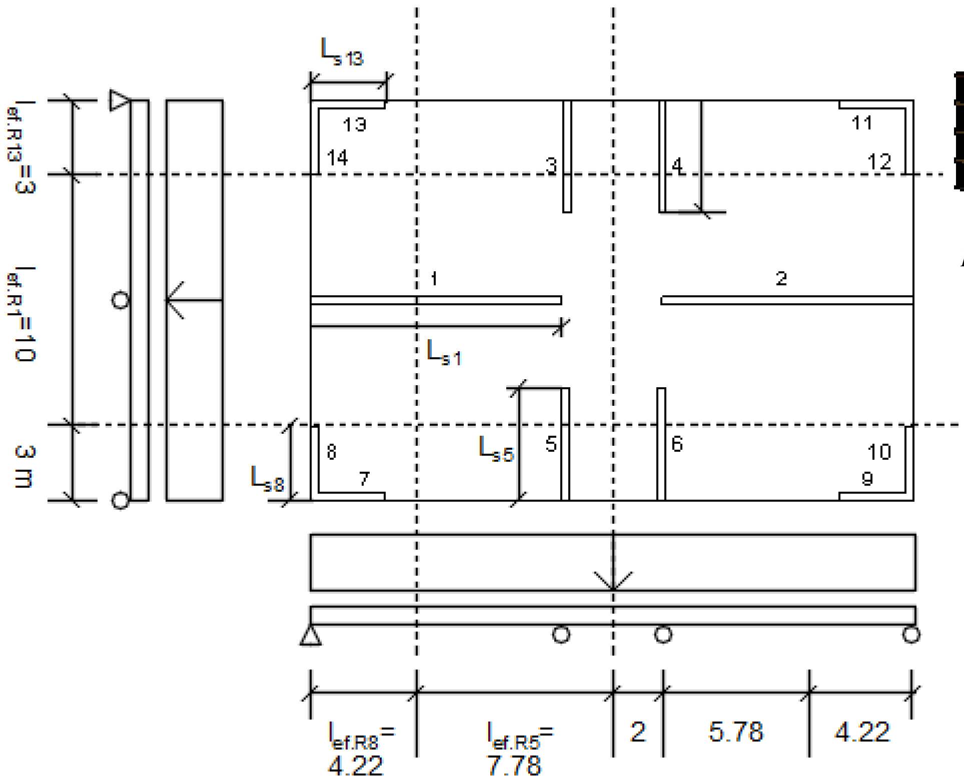


Figure B18: Wind loads acting on the building (wind coming on short facade).

APPENDIX C: Reference building - 4 stories -Uplifting forces



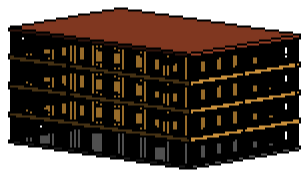


Figure C1 : Building in 3D.

Figure C2 : Plan with shear walls and effective lengths.

Geometry

$h_{storey} := 2.4m$	(Inner) Height of one storey
$h_{floor7} := 2.7m$	Height of floor 7 including roof
$h_{floor} := 2.9m$	Height of floor 1-6 including floor slab
$h_{tot} := 11.4m$	Total height of building
$B := 16m$	Width of building
$L := 24m$	Length of building
$L_{s5} := 4.483m$	Length of stabilizing wall 5
$L_{s8} := 3m$	Length of stabilizing wall 8
$l_{ef.R5} := 7.78m$	Effective width of wall resisted by stabilizing wall 5
$l_{ef.R8} := 4.22m$	Effective width of wall resisted by stabilizing wall 8
$L_{s1} := 9.997m$	Length of stabilizing wall 1
$L_{s13} := 3m$	Length of stabilizing wall 13
$l_{ef.R1} := 10m$	Effective width of wall resisted by stabilizing wall R1
$l_{ef.R13} := 3m$	Effective width of wall resisted by stabilizing wall R13
$H_{d1} := 7.903kN$	Unintended inclination on first floor

Wind load (shear walls, wind coming on long facade)

$\gamma_G := 1.5$

$G_{k.shaft} := 1.201 \frac{kN}{m^2}$ Self-weight of shaft

$g_{stab.wall.5} := G_{k.shaft} \cdot h_{storey} \cdot L_{s5} = 12.922 \cdot kN$ Self-weight of wall 5 on each floor

$g_{stab.wall.8} := 0.8 \frac{kN}{m^2} \cdot h_{storey} \cdot L_{s8} = 5.76 \cdot kN$ Self-weight of wall 8 on each floor

$q_{tot.wall} := 1.147 \frac{kN}{m^2}$ Total wind load on wall, from Appendix A

$q_5 := \frac{q_{tot.wall} \cdot (h_{tot} - 0.5 \cdot h_{storey}) \cdot l_{ef.R5}}{2} = 45.511 \cdot kN$ Wind load resisted by wall 5. (Half of the total wind load is resisted by wall 5, the other half by wall 3.)

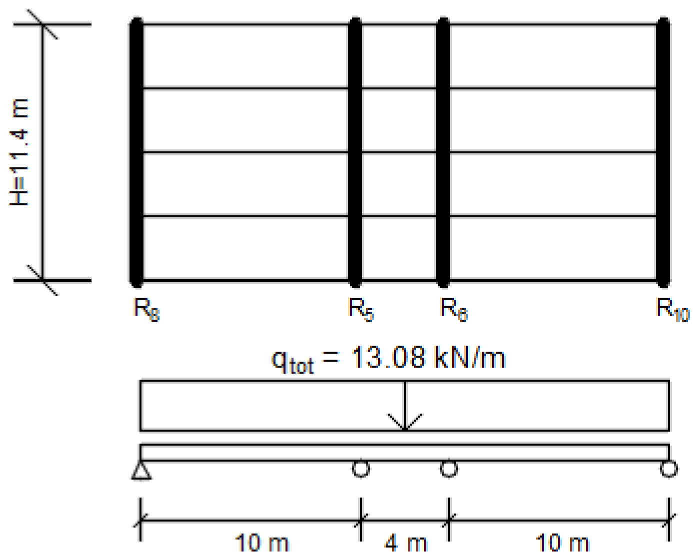
$R_5 := q_5 = 45.511 \cdot kN$ Total wind load resisted by wall 5

$q_8 := \frac{q_{tot.wall} \cdot (h_{tot} - 0.5 \cdot h_{storey}) \cdot l_{ef.R8}}{2} = 24.686 \cdot kN$ Wind load resisted by wall 8. (Half of the total wind load is resisted by wall 8, the other half by wall 14.)

$R_8 := q_8 = 24.686 \cdot kN$ Total wind load resisted by wall 8

$H_1 := 7.903kN$ Unintended inclination on the bottom floor

$q_{tot} := q_{tot.wall} \cdot h_{tot} = 13.076 \cdot \frac{kN}{m}$ Distributed wind load in kN/m



R_i = Stabilizing wall

Figure C3 : Section of long facade with shear walls.

Shear wall 5 (massive timber)

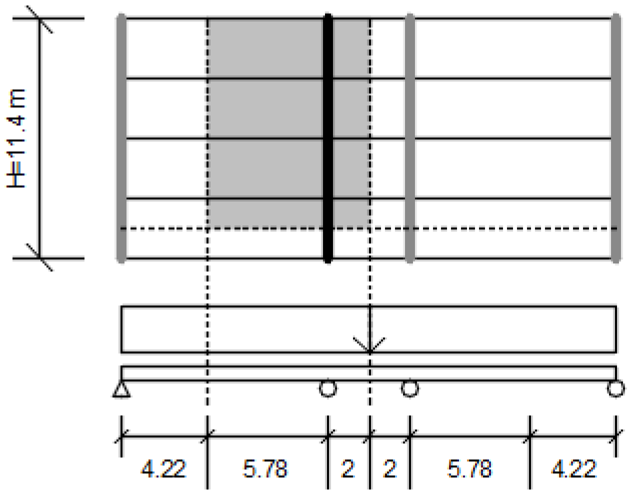


Figure C4: Elevation of long facade and affected area by wind load for shear wall 5.

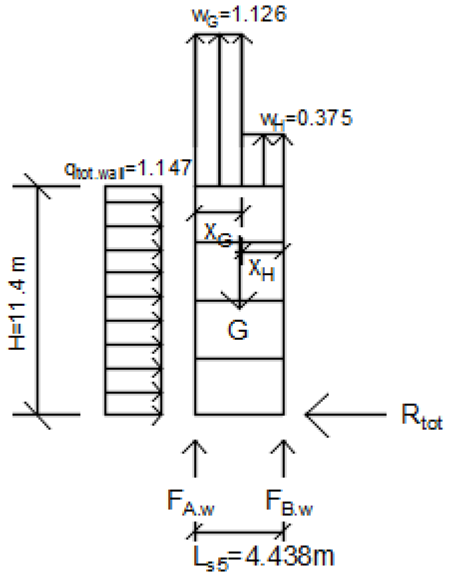


Figure C5: Elevation of shear wall 5.

$$w_G := 1.126\frac{kN}{m^2} \qquad x_G := 2.28m$$

$$w_H := 0.375\frac{kN}{m^2} \qquad x_H := L_{s5} - x_G = 2.203\,m$$

Wind loads on roof, see Appendix A

Moment around B

There are 3 stabilizing walls above the one on the first floor.

$$F_{A.w5} := \frac{g_{stab.wall.5}\cdot 3\cdot\frac{L_{s5}}{2} - H_1\cdot h_{floor} - \gamma_G\cdot\left[R_5\cdot\left[0.5\cdot h_{storey} + \frac{\left(h_{tot} - 0.5\cdot h_{storey}\right)}{2}\right]\right] \cdots + -w_G\cdot x_G\cdot L_{s5}\cdot\left(L_{s5} - \frac{x_G}{2}\right) - w_H\cdot x_H\cdot L_{s5}\cdot\left(\frac{x_H}{2}\right) - H_{d1}\cdot 2.9m}{L_{s5}} = -96.269\cdot kN$$

$$F_{B.w5} := g_{stab.wall.5}\cdot 3 + \left|F_{A.w5}\right| - w_G\cdot x_G\cdot L_{s5} - w_H\cdot x_H\cdot L_{s5} = 119.822\cdot kN$$

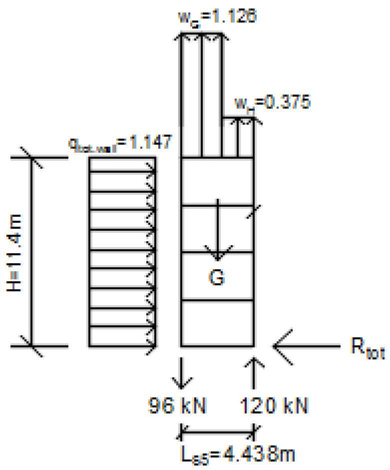


Figure C6: Tensile and compression forces for shear wall 5.

Shear wall 8 (exterior studs with panels)

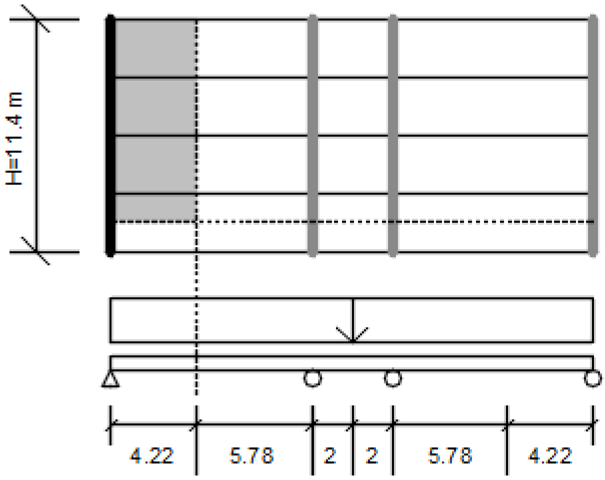


Figure C7: Elevation of long facade and affected area by wind load for shear wall 8.

Total horizontal wind load acting on shear wall at the bottom floor

$V_{d.shear} := R_8 = 24.686 \cdot kN$

Unintended inclination at the bottom floor

$H_{d4} := 1.985kN$

$H_{d3} := 7.903kN$

$H_{d2} := 7.903kN$

$H_{d1} := 7.903kN$

$L_{s8} = 3\ m$ — Length of stabilizing wall 8

$b_{window} := 1.2m$ — Width of window

$n := 1$ — Number of windows

$L_{eff8} := L_{s8} - n \cdot b_{window} = 1.8\ m$ — Effective length of stabilizing wall 8

Panels are made of OSB sheet, 12 mm thick on both sides

$b_{panel} := 1.2m$ — Width of panel

$h_{panel} := 2.9m$ — Height of panel (storey heigth and floor thickness)

Number of panels needed:

$$n_{panel} := \frac{L_{eff8}}{b_{panel}} = 1.5$$

2 panels are needed

According to EC5 9.2.4.2 (2), if the width of the panel is less than h/4 the contribution to transmitting shear forces wil be neglected. This means 2 panels are used.

$n_{panel} := 2$

$$F_{anch8} := \frac{g_{stab.wall.8} \cdot 3 \cdot \frac{L_{s8}}{2} - H_1 \cdot \frac{h_{storey}}{2} - \gamma_G \cdot \left[R_8 \cdot \left[0.5 \cdot h_{storey} + \frac{\left(h_{tot} - 0.5 \cdot h_{storey}\right)}{2} \right] \right] \ldots + -w_G \cdot x_G \cdot L_{s8} \cdot \left(L_{s13} - \frac{x_G}{2} \right) - w_H \cdot \left(L_{s8} - x_G\right) \cdot L_{s8} \cdot \left(\frac{L_{s8} - x_G}{2} \right) - H_{d1} \cdot 2.9m}{b_{panel} \cdot n_{panel}} = -105.991 \cdot kN$$

Wind load (shear walls, wind coming on short facade)

$L_{s1} = 9.997\,m$ — Length of stabilizing wall R1

$L_{s13} = 3\,m$ — Length of stabilizing wall R13

$l_{ef.R1} = 10\,m$ — Effective length of stabilizing wall 1

$l_{ef.R13} = 3\,m$ — Effective length of stabilizing wall 13

$g_{stab.wall.1} := G_{k.shaft} \cdot h_{storey} \cdot L_{s1} = 28.815 \cdot kN$ — Self-weight of wall 1 on each floor

$g_{stab.wall.13} := 0.8 \frac{kN}{m^2} \cdot h_{storey} \cdot L_{s13} = 5.76 \cdot kN$ — Self-weight of wall 13 on each floor

$q_{tot.wall} := 1.023 \frac{kN}{m^2}$ — Total wind load on wall, from Appendix A

$q_1 := \frac{q_{tot.wall} \cdot (h_{tot} - 0.5 \cdot h_{storey}) \cdot l_{ef.R1}}{2} = 52.173 \cdot kN$ — Wind load resisted by wall 1. (Half of the total wind load is resisted by wall 1, the other half by wall 2.)

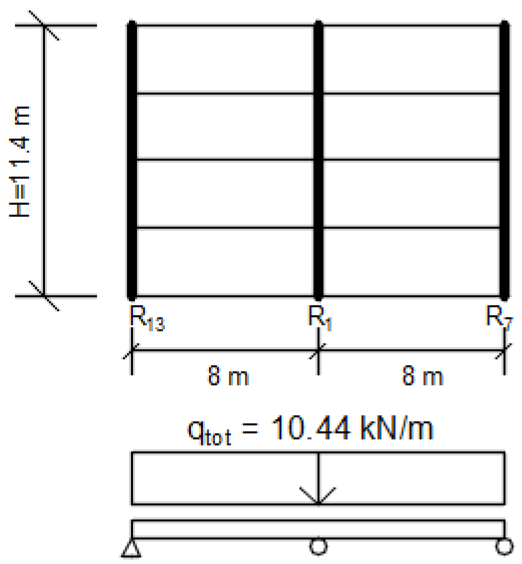
$R_1 := q_1 = 52.173 \cdot kN$ — Total wind load resisted by wall 1

$q_{13} := \frac{q_{tot.wall} \cdot (h_{tot} - 0.5 \cdot h_{storey}) \cdot l_{ef.R13}}{2} = 15.652 \cdot kN$ — Wind load resisted by wall 13. (Half of the total wind loa is resisted by wall 13, the other half by wall 11.)

$R_{13} := q_{13} = 15.652 \cdot kN$ — Total wind load resisted by wall 13

$H_1 := 6.193 kN$ — Unintended inclination on the bottom floor

$q_{tot} := q_{tot.wall} \cdot (h_{tot} - 0.5 \cdot h_{storey}) = 10.435 \cdot \frac{kN}{m}$ — Distributed wind load in kN/m



R_i = Stabilizing wall

Figure C8: Section of short facade with shear walls.

Shear wall 1

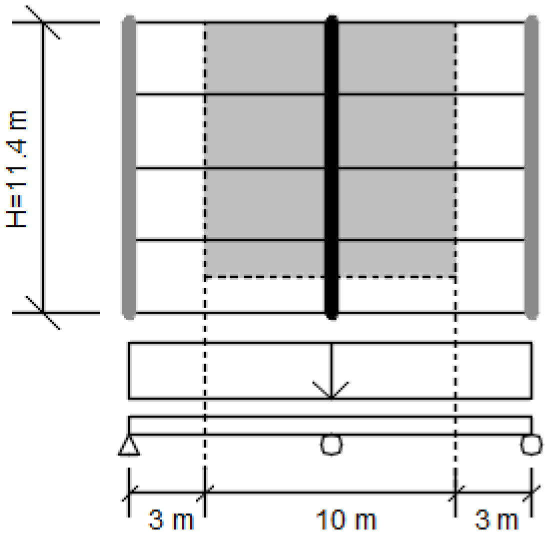


Figure C9: Section of the short facade with shear walls.

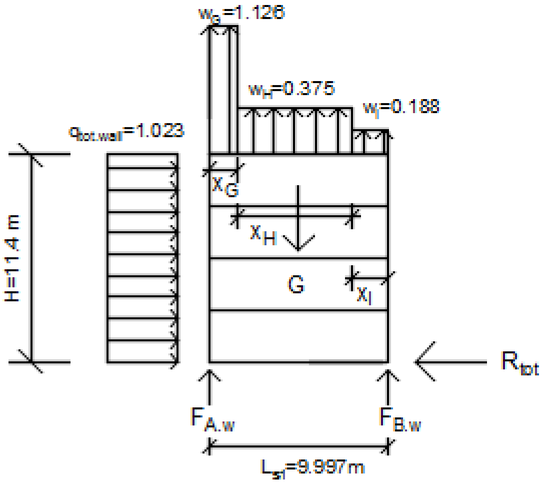


Figure C10: Shear wall subjected to wind load on the short facade.

$$w_G := 1.126\frac{kN}{m^2} \qquad x_G := 1.6m$$

$$w_H := 0.375\frac{kN}{m^2} \qquad x_H := 6.4m$$

$$w_I := 0.188\frac{kN}{m^2} \qquad x_I := L_{s1} - x_G - x_H = 1.997\,m$$

Moment around B

There are 3 stabilizing walls above the one on the first floor.

$$F_{A.w1} := \frac{g_{stab.wall.1}\cdot 3\cdot\frac{L_{s1}}{2} - H_1\cdot\frac{h_{storey}}{2} - \gamma_G\cdot\left[R_1\cdot\left[0.5\cdot h_{storey} + \frac{\left(h_{tot} - 0.5\cdot h_{storey}\right)}{2}\right]\right]\ \ldots \\ + -w_G\cdot x_G\cdot L_{s1}\cdot\left(L_{s1} - \frac{x_G}{2}\right) - w_H\cdot x_H\cdot L_{s1}\cdot\left(L_{s1} - x_G - \frac{x_H}{2}\right) - w_I\cdot x_I\cdot L_{s1}\cdot\left(\frac{x_I}{2}\right) - H_{d1}\cdot 2.9m}{L_{s1}} = -38.548\cdot k$$

$$F_{B.w1} := g_{stab.wall.1}\cdot 3 + \left|F_{A.w1}\right| - w_G\cdot x_G\cdot B - w_H\cdot x_H\cdot B - w_I\cdot x_I\cdot B = 51.762\cdot kN$$

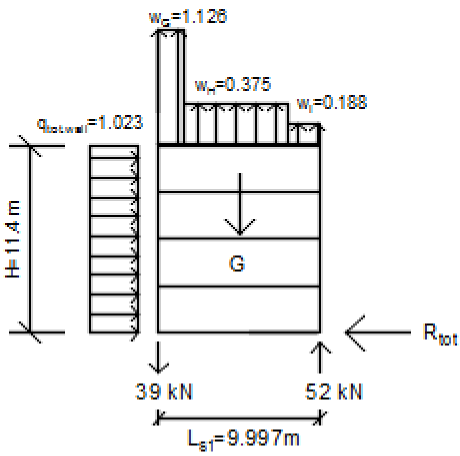


Figure C11: Tensile and compression forces for shear wall 1.

Shear wall 13 (exterior studs with panels)

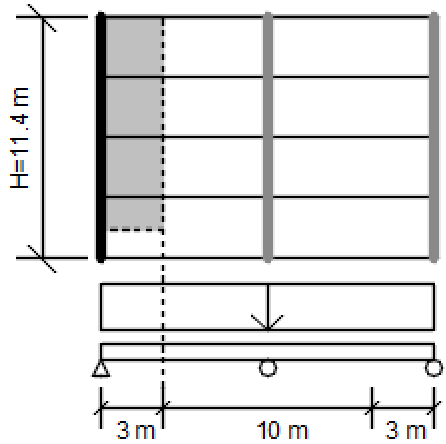


Figure C12: Section of the short facade with shear walls.

Total horizontal wind load acting on shear wall at the bottom floor

$V_{d.shear} := R_{13} = 15.652 \cdot kN$

Unintended inclination at the bottom floor

$H_{d4} := 1.555kN$

$H_{d3} := 6.193kN$

$H_{d2} := 6.193kN$

$H_{d1} := 6.193kN$

Total racking force

$H_{rack} := V_{d.shear} + H_1 = 21.845 \cdot kN$

$L_{s13} = 3\,m$ — Length of stabilizing wall 1

$b_{window} := 1.2m$ — Width of window

$n := 1$ — Number of windows

$L_{eff13} := L_{s13} - n \cdot b_{window} = 1.8\,m$ — Effective length of stabilizing wall 13

Panels are made of OSB sheet, 12 mm thick on both sides

$b_{panel} := 1.2m$ — Width of panel

$h_{panel} := 2.9m$ — Height of panel (storey heigth and floor thickness)

Number of panels needed:

$$n_{panel} := \frac{L_{eff13}}{b_{panel}} = 1.5$$

According to EC5 9.2.4.2 (2), if the width of the panel is less than h/4 the contribution to transmitting shear forces will be neglected. This means 2 panels are used.

$n_{panel} := 2$ — 2 panels are needed

$$F_{anch13} := \frac{g_{stab.wall.13} \cdot 3 \cdot \frac{L_{s13}}{2} - H_1 \cdot \frac{h_{storey}}{2} - \gamma_G \cdot \left[R_{13} \cdot \left[0.5 \cdot h_{storey} + \frac{\left(h_{tot} - 0.5 \cdot h_{storey}\right)}{2}\right]\right] \ldots + -w_G \cdot x_G \cdot L_{s13} \cdot \left(L_{s13} - \frac{x_G}{2}\right) - w_H \cdot \left(L_{s13} - x_G\right) \cdot L_{s13} \cdot \left(\frac{L_{s13} - x_G}{2}\right) - H_{d1} \cdot 2.9m}{b_{panel} \cdot n_{panel}} = -66.823 \cdot kN$$

Force that needs to be anchored

APPENDIX D - Reference building - 7 stories - Uplifting forces

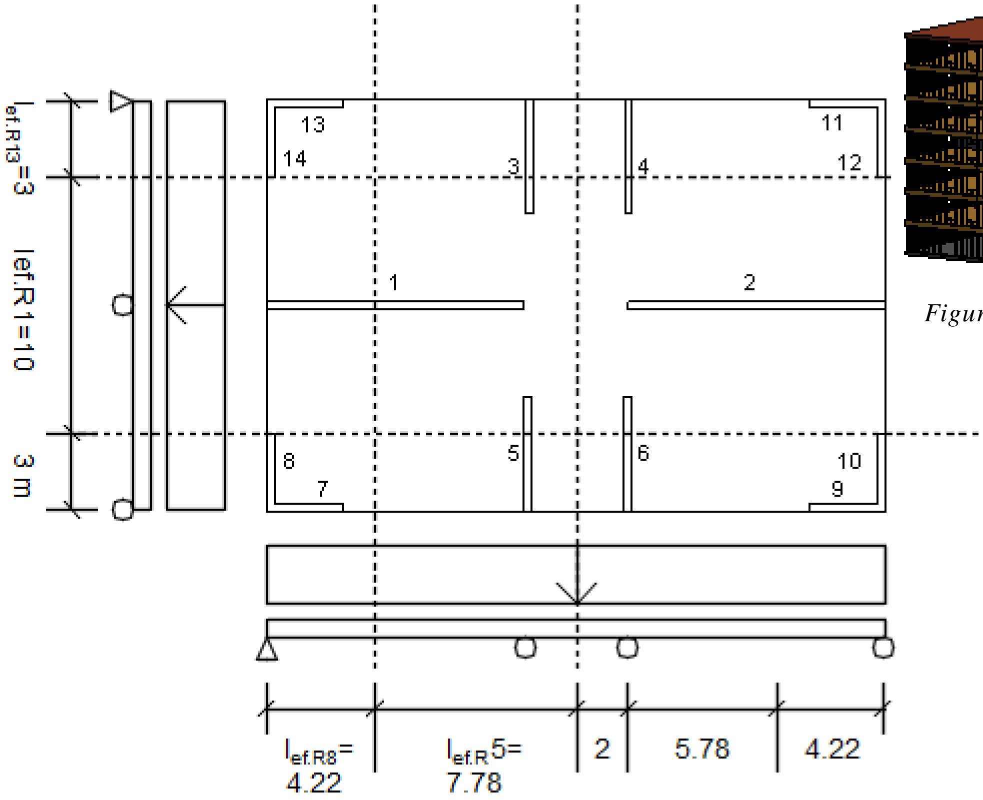


Figure D2 : Plan with shear walls and effective length.

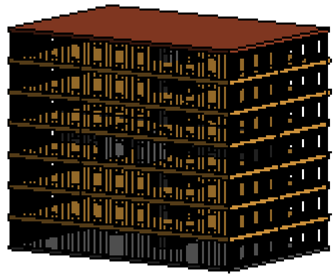


Figure D1 : Building in 3D.

Geometry

$h_{storey} := 2.4m$	(Inner) Height of one storey
$h_{floor7} := 2.7m$	Height of floor 7 including roof
$h_{floor} := 2.9m$	Height of floor 1-6 including floor slab
$h_{tot} := 20.1m$	Total height of building
$B := 16m$	Width of building
$L := 24m$	Length of building
$L_{s5} := 4.483m$	Length of stabilizing wall, R5
$L_{s8} := 3m$	Length of stabilizing wall, R8
$l_{ef.R5} := 7.78m$	Effective width of wall resisted by stabilizing wall R5
$l_{ef.R8} := 4.22m$	Effective width of wall resisted by stabilizing wall R8
$L_{s1} := 9.997m$	Length of stabilizing wall, R1
$L_{s13} := 3m$	Length of stabilizing wall, R13
$l_{ef.R1} := 10m$	Effective width of wall resisted by stabilizing wall, R1
$l_{ef.R13} := 3m$	Effective width of wall resisted by stabilizing wall R13
$H_{d1} := 7.903kN$	Unintended inclination on first floor

Wind loads (shear walls, wind coming on long facade)

$\gamma_G := 1.5$

$G_{k.shaft} := 1.201 \frac{kN}{m^2}$ Self-weight of shaft

$g_{stab.wall.5} := G_{k.shaft} \cdot h_{storey} \cdot L_{s5} = 12.922 \cdot kN$ Self-weight of wall 5

$g_{stab.wall.8} := 0.8 \frac{kN}{m^2} \cdot h_{storey} \cdot L_{s8} = 5.76 \cdot kN$ Self-weight of wall 8

$q_{tot.wall.h} := 1.433 \frac{kN}{m^2}$ Total wind load on top wall, from Appendix A

$q_{tot.wall.b} := 1.356 \frac{kN}{m^2}$ Total wind load on bottom wall, from Appendix A

$q_{h5} := \frac{q_{tot.wall.h} \cdot (h_{tot} - B) \cdot l_{ef.R5}}{2} = 22.855 \cdot kN$ Wind load resisted by top wall 5. (Half of the total wind load is resisted by wall 5, the other half by wall 3.)

$R_{5h} := q_{h5} = 22.855 \cdot kN$

$q_{b5} := \frac{q_{tot.wall.b} \cdot \left(B - \frac{h_{storey}}{2}\right) \cdot l_{ef.R5}}{2} = 78.068 \cdot kN$ Wind load resisted by bottom wall 5. (Half of the total wind load is resisted by wall 5, the other half by wall 3.)

$R_{5b} := q_{b5} = 78.068 \cdot kN$

$R_5 := R_{5h} + R_{5b} = 100.923 \cdot kN$ Total wind load resisted by wall 5

$q_{h8} := \frac{q_{tot.wall.h} \cdot (h_{tot} - B) \cdot l_{ef.R8}}{2} = 12.397 \cdot kN$ Wind load resisted by top wall 8. (Half of the total wind load is resisted by wall 8, the other half by wall 14.)

$R_{8h} := q_{h8} = 12.397 \cdot kN$

$q_{b8} := \frac{q_{tot.wall.b} \cdot \left(B - \frac{h_{storey}}{2}\right) \cdot l_{ef.R8}}{2} = 42.345 \cdot kN$ Wind load resisted by bottom wall 8. (Half of the total wind load is resisted by wall 8, the other half by wall 14.)

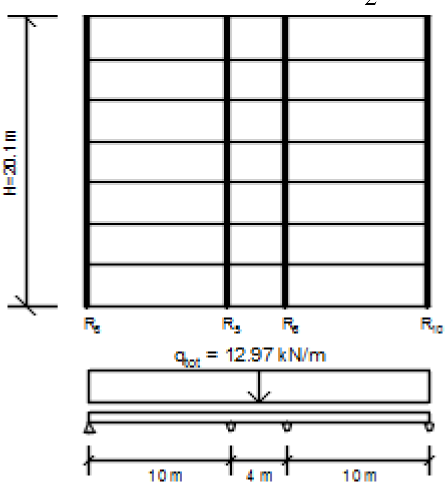
$R_{8b} := q_{b8} = 42.345 \cdot kN$

$R_8 := R_{8h} + R_{8b} = 54.742 \cdot kN$ Total wind load resisted by wall 8

Unintended inclination on the bottom floor

$H_1 := 7.903 kN$

$q_{tot} := \frac{q_{tot.wall.h} \cdot (h_{tot} - B) + q_{tot.wall.b} \cdot (B - 0.5 \cdot h_{storey})}{2} = 12.972 \cdot \frac{kN}{m}$ Distributed wind load in kN/m



R_i = Stabilizing wall

Figure D3: Section of long facade with shear walls.

Shear wall 5

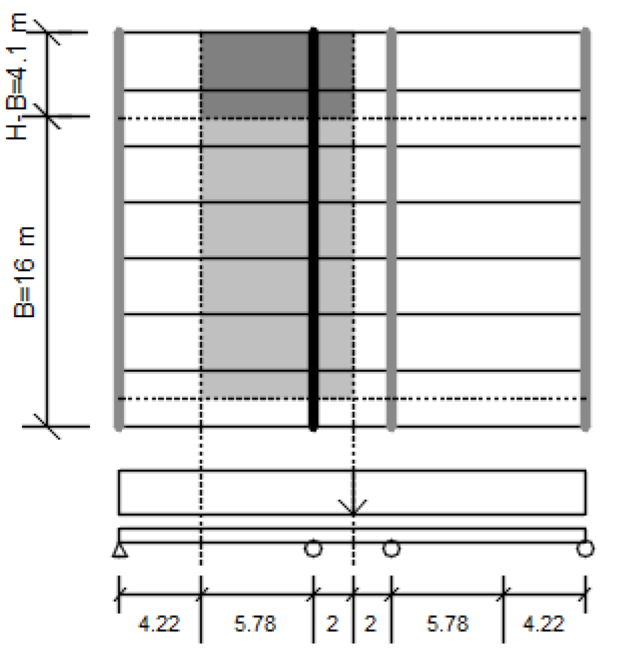


Figure D4: Elevation of long facade and affected area by wind load for shear wall 5.

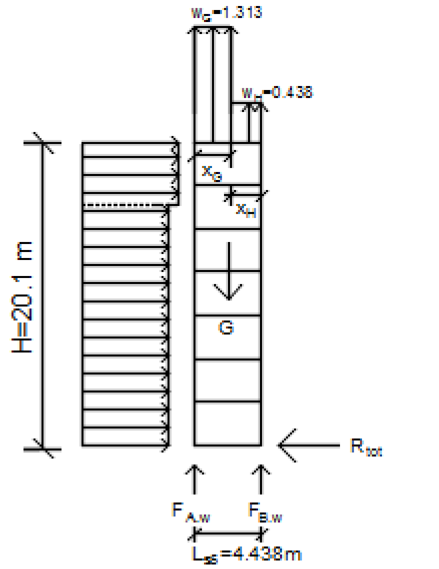


Figure D5: Elevation of shear wall 5.

$$w_G := 1.313\frac{kN}{m^2} \qquad x_G := 2.4m$$

$$w_H := 0.438\frac{kN}{m^2} \qquad x_H := L_{s5} - x_G = 2.083\,m$$

Wind loads on roof, see Appendix A

Moment around B

There are 6 stabilizing walls above the one on the first floor.

$$F_{A.w5} := \frac{g_{stab.wall.5}\cdot 6\cdot\frac{L_{s5}}{2} - H_1\cdot\frac{h_{storey}}{2} - \gamma_G\cdot\left[R_{5h}\cdot\left(B + \frac{h_{tot} - B}{2}\right) + R_{5b}\cdot\frac{\left(B - 0.5\cdot h_{storey}\right)}{2}\right] \ldots \\ + -w_G\cdot x_G\cdot L_{s5}\cdot\left(L_{s5} - \frac{x_G}{2}\right) - w_H\cdot x_H\cdot L_{s5}\cdot\left(\frac{x_H}{2}\right) - H_{d1}\cdot 2.9m}{L_{s5}} = -311.087\cdot kN$$

$$F_{B.w5} := g_{stab.wall.5}\cdot 6 + \left|F_{A.w5}\right| - w_G\cdot x_G\cdot L_{s5} - w_H\cdot x_H\cdot L_{s5} = 370.401\cdot kN$$

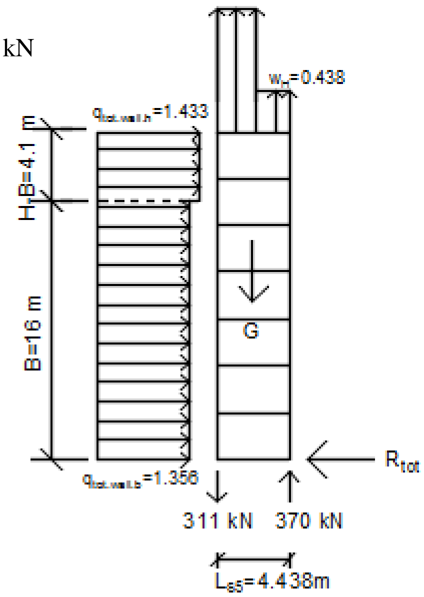


Figure D6: Tensile and compression forces for shear wall 5.

Shear wall 8 (exterior studs with panels)

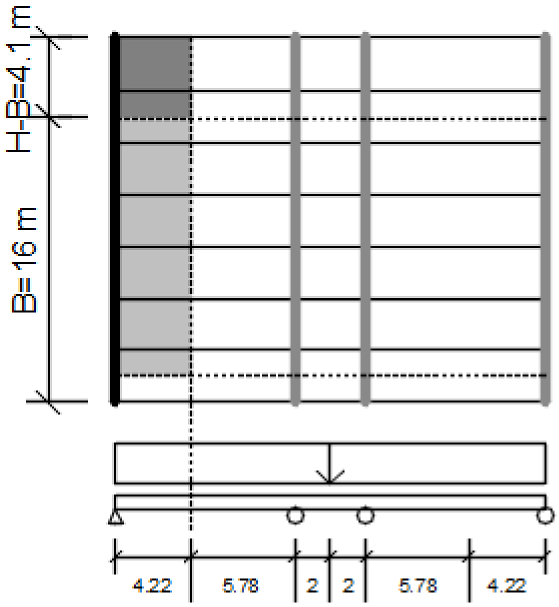


Figure D7: Elevation of long facade and affected area by wind load for shear wall 8.

Total horizontal wind load acting on shear wall at the bottom floor

$V_{d.shear} := R_8 = 54.742 \cdot kN$

Unintended inclination at the bottom floor

$H_{d7} := 1.985kN$

$H_{d6} := 7.903kN$

$H_{d5} := 7.903kN$

$H_{d4} := 7.903kN$

$H_{d3} := 7.903kN$

$H_{d2} := 7.903kN$

$H_{d1} := 7.903kN$

Total racking force

$H_{rack} := V_{d.shear} + H_1 = 62.645 \cdot kN$ Racking force on bottom floor

$L_{s8} = 3\,m$ Length of stabilizing wall 8

$b_{window} := 1.2m$ Width of window

$n := 1$ Number of windows

$L_{eff8} := L_{s8} - n \cdot b_{window} = 1.8\,m$ Effective length of stabilizing wall 8

Panels are made of OSB sheet, 12 mm thick on both sides

$b_{panel} := 1.2m$ Width of panel

$h_{panel} := 2.9m$ Height of panel (storey heigth and floor thickness)

Number of panels needed:

$$n_{panel} := \frac{L_{eff8}}{b_{panel}} = 1.5$$

2 panels are needed

According to EC5 9.2.4.2 (2), if the width of the panel is less than h/4 the contribution to transmitting shear forces will be neglected. This means 2 panels are used.

$n_{panel} := 2$

Wind load has not been taken into account since it is a favourable effect.

$$F_{anch8} := \frac{g_{stab.wall.8} \cdot 6 \cdot \frac{L_{s8}}{2} - H_1 \cdot \frac{h_{storey}}{2} - \gamma_G \cdot \left[R_{8h} \cdot \left(B + \frac{h_{tot} - B}{2} \right) + R_{8b} \cdot \frac{\left(B - 0.5 \cdot h_{storey} \right)}{2} \right] \ldots \\ + -w_G \cdot x_G \cdot L_{s8} \cdot \left(L_{s8} - \frac{x_G}{2} \right) - w_H \cdot \left(L_{s8} - x_G \right) \cdot L_{s8} \cdot \left(\frac{L_{s8} - x_G}{2} \right) - H_{d1} \cdot 2.9m}{b_{panel} \cdot n_{panel}} = -334.788 \cdot k$$

Wind loads (shear walls, wind coming on short facade)

$L_{s1} = 9.997\,m$ — Length of stabilizing wall R1

$L_{s13} = 3\,m$ — Length of stabilizing wall R13

$l_{ef.R1} = 10\,m$ — Effective width of wall resisted by stabilizing wall R1

$l_{ef.R13} = 3\,m$ — Effective width of wall resisted by stabilizing wall R13

$g_{stab.wall.1} := G_{k.shaft} \cdot h_{storey} \cdot L_{s1} = 28.815 \cdot kN$ — Self-weight of wall 1 on one storey

$g_{stab.wall.13} := 0.8 \frac{kN}{2} \cdot h_{storey} \cdot L_{s13} = 5.76 \cdot kN$ — Self-weight of wall 13 on one storey

$q_{tot.wall} := 1.378 \frac{kN}{m^2}$

$$q_1 := \frac{q_{tot.wall} \cdot (h_{tot} - 0.5 \cdot h_{storey}) \cdot l_{ef.R1}}{2} = 130.221 \cdot kN$$

Wind load resisted by bottom wall 1. (Half of the total wind load is resisted by wall 1, the other half by wall 2.)

$R_1 := q_1 = 130.221 \cdot kN$ — Total wind load resisted by wall 1

$$q_{13} := \frac{q_{tot.wall} \cdot (h_{tot} - 0.5 \cdot h_{storey}) \cdot l_{ef.R13}}{2} = 39.066 \cdot kN$$

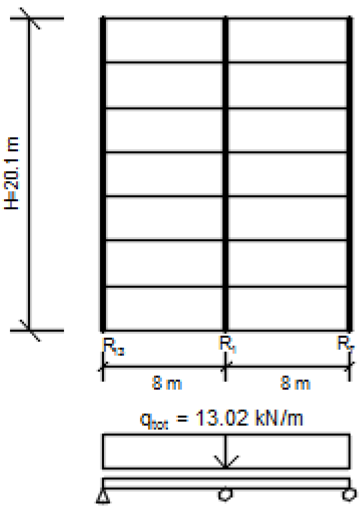
Wind load resisted by bottom wall 13. (Half of the total wind load is resisted by wall 13, the other half by wall 11.)

$R_{13} := q_{13} = 39.066 \cdot kN$ — Total wind load resisted by wall 13

$H_1 := 6.193 kN$ — Unintended inclination on the bottom floor

$$q_{tot} := \frac{q_{tot.wall} \cdot (h_{tot} - 0.5 \cdot h_{storey})}{2} = 13.022 \cdot \frac{kN}{m}$$

Distributed wind load in kN/m



R_i = Stabilizing wall

Figure D8: Elevation of short facade.

Shear wall 1

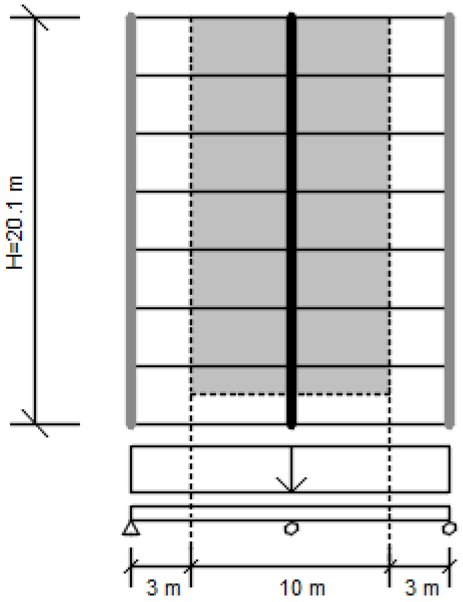


Figure D9: Section of the short facade with shear walls.

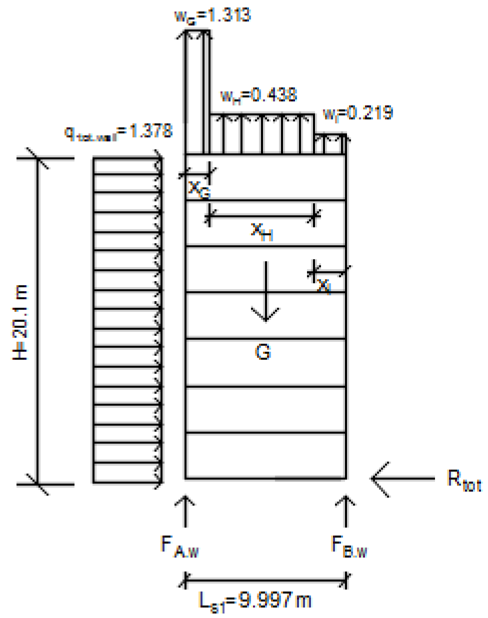


Figure D10: Shear wall subjected to wind load on the short facade.

$$w_G := 1.313 \frac{kN}{m^2} \qquad x_G := 1.6m$$

$$w_H := 0.438 \frac{kN}{m^2} \qquad x_H := 6.4m$$

$$w_I := 0.219 \frac{kN}{m^2} \qquad x_I := L_{s1} - x_G - x_H = 1.997\,m$$

Moment around B

There are 6 stabilizing walls above the one on the first floor.

$$F_{A.w1} := \frac{g_{stab.wall.1} \cdot 6 \cdot \frac{L_{s1}}{2} - H_1 \cdot \frac{h_{storey}}{2} - \gamma_G \cdot \left(R_1 \cdot \frac{h_{tot} - 0.5 \cdot h_{storey}}{2}\right) \ldots + -w_G \cdot x_G \cdot L_{s1} \cdot \left(L_{s1} - \frac{x_G}{2}\right) - w_H \cdot x_H \cdot L_{s1} \cdot \left(L_{s1} - x_G - \frac{x_H}{2}\right) - w_I \cdot x_I \cdot L_{s1} \cdot \left(\frac{x_I}{2}\right)}{L_{s1}} = -133.267 \cdot kN$$

$$F_{B.w1} := g_{stab.wall.1} \cdot 6 + \left|F_{A.w1}\right| - w_G \cdot x_G \cdot L_{s1} - w_H \cdot x_H \cdot L_{s1} - w_I \cdot x_I \cdot L_{s1} = 252.762 \cdot kN$$

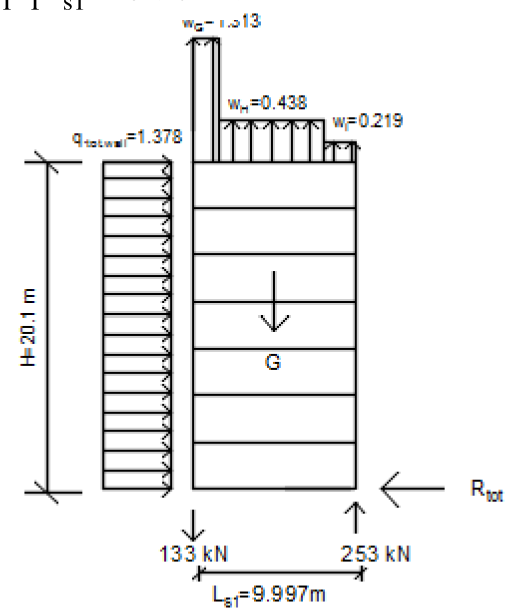


Figure D11: Tensile and compression forces for shear wall 1.

Shear wall 13 (exterior studs with panels)

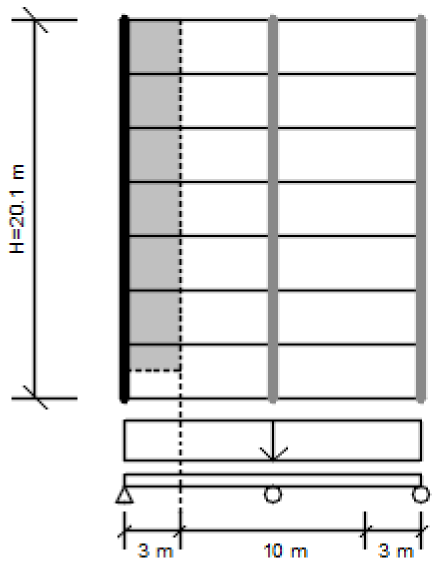


Figure D12: Section of the short facade with shear walls.

Total horizontal wind load acting on shear wall at the bottom floor

$V_{d.shear} := R_{13} = 39.066 \cdot kN$

Unintended inclination at the bottom floor

$H_1 = 6.193 \cdot kN$

Total racking force

$H_{rack} := V_{d.shear} + H_1 = 45.259 \cdot kN$

$L_{s13} = 3\,m$ — Length of stabilizing wall 13

$b_{window} := 1.2m$ — Width of window

$n := 1$ — Number of windows

$L_{eff} := L_{s13} - n \cdot b_{window} = 1.8\,m$ — Effective length of stabilizing wall 1

Panels are made of OSB sheet, 12 mm thick on both sides

$b_{panel} := 1.2m$ — Width of panel

$h_{panel} := 2.9m$ — Height of panel (storey heigth and floor thickness)

Number of panels needed:

$n_{panel} := \dfrac{L_{eff}}{b_{panel}} = 1.5$ — 2 panels are needed

According to EC5 9.2.4.2 (2), if the width of the panel is less than h/4 the contribution to transmitting shear forces will be neglected. This means 2 panels are used.

$n_{panel} := 2$

$$F_{anch13} := \frac{g_{stab.wall.13} \cdot 6 \cdot \dfrac{L_{s13}}{2} - H_1 \cdot \dfrac{h_{storey}}{2} - \gamma_G \cdot \left[R_{13} \cdot \left[0.5 \cdot h_{storey} + \dfrac{\left(h_{tot} - 0.5 \cdot h_{storey}\right)}{2} \right] \right] \ldots \\ + -w_G \cdot x_G \cdot L_{s13} \cdot \left(L_{s13} - \dfrac{x_G}{2} \right) - w_H \cdot \left(L_{s13} - x_G\right) \cdot L_{s13} \cdot \left(L_{s13} - x_G\right)}{b_{panel} \cdot n_{panel}} = -248.382 \cdot kN$$

APPENDIX E - Anchorage with glued-in rods

$d := 24mm$ Diameter of rod

$l_a := 0.350$ Anchorage length

BONDED LINE FAILURE

$d := 24mm$ Diameter of rod

$l_a := 0.350m$ Anchorage length

$f_{k1.k} := 5.25\frac{N}{mm^2} - 0.005\frac{N}{mm^3} \cdot l_a = 3.5 \cdot MPa$ Characteristic value of bond line strength

$k_{mod} := 0.8$ Medium term action, solid timber and glulam

$\gamma_M := 1.3$ Partial factor, connections

$f_{k1.d} := k_{mod} \cdot \frac{f_{k1.k}}{\gamma_M} = 2.154 \cdot MPa$ Design value of bond line strength

$R_{ax.d} := \pi \cdot d \cdot l_a \cdot f_{k1.d} = 56.839 \cdot kN$ Design anchorage capacity

$F_{A.w5} = -311.087 \cdot kN$ Tensile force that needs to be resisted

$n_{rod} := \frac{|F_{A.w5}|}{R_{ax.d}} = 5.473$ Number of rods needed

CHECK YIELDING

$f_y := 355MPa$

$\sigma_y := \frac{f_y}{\gamma_M} = 273.077 \cdot MPa$ Yielding stress

$\sigma_y \geq \frac{F_{A.w5}}{A_{needed}}$

$A_{needed} := \frac{R_{ax.d}}{\sigma_y} = 2.081 \times 10^{-4} m^2$ Largest stress one rod can take, $R_{ax.d}$

$A_{needed} = \pi \cdot r^2$

$r := \sqrt{\frac{A_{needed}}{\pi}} = 8.14 \cdot mm$ Radius required to not yield

$d_{rod} := 2 \cdot r = 16.279 \cdot mm$ OK! Diameter needed to not yield

CHECK TENSILE FAILURE

$$\sigma = \frac{F_{A.w5}}{A} < \frac{f_y}{\gamma_M}$$

$$A := \pi \cdot \left(\frac{d}{2}\right)^2 \cdot n_{rod} = 2.476 \times 10^3 \cdot mm^2$$

$$\frac{\left|F_{A.w5}\right|}{A} = 125.641 \cdot MPa$$

$$\frac{F_{A.w5}}{A} < \frac{f_y}{\gamma_M} = 1$$ OK!

PULL-OUT STRENGTH

$d := 24mm$ — Diameter of rod

$l_a := 0.350 \cdot m$ — Anchorage length

$f_{v3} := 3.9 \frac{N}{mm^2} - 0.05 \frac{N}{mm^2} \frac{l_a}{d} = 3.171 \cdot MPa$ — Strength parameter (derived empirically by Riberholt)

$F_{tk} := \pi \cdot d \cdot l_a \cdot f_{v3} = 83.676 \cdot kN$ — Charcteristic pull-out strength of the rod

$k_{mod} := 0.8$ — Medium term action, solid timber and glulam

$\gamma_M := 1.3$ — Partial factor, connections

$F_{td} := \frac{k_{mod} \cdot F_{tk}}{\gamma_M} = 51.493 \cdot kN$ — Design pull-out strength of the rod

$F_{A.w5} = -311.087 \cdot kN$ — Tensile force that needs to be resisted

$$n_{rods} := \frac{\left|F_{A.w5}\right|}{F_{td}} = 6.041$$

7 rods are needed for anchoring the uplift force.

$n_{rod} := 7$

To avoid group effect and splitting failure, distance between rods should be larger or equal to 4d. Edge distance should be at least 2.5d.

$4 \cdot d = 96 \cdot mm$

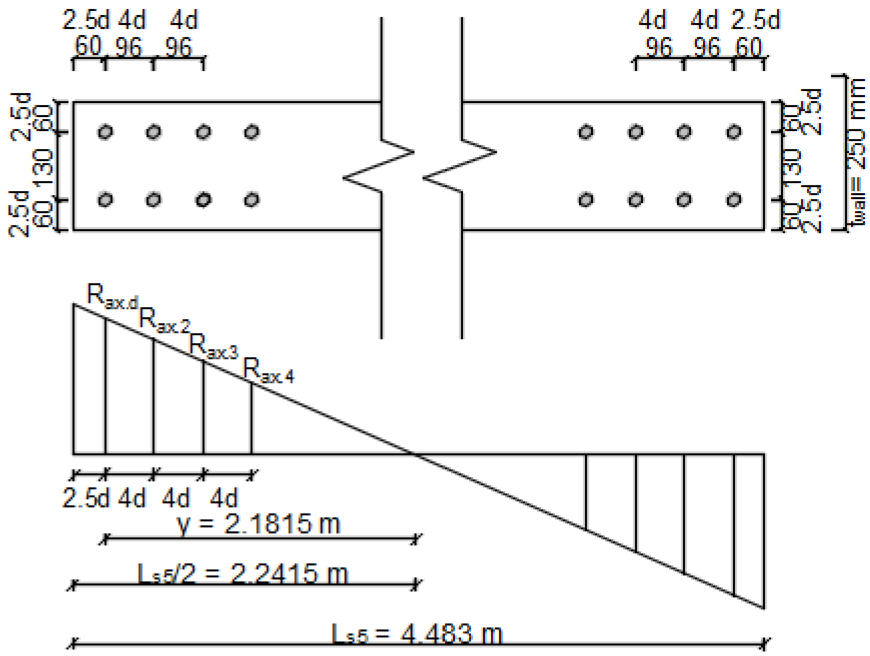


Figure E1: Section of wall from above with glued-in rods placement and shear force diagram - Linear force distribution

$y := \frac{L_{s5}}{2} - 2.5 \cdot d = 2.181\,m$ $\qquad R_{ax.d} = 56.839 \cdot kN$ $\qquad$ Smallest anchorage capacity (failure in bond)

Linear relationship

$\frac{R_{ax.d}}{y} = \frac{R_{ax.2}}{y - 4 \cdot d}$ $\qquad R_{ax.2} := (y - 4 \cdot d) \cdot \frac{R_{ax.d}}{y} = 54.337 \cdot kN$

$\frac{R_{ax.d}}{y} = \frac{R_{ax.3}}{y - 2 \cdot 4 \cdot d}$ $\qquad R_{ax.3} := (y - 2 \cdot 4 \cdot d) \cdot \frac{R_{ax.d}}{y} = 51.836 \cdot kN$

$\frac{R_{ax.d}}{y} = \frac{R_{ax.4}}{y - 3 \cdot 4 \cdot d}$ $\qquad R_{ax.4} := (y - 3 \cdot 4 \cdot d) \cdot \frac{R_{ax.d}}{y} = 49.335 \cdot kN$

$n_{row} := 2$

$R_{res} := n_{row} \left(R_{ax.d} + R_{ax.2} + R_{ax.3} + R_{ax.4} \right) = 424.694 \cdot kN$ $\qquad$ Total resistance with glued-in rods

$R_{res} > \left| F_{A.w5} \right| = 1$ $\qquad$ OK

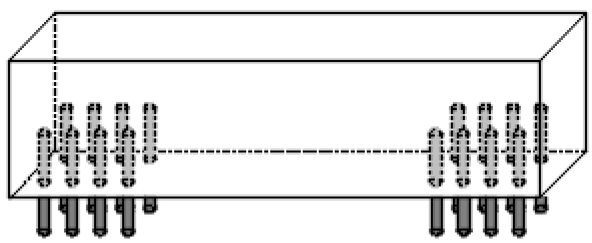


Figure E2: 3D sketch of cut wall with glued-in rods.

APPENDIX F: Anchorage with slotted-in steel plates

Calculating slotted-in steel plates resistance as anchorage for one steel plate

Timber density

$$\rho_k := 490 \frac{kg}{m^3}$$

Diameter of dowel

$$d := 20mm$$

Thickness of the timber member on both sides of the steel plate

$$t_1 := 100mm$$

Characteristic tensile strength

$$f_{u.k} := 600 \frac{N}{mm^2}$$

Characteristic fastener yield moment

$$M_{y.Rk} := 0.3 \cdot f_{u.k} \cdot d^{2.6} \cdot m^{0.4} = 6.886 \times 10^6 \cdot N \cdot mm$$

Characteristic embedment strength parallel to the grain

$$f_{h.1.k} := 0.082 \cdot \frac{m^2}{kg} \cdot \frac{N}{mm^2} \cdot (1m - 0.01 \cdot d \cdot 1000) \cdot \rho_k = 32.144 \cdot \frac{N}{mm^2}$$

Expression for the resistance of slotted-in steel plates

$$F_{v.Rk1} := f_{h.1.k} \cdot t_1 \cdot d = 64.288 \cdot kN$$

$$F_{v.Rk2} := f_{h.1.k} \cdot t_1 \cdot d \cdot \left(\sqrt{2 + \frac{4 \cdot M_{y.Rk}}{f_{h.1.k} \cdot d \cdot t_1^2}} - 1 \right) = 96.873 \cdot kN$$

$$F_{v.Rk3} := \sqrt{2} \cdot \sqrt{2 \cdot M_{y.Rk} \cdot f_{h.1.k} \cdot d} = 133.067 \cdot kN$$

$$F_{v.Rk} := \min\left(F_{v.Rk1}, F_{v.Rk2}, F_{v.Rk3}\right) = 64.288 \cdot kN$$

Design load per dowel

$k_{mod} := 0.8$

Solid timber, medium term action, serviece class 2

$\gamma_m := 1.3$

Partial factor - connections

$$F_{v.Rd} := k_{mod} \cdot \frac{F_{v.Rk}}{\gamma_m} = 39.562 \cdot kN$$

Force acting on the anchorage for a 7 storey building

$F_{A.w5} := 311.087kN$

Number of dowels needed

$$n := \frac{F_{A.w5}}{F_{v.Rd}} = 7.863$$

Number of dowels used are: 8

The number of dowels needed (8) are multiplied with the force a single dowel can resist which results in the total anchorage force.

$$F_{v.Rd.tot7} := 8 \cdot F_{v.Rd} = 316.495 \cdot kN$$

$F_{v.Rd.tot7} \geq F_{A.w5} = 1$ OK!

Minimum spacing or edge/end distance for an anchorage of a 7 storey building

$\alpha_1 := 0deg$

$\alpha_2 := 90deg$

Minimum spacing and edge and end distances for dowels

Minimum spacing between dowels parallel to the grain

$$a_{1.7} := \left(3 + 2 \cdot \left|\cos(\alpha_1)\right|\right) \cdot d = 0.1\,m$$

Minimum spacing between dowels perpendicular to the grain

$$a_{2.7} := 3 \cdot d = 0.06\,m$$

Minimum spacing between dowels and edge parallel and perpendicular to the grain

$$a_{3.t.7} := \max(7 \cdot d, 0.08m) = 0.14\,m$$

$$a_{4.t.7} := \max\left[\left(2 + 2 \cdot \sin(\alpha_2)\right) \cdot d, 3 \cdot d\right] = 0.08\,m$$

Checking the steel plate resistance

$\gamma_M := 1$

$t_7 := 8mm$ Thickness of the steel plate

$b_7 := a_{2.7} + 2 \cdot a_{4.t.7} = 0.22\,m$ Lenght of the steel plate

$\frac{F_{v.Rd.tot7}}{b_7 \cdot t_7} \leq \frac{f_{u.k}}{\gamma_M} = 1$ OK!

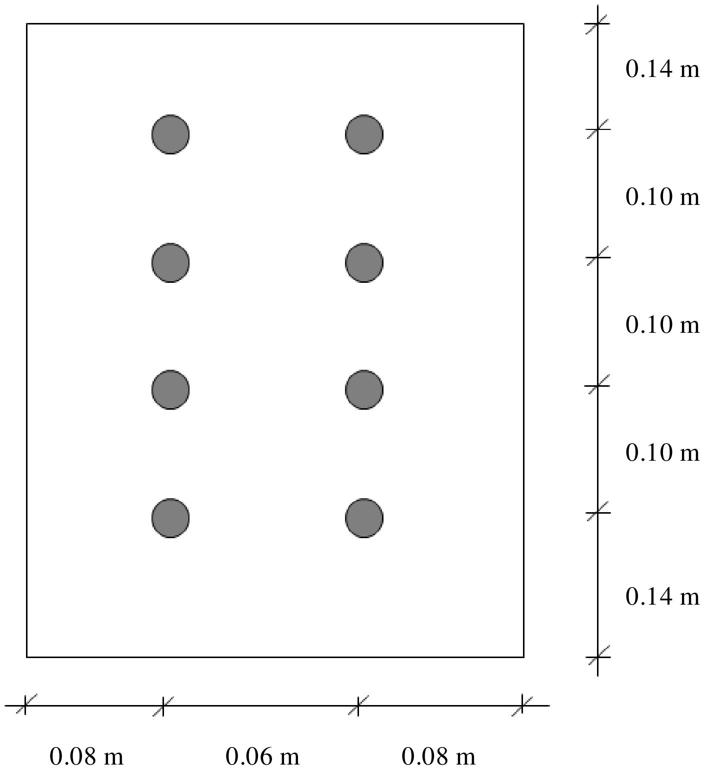


Figure F1: Design of slotted-in steel plates for a seven storey building.

Calculating slotted-in steel plates resistance as anchorage for two steel plates

Thickness of the timber member between the edge and steel plate

$t_1 := 30\text{mm}$

Thickness of the timber member between two steel plates

$t_2 := 60\text{mm}$

Expression for the resistance of 2 steel-in plates

Number of plates

$n_{s.2} := 2$

$$C_{1.2} := 2 \cdot t_1 + \left(n_{s.2} - 1\right) \cdot t_2 = 0.12\,\text{m}$$

$$C_{2.2} := 2 \cdot t_1 \cdot \left[\sqrt{2 + \frac{2}{3} \cdot \frac{f_{u.k}}{f_{h.1.k}} \cdot \left(\frac{d}{t_1}\right)^2} - 1\right] + \left(n_{s.2} - 1\right) \cdot t_2 = 0.165\,\text{m}$$

$$C_{3.2} := d \cdot \sqrt{\frac{8}{3} \cdot \frac{f_{u.k}}{f_{h.1.k}}} + \left(n_{s.2} - 1\right) \cdot t_2 = 0.201\,\text{m}$$

$$C_{4.2} := 2 \cdot t_1 + \left(n_{s.2} - 1\right) \cdot d \cdot \sqrt{\frac{8}{3} \cdot \frac{f_{u.k}}{f_{h.1.k}}} = 0.201\,\text{m}$$

$$C_{5.2} := 2 \cdot t_1 \cdot \left[\sqrt{2 + \frac{2}{3} \cdot \frac{f_{u.k}}{f_{h.1.k}} \cdot \left(\frac{d}{t_1}\right)^2} - 1\right] + \left(n_{s.2} - 1\right) \cdot d \cdot \sqrt{\frac{8}{3} \cdot \frac{f_{u.k}}{f_{h.1.k}}} = 0.246\,\text{m}$$

$$C_{6.2} := n_{s.2} \cdot d \cdot \sqrt{\frac{8}{3} \cdot \frac{f_{u.k}}{f_{h.1.k}}} = 0.282\,\text{m}$$

$$C_2 := \min\left(C_{1.2}, C_{2.2}, C_{3.2}, C_{4.2}, C_{5.2}, C_{6.2}\right) = 0.12\,\text{m}$$

Shear strenght of the dowel with two slotted-in steel plates

$$F_{v.Rk2} := C_2 \cdot f_{h.1.k} \cdot d = 77.146 \cdot \text{kN}$$

Design load per dowel

$k_{mod} := 0.8$ Solid timber, medium term action, serviece class 2

$\gamma_m := 1.3$ Partial factor - connections

$$F_{v.Rd2} := k_{mod} \cdot \frac{F_{v.Rk2}}{\gamma_m} = 47.474 \cdot kN$$

Force acting on the anchorage for a <u>7 storey</u> building

$F_{A.w5} := 311.087kN$

Number of dowels needed

$$n := \frac{F_{A.w5}}{F_{v.Rd2}} = 6.553$$

Number of dowels used are: 8

The number of dowels needed (8) are multiplied with the force a single dowel can resist which results in the total anchorage force.

$$F_{v.Rd.tot2} := 8 \cdot F_{v.Rd2} = 379.794 \cdot kN$$

$F_{v.Rd.tot2} \geq F_{A.w5} = 1$ OK!

Checking the steel plate resistance for two slotted-in steel plates

$\gamma_M := 1$

$t_7 := 8mm$ Thickness of the steel plate

$b_7 := a_{2.7} + 2 \cdot a_{4.t.7} = 0.22\,m$ Lenght of the steel plate

$\frac{F_{v.Rd.tot2}}{2b_7 \cdot t_7} \leq \frac{f_{u.k}}{\gamma_M} = 1$ OK!

Expression for the resistance of 3 steel-in plates

Number of plates

$$n_{s.3} := 3$$

$$C_{1.3} := 2 \cdot t_1 + \left(n_{s.3} - 1\right) \cdot t_2 = 0.18\,\mathrm{m}$$

$$C_{2.3} := 2 \cdot t_1 \cdot \left[\sqrt{2 + \frac{2}{3} \cdot \frac{f_{u.k}}{f_{h.1.k}} \cdot \left(\frac{d}{t_1}\right)^2} - 1\right] + \left(n_{s.3} - 1\right) \cdot t_2 = 0.225\,\mathrm{m}$$

$$C_{3.3} := d \cdot \sqrt{\frac{8}{3} \cdot \frac{f_{u.k}}{f_{h.1.k}}} + \left(n_{s.3} - 1\right) \cdot t_2 = 0.261\,\mathrm{m}$$

$$C_{4.3} := 2 \cdot t_1 + \left(n_{s.3} - 1\right) \cdot d \cdot \sqrt{\frac{8}{3} \cdot \frac{f_{u.k}}{f_{h.1.k}}} = 0.342\,\mathrm{m}$$

$$C_{5.3} := 2 \cdot t_1 \cdot \left[\sqrt{2 + \frac{2}{3} \cdot \frac{f_{u.k}}{f_{h.1.k}} \cdot \left(\frac{d}{t_1}\right)^2} - 1\right] + \left(n_{s.3} - 1\right) \cdot d \cdot \sqrt{\frac{8}{3} \cdot \frac{f_{u.k}}{f_{h.1.k}}} = 0.387\,\mathrm{m}$$

$$C_{6.3} := n_{s.3} \cdot d \cdot \sqrt{\frac{8}{3} \cdot \frac{f_{u.k}}{f_{h.1.k}}} = 0.423\,\mathrm{m}$$

$$C_3 := \min\left(C_{1.3}, C_{2.3}, C_{3.3}, C_{4.3}, C_{5.3}, C_{6.3}\right) = 0.18\,\mathrm{m}$$

Shear strenght of the dowel with three slotted-in steel plates

$$F_{v.Rk3} := C_3 \cdot f_{h.1.k} \cdot d = 115.718 \cdot \mathrm{kN}$$

Design load per dowel

$$k_{mod} := 0.8$$

Solid timber, medium term action, serviece class 2

$$\gamma_m := 1.3$$

Partial factor - connections

$$F_{v.Rd3} := k_{mod} \cdot \frac{F_{v.Rk3}}{\gamma_m} = 71.211 \cdot \mathrm{kN}$$

Force acting on the anchorage for a 7 storey building

$F_{A.w5} := 311.087kN$

Number of dowels needed

$n := \frac{F_{A.w5}}{F_{v.Rd3}} = 4.369$ Number of dowels used are: 6

The number of dowels needed (8) are multiplied with the force a single dowel can resist which results in the total anchorage force.

$F_{v.Rd.tot3} := 6 \cdot F_{v.Rd3} = 427.268 \cdot kN$

$F_{v.Rd.tot3} \geq F_{A.w5} = 1$ OK!

The minimum spacing and edge and end distances for dowels with two or three steel plates are the same as for single slotted-in steel plates

Checking the steel plate resistance for three slotted-in steel plates

$\gamma_M := 1$

$t_7 := 8mm$ Thickness of the steel plate

$b_7 := a_{2.7} + 2 \cdot a_{4.t.7} = 0.22\,m$ Lenght of the steel plate

$\frac{F_{v.Rd.tot3}}{3b_7 \cdot t_7} \leq \frac{f_{u.k}}{\gamma_M} = 1$ OK!

APPENDIX G: Hold-down connector

The chosen connector is a hold-down connector from Rothoblaas. TYP WHT 620. S 355 quality steel according to EN 10025-2:2004 with fy,k ≥ 350 N/mm2 FeZn 12c galvanic zinc with minimum thickness 12 μm. The height of the connector is 620 mm and the width is 80 mm. The washer plate at the bottom has a width of 80 mm and is anchored with a 20mm in diameter bolt (hole is 21 mm in diameter). See Figure for dimensions.

The hold-downs are fastened with total nailing;
anchor nails: $\phi 4.0 \times 60$
special screws: $\phi 5.0 \times 50$

See Tables below for values.

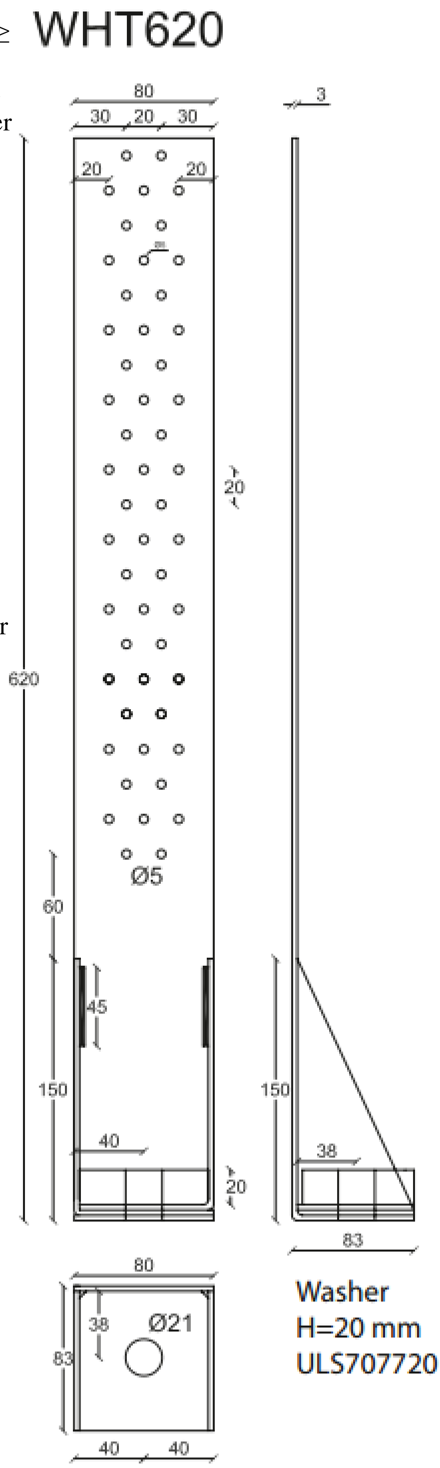


Figure Gl : Detail of hold-down (Rothoblass).

Vertical component junction on wood

$$R_{d.WHT} = \min\left(\frac{R_{k.wood} \cdot k_{mod}}{\gamma_{M.conn}}, \frac{R_{k.steel}}{\gamma_{M.steel}}\right)$$ Design value for the joint

$n_{conn} := 52$ Numbers of connectors

$R_{k.conn} := 1.93kN$ Resistance of a single connector

$R_{k.wood} := n_{conn} \cdot R_{k.conn} = 100.36 \cdot kN$ Characteristic tensile strength on wood side

$R_{k.steel} := 85.2kN$ Characteristic tensile strength on steel side

$k_{mod} := 0.8$

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

$\gamma_{M.steel} := 1.15$

$$R_{d.WHT} := \min\left(\frac{R_{k.wood} \cdot k_{mod}}{\gamma_{M.conn}}, \frac{R_{k.steel}}{\gamma_{M.steel}}\right) = 61.76 \cdot kN$$

$F_{anch8} := 334kN$ Tensile force that needs to be anchored, wall 8

$$n_{holddowns8} := \frac{F_{anch8}}{R_{d.WHT}} = 5.408$$ Number of hold-downs needed

$n_{needed} := 6$ Number of hold-downs used

6 hold-downs are needed to anchor shear wall 8 (corner stud wall that is subjected to the largest tensile force).

$F_{A.w5} := 311kN$ Tensile force that needs to be anchored, wall 5

$$n_{holddowns5} := \frac{F_{A.w5}}{R_{d.WHT}} = 5.036$$ Number of hold-downs needed

6 hold-downs are needed to anchor shear wall 5 (solid timber wall that is subjected to the largest tensile force).

Table G1: Characteristic tensile strength on the wood side and the steel side (Rothoblaas).

WHT - TOTAL NAILING			Characteristic tensile strength			
TYP WHT	Ø 5 Holes Fastening (connectors)		R_k wood side		R_k steel side	
			n_{conn}	$R_{k,\,wood}$	Washer	$R_{k,\,steel}$
	Anker Nails	Special Screws	[pcs.]	[kN]		[KN]
340	ø 4,0 x 40	ø 5,0 x 40	20	31,4	-	42,0
	ø 4,0 x 60	ø 5,0 x 50		38,6		
440	ø 4,0 x 40	ø 5,0 x 40	30	47,1	* H	63,4
	ø 4,0 x 60	ø 5,0 x 50		57,9	10 mm	
540	ø 4,0 x 40	ø 5,0 x 40	42	65,9	* H	63,4
	ø 4,0 x 60	ø 5,0 x 50		81,1	10 mm	
620	ø 4,0 x 40	ø 5,0 x 40	52	81,6	** H	85,2
	ø 4,0 x 60	ø 5,0 x 50		100,4	20 mm	

* Washer ULS505610 ** Washer ULS707720

Junction on concrete

Threaded bar, Steel class 5.8

$\phi_{bolt} := 20mm$ Diameter of anchor bolt

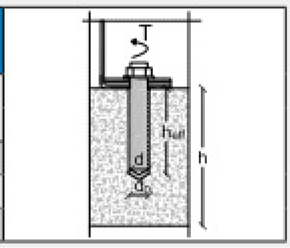
$N_{k.extract} := 122.0kN$ Characteristic resistance to extraction

$\gamma_M := 1.5$ Partial value

$N_{d.extract} := \dfrac{N_{k.extract}}{\gamma_M} = 81.333 \cdot kN$ Design resistance to extraction

Table G2: Characteristic extraction resistance (Rothoblaas).

RESIN CHEMICAL ANCHOR				
Threaded Bar		1 h_{eff}	$N_{k,\,extract}$	γ_m
Ø [mm]	Steel class	[mm]	[kN]	
10	5.8	90	22,6	1,8
16	5.8	160	78,0	1,5
20	5.8	200	122,0	1,5



APPENDIX H: Anchorage with threaded rods through stud walls

Wall 8 resists the larger load compared with wall 13

$F_{anch8} = -334.788 \cdot kN$

Rods are made of steel 355 MPa

$f_y := 355MPa$ Yield limit of steel

$A_{steel} := \frac{|F_{anch8}|}{f_y} = 9.431 \times 10^{-4} m^2$ Needed area

$d_{rod} := 24mm$

$A_{rod} := \frac{\pi \cdot d_{rod}^2}{4} = 4.524 \times 10^{-4} m^2$

$n_{rods} := \frac{A_{steel}}{A_{rod}} = 2.085$ Number of rods needed

Stud wall with steel rods of Φ24, 3 rods are needed at both edges of each panel.

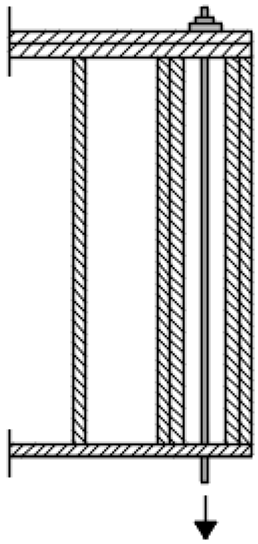


Figure H1 : Threaded rod through stud wall.