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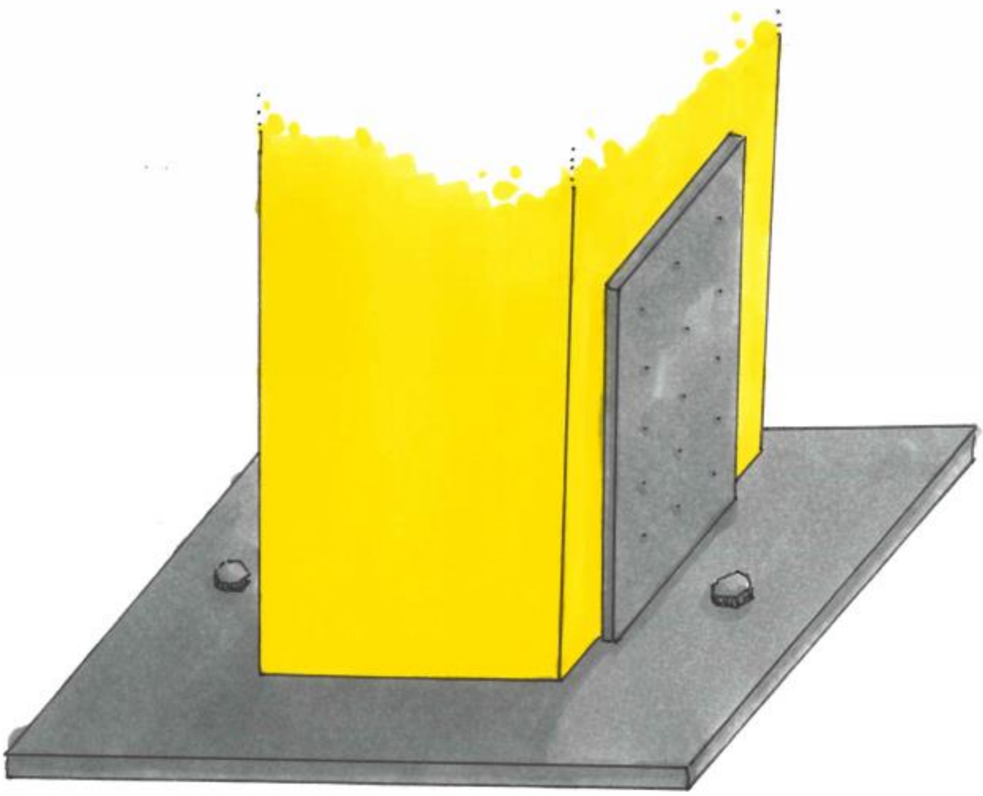

Investigation of floor addition in timber on an existing multi-activity building

Master's Thesis in the Master's Programme Structural Engineering and Building Technology

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Steel and Timber Structures
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Cover:
Design of the suggested connection consisting of nailed steel plates and a base plate with expanders. Illustration made by Elin Tjäder.

Department of Civil and Environmental Engineering Göteborg, Sweden, 2016

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ABSTRACT

Investigations show that more and more people moves from the countryside to the Swedish cities. Due to this urbanization combined with a continuously growing population and ongoing immigration, the need for residences constantly increase. One solution to solve the lack of residences is adding floors on existing buildings. When adding floors, it is desirable to minimize the impact on the existing structural system, which can be achieved by using a lightweight material.

In Sweden, timber has been used as a construction material for a very long time. Timber has the advantage to possess a high strength in relation to a low self-weight. Therefore, timber is a favorable material to use when adding floors. The aim of this study was to find the most suitable concept to perform an addition of floors in timber on the multi-activity building Strömshuset, located in the central part of Gothenburg.

The project initiated with a literature study. In the next step, necessary information about Strömshuset was gathered. Further, an evaluation of four predefined concepts of common building methods was performed according to relevant criteria. Finally, a principle design of the identified concept was made by hand calculations, to investigate what might limit the number of added floors.

The most suitable concept for adding floors on Strömshuset turned out to be a beam-post system built on site. From the design calculations, it could be stated that the horizontal loads are limiting the number of floors that can be added. The reason was that the shear walls had not enough capacity to resist the rotational moment of the building. Therefore, more shear walls needs to be added.

Key words: timber, lightweight material, adding floors, Strömshuset, evaluation, principle design, beam-post system, horizontal loads, shear walls

Undersökning av påbyggnation i trä på en befintlig multi-aktivitets byggnad
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SAMMANFATTNING

Undersökningar visar att allt fler människor väljer att flytta från landsbyggden och bosätta sig i de svenska städerna. Som följd av denna urbanisering, kombinerat med en konstant ökande population och pågående invandring, ökar behovet av bostäder konstant. En lösning på problemet med bostadsbrist är att utföra påbyggnationer på befintliga byggnader. Vid en påbyggnation är det fördelaktigt att minimera påverkan på det befintliga bärande systemet, vilket kan uppnås genom att använda ett lättviktsmaterial.

I Sverige har trä använts som byggnadsmaterial väldigt länge. Trä har fördelen att inneha hög hållfasthet i relation till en låg egenvikt. Detta medför att trä är ett fördelaktigt material att använda vid en påbyggnation. Syftet med denna studie var att hitta det mest lämpliga konceptet för att utför en påbyggnation i trä på den befintliga multi-aktivitetsbyggnaden Strömshuset, belägen i de centrala delarna av Göteborg.

Projektet inleddes med en litteraturstudie. I nästa steg samlades nödvändig information om Strömshuset in. Vidare utvärderades fyra förbestämmda koncept bestående av vanliga byggnadsmetoder i förhållande till fem relevanta kriterier. Slutligen utfördes en preliminär design av det identifierade konceptet för att undersöka vad som begränsar antalet våningar som kan byggas på.

Det mest lämpliga konceptet för att utföra en påbyggnation på Strömshuset visade sig vara ett platsbyggt pelar-balksystem. Från beräkningarna kunde det konstateras att de horisontella lasterna begränsar antalet våningar som kan byggas på. Anledningen var att skjuvväggarna inte hade tillräckligt med kapacitet för att motstå det roterande moment som byggnaden utsätts för. Därför behöver mer skjuvväggar adderas till byggnaden.

Nyckelord: trä, lättviktsmaterial, påbyggnation, Strömshuset, utvärdering, principiell design, balk-pelar system, horisontella laster, skjuvväggar

Contents

Preface

In this master thesis, an investigation of floor addition in timber on an existing multi-activity building has been performed. The study has been carried out between February and June 2016 and is collaboration between the consultant company COWI and the Division of Structural Engineering, Steel and Timber structures, Chalmers University of Technology, Sweden.

We would like to thank our supervisors at COWI, Aino Granberg, Oscar Pagrotsky and Thomas Hallgren for the support and supervision during the project. Also, we would like to thank all employees at COWI for making us feel welcome and having a pleasant time.

Furthermore, we would like to thank the company Moelven in Sandsjöfors, Sweden, for the educational study visit and taking time to answer our questions.

Also, an appreciation is given to our friend Elin Tjäder for helping us with the illustration of the front page of this master thesis.

Finally, we would like to thank our supervisor and examiner Reza Haghani, Associate Professor at the Division of Structural Engineering at Chalmers University of Technology, for his support and appreciated inputs throughout the project.

Göteborg June 2016

Cornelia Andersson, Marie Eriksson

Notations

Roman upper case letters

A	Cross-sectional area of a member
C	Circumference of the building
E	Elastic modulus
E_{mean}	Mean value of the elastic modulus for timber
$E_{0.05}$	Elastic modulus parallel to the grain
E_{fi}	Elastic modulus due to fire
EI	Bending stiffness
F	Equivalent horizontal force due to wind loads
F_{ax}	Withdrawal capacity
F_{bs}	Design capacity due to block tearing
F_{Rd}	Design capacity per nail and shear plane
F_x	Resulting force caused by moment in the design of connections
F_y	Force between the steel plate and the column
H	Height of the building
H_{column}	Height of the column
H_d	Equivalent horizontal force due to unintended inclination
I	Second moment of inertia
L	Length of the building
L_c	Buckling length of column
M_{Ed}	Design moment
M_{Rd}	Resisting moment
M_y	Yield moment
N_{cr}	Capacity due to axial forces
N_{Rd}	Axial capacity of the steel plate
P_{point}	Static point load from human response
Q	Total vertical load
Q_{fire}	Total vertical load according to fire
S	Snow load
V_d	Vertical load from a specific storey when determining the equivalent horizontal force due to unintended inclination

V_{Ed}	Maximum shear force
W	Sectional modulus

Roman lower case letters

a_1, a_2, a_3	Distance between the nails in a connection
b_{eff}	Effective width
$d_{charn,n}$	Charring depth for unprotected timber during fire
f_1	Velocity response
$f_{c.0.k}$	Characteristic compression strength parallel to the grain
$f_{c.0.d}$	Design compression strength parallel to the grain
$f_{c.90.k}$	Characteristic compression strength perpendicular to the grain
$f_{c.90.d}$	Design compression strength perpendicular to the grain
$f_{m.g.k}$	Characteristic bending parallel to the grain
$f_{m.g.d}$	Design value for bending parallel to the grain
$f_{pv.k}, f_{v.g.k}$	Characteristic value for panel shear
$f_{pv.d}$	Design value of panel shear
$f_{t.o.k}$	Characteristic tension strength parallel to the grain
$f_{t.o.d}$	Design tension strength parallel to the grain
f_u	Tensile strength for steel
f_{uk}	Ultimate strength of steel member
$f_{v.g.k}$	Characteristic shear strength
$f_{v.g.d}$	Design shear strength
f_{yk}, f_y	Yield strength for steel
g	Gravitational constant
g_k	Characteristic value for permanent loads
h	Height of a cross-section
i	Radius of gyration
k_c	Instability factor
k_{cr}	Modification factor due to influence of cracks
k_{def}	Deformation modification factor
k_{fi}	Modification factor in design due to fire
k_h	Effect of member size
k_m	Factor that takes inhomogeneity and redistribution of stresses into account

k_{mod}	Conversion factor for timber
$k_{\mathrm{mod},fi}$	Conversion factor for timber in fire design
k_p	Peak factor
l	Span length of beam
m_{floor}	Self-weight of floor structure
n_{40}	Number of modes below 40 Hz
q_k	Characteristic value for variable loads
t_{fire}	Number of minutes due to the fire safety requirement
$t_{ef.d}$	Effective depth for a nail
v_b	Basic wind velocity
w	Width of cross-section
w_{floor}	Static deflection of the floor structure
w_{fin}	Final deflection of the floor structure

Greek lower case letters

α	Imperfection factor
α_{md}	Unintended inclination angle
β	Reduction factor due to buckling length
β_c	Reduction factor due to instability
γ_M	Partial coefficient
$\gamma_{M,fi}$	Partial coefficient for timber exposed to fire
λ	Slenderness ratio of column
λ_c, λ_{rel}	Relative slenderness ratio
ν	Peak velocity
ξ	Modal damping ratio
ρ	Density of a material
$\sigma_{0.d}$	Design compressive stress parallel to the grain
$\sigma_{90.d}$	Design compressive stress perpendicular to the grain
τ_d	Design shear force
χ	Reduction factor due to slenderness
ψ_0	Combination coefficient for variable loads
ψ_2	Combination coefficient for variable loads, load case deflection

1 Introduction

In the introductory chapter the background, problem description, aim and objectives and methodology of the project are presented. Finally, the limitations of the study are listed.

1.1 Background

Today more and more people move from the countryside to the cities in Sweden. Investigations show that in the year 1960, 72.5 % of the Swedish population lived in cities. Today around 86 % live in the cities and in the year 2050 this number is expected to reach 90.3 % (WHO, 2015). The consequences of this urbanization are that the cities become more compact and the need for more residential buildings increase. In Gothenburg, as a result of the increased urbanization, the need for more residences has increased a lot in the past years.

One solution to the problem with lack of residence is to add more floors on already existing buildings. In Gothenburg most of the older buildings consist of only a few floors, which makes many of them suitable for adding floors. When adding floors it is desirable to minimize the impact on the load bearing system in the existing building. The impact can be minimized by using a lightweight material, since the total weight of the added construction will be lowered.

In Sweden, timber has been used as a construction material for a very long time. Timber is also a renewable material, which provides it environmental benefits compared to other building materials. In the past years, timber has been implemented more frequently in the construction industry (Svenskt trä, 2015a). In constructions, timber has the benefit of being a lightweight material, which means that the total weight of the building can be reduced. This makes timber a favorable building material to use when adding floors on an existing building (Wik & Karlsson, 2007).

1.2 Problem description

When adding more floors to an existing building, some different problems might occur. Since the existing building was designed for its own weight and loads, there might occur problems in some parts of the load bearing system.

Another problem that might exist when adding a lightweight structure in timber is the design of the connection between the existing and new building, since the wind load might cause uplifting forces in connections.

Also, another challenge when using timber can be the difference in span length between the structural system of the added floors and existing building. Since timber is a lightweight material, longer span lengths might require larger dimensions, which can lead to unnecessary material consumption.

1.3 Aim and objective

The aim of the study was to find the most suitable concept to perform an addition of floors in timber for residences, on an existing multi-activity building in the central part of Gothenburg. Further, a principle design of the identified concept was made.

The objectives were to identify and suggest suitable details in the transition between the existing building and added floors. Also, to determine what might limit the total number of added floors.

1.4 Methodology

The study initiated with a literature study and continued with an analysis in three different phases. The three phases were:

- Studying the reference building
- Evaluation of the four concepts
- Principle design of the new added floors and identify a suitable detail in the transition between the existing building and added floors

This approaching method was set up in consultation with the supervisors at COWI and at Chalmers.

The literature study was performed with the aim to deepen the knowledge about the subject. More specific the study included timber, FRP and steel as construction materials and previous projects with floor addition in timber. In continuation, different construction methods according to the predefined concepts were studied and possible connections in the transition were identified. The literature study about the predefined concepts was supposed to be the basis for the evaluation of the four concepts. The four concepts in this study were:

- Timber built on site
- Prefabricated timber modules
- Timber reinforced with FRP
- Timber and steel structure

The first phase consisted of studying the provided building Strömshuset, which was chosen together with the supervisors at COWI. Strömshuset was a suitable reference building since it has already been evaluated for the purpose of adding floors. First, the architectural and structural drawings of the building were studied to determine the geometry of the building and the structural system. The necessary information that was noted from the provided drawings was put together to new digital drawings in AutoCAD. Finally, the bearing capacity was calculated to determine the remaining capacity of the structural system due to vertical loads.

In the second phase, a matrix was put together to weigh the ciritera against each other. Afterwards, the concepts were evaluated according to the five criteria. The different criteria were described and motivated and all concepts were compared to each other for each criterion. The evaluation resulted in finding the most suitable solution for this type of building.

The four different concepts were evaluated according to the following criteria:

- Fire
- Production time
- Environmental impact
- Adaptation to the existing building
- Self-weight

The third and final phase aimed to perform a deeper analysis of the final concept. The analysis included design of columns, beams and floor structure due to vertical and horizontal stability. Also, suitable detail in the transition between the existing building and added floors was suggested. The calculations were made by hand.

Also, a study visit at Moelven Byggmoduler AB in Sandsjöfors was made to increase the knowledge about prefabricated timber modules and the manufacturing process.

1.5 Limitations

Due to the time limitation of the study, the four concepts to construct the additional floors were decided on beforehand. The time saved could be put on a deeper analysis on the most suitable solution for this type of building, which was the aim of the study.

The evaluation of the different concepts was limited to five criteria that were decided to be the most important for this kind of building. Therefore, only technical and environmental aspects were considered.

No calculations on the foundation were made since it was not part of the aim for this study.

Due to the limited time of the project the analysis was only performed on the building Strömshuset. The adjacent buildings Varuhuset number 12 and 17, were not investigated or considered in this study. Also, the calculations were limited to add up to five floors and no consideration to the local plan of the area was taken.

2 Densification of the cities

During the last years, trends are showing that more and more people settle down in the big cities. In the past, the urbanization mostly depended on relocating from the countryside to the cities, but the reasons for the densification have changed over time. Today the main reason is that more people are born in the cities than in the countryside. Also, the immigration is one factor connected to the densification (Svanström, 2015). Investigations made by the World Health Organization show that in the year 1960, 72.5 % of the Swedish population lived in the cities. Today around 86 % live in the cities and in the year 2050, the percentage is predicted to reach 90.3 % (WHO, 2015).

There are some economic, social and environmental benefits coming with the densification of cities. Densifying contributes to an increased utilization of existing infrastructure compared to the residential areas would expand outside the cities. This leads to a decreased use of cars and therefore fossil fuels (Larsheim, 2010). It also results in the opportunities to create an attractive living environment close to service and places of work (Andersson, et al., 2013).

A continuously growing population and the ongoing immigration, generate higher demands on today´s cities. This is especially noticeable by the current lack of residences in the largest cities of Sweden (Nyberg & Thunman, 2014). One solution to the problem, which do not involves building new residential buildings or place new ones between existing, is to add more floors on existing buildings. A great advantage with this solution is that green spaces can be preserved, which are important for the citizens and the urban environment. Another advantage with adding floors is the possibility to create more integrated cities with residences, offices and services in the same area (Larsheim, 2010).

In most cases when a reconstruction of a building takes place, like adding floors, a renovation also is made on the existing building. A renovation results in a longer lifespan and an upgrade of the energy efficiency of the building (Beyer, et al., 2006). If timber is used as the main construction material when adding floors to an existing building, the low self-weight is favorable and makes timber a suitable material to use for these projects (Wik & Karlsson, 2007).

2.1 Amendment of three-dimension property

In January 2004, the government implemented a legislative amendment current three dimension property utilization. The aim was to contribute to a more effective use of buildings and facilities in the cities. A result of this amendment is that existing buildings more easily can be developed with more floors, which generate a higher utilization of existing resources, as well as more residences (Lantmäteriet, 2016). With the term three dimension property a building can be delimitated in both horizontal and vertical directions and constitute a volume instead of only be delaminated horizontal and constitute a surface. It also results in the opportunity for a floor in a building to be defined as an own property, which facilitates the possibility to add more floors. Figure 1 illustrates a possibility with a three-dimension building, where area 2 is separated from area 1 and categorized as an own property (Boverket, 2004).

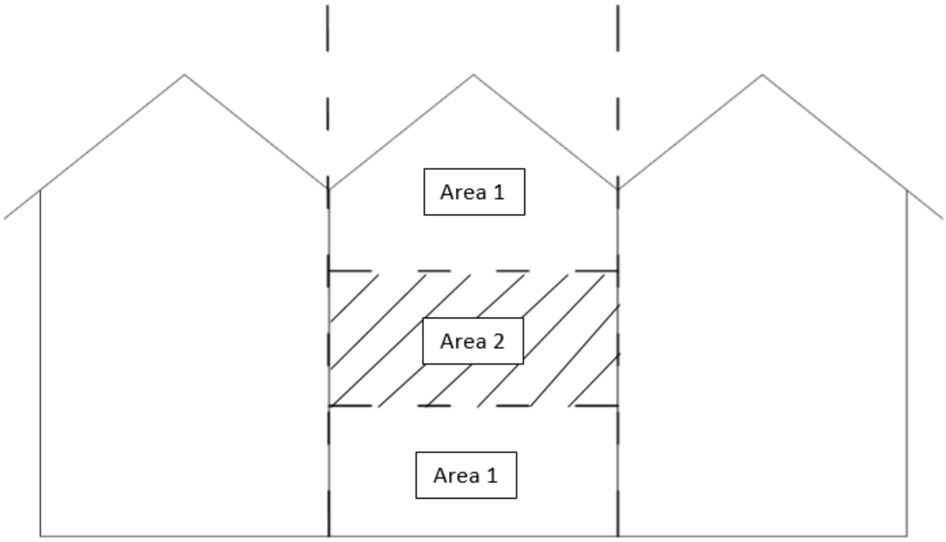

Figure 1 – Illustration of the principle with three-dimension property. Area 2 is an own property separated from Area 1.

2.2 Previous projects of floor addition in timber

In this section some previous projects of adding floors in timber are presented. Since not so many projects of adding floors in timber have taken place in Sweden, the lack of experience generates a low proliferation of the knowledge about this type of projects. Therefore, information from timber suppliers in Sweden is the only references found.

2.2.1 The neighbourhood Embla in Umeå

One of the buildings in the neighborhood Embla in Umeå was added with three new floors in year 2015. The reason that only three floors were added was because of the local plan in the area. The new load bearing system consists of prefabricated modules in glulaminated timber. Because of its lightweight, timber modules were used for the added structure. Timber modules also contribute to a fast building process due to the high degree of prefabrication (Martinsons, 2012).

A Structural Design Manager[1] at Martinsons informed that the existing building consists of a beam and post system in concrete with stabilizing concrete walls. At one location in the existing building, strengthening of the structural system had to be done so that the additional horizontal forces could be taken care of. In this case, an extra stabilizing wall was added.

The transition between the existing building and the new added floors were specially designed, since there were differences in level for the existing concrete. A common problem when extending buildings is the correspondence of the

[1] Structural Design Manager, Martinsons, Interviewed 22 February 2016

existing building and the drawings. The reason is often that the drawings are not entirely updated after renovations have taken place[2].

2.2.2 Tegeludden in Stockholm

In year 2009, the buildings in Tegeludden in Stockholm were rebuilt from office buildings to residences. In connection to this, two new floors of timber modules were added. At a study visit at the factory of Moelven in Sandsjöfors the Technical Director[3] informed that the arguments for building with prefabricated timber modules were the same as for project Embla. The new floors are shown in Figure 2 and Figure 3.

The load bearing system in the existing building consisted of load bearing walls and stabilizing cores consisting of elevator shafts and stairwells in concrete. Compared to the neighborhood Embla, there was no need for strengthening in the existing building. Further, the local plan limited the extension to two floors. Thus, more floors could have been added if the local plan would not have limited the project[3].

Figure 2 - The production of the new added floors in Tegeludden (Moelven, 2016)

Figure 3 - The neighborhood Tegeludden after completion (Moelven, 2016)

[2] Structural Design Manager, Martinsons, Interviewed 22 February 2016
[3] Technical Director, Moelven, Interviewed 10 March 2016

3 Construction materials

In this chapter the materials that are used in the four concepts are presented. The five criteria have been the basis for the information of the materials presented in this chapter.

3.1 Timber

Around 70 % of the area in Sweden is covered by forest, mostly spruce and pine, but in the southern parts there are also some leafy trees (Svenskt trä, 2015b). The growth of Swedish forests is larger than the felling, and 12 % of the total export is represented by pulp and paper industry and sawn timber engineered products. The construction industry uses 55 % of the sawn timber in Sweden (Crocetti, et al., 2011).

Timber can be used in several ways in the construction industry and is the building material with the oldest traditions in Sweden. Until year 1994, there was a law in Sweden that did not allowed more than two storey-buildings in timber, partly because of the high risk of fire. After this amendment, the construction industry started to implement timber more in the constructions (Svenskt trä, 2015a).

There are many benefits with timber, both economic and technical. For example, timber is a strong material in relation to its weight. It is also environmentally friendly since it is a renewable material. These properties generate the possibility of using timber as the main construction material in many types of buildings (Wik & Karlsson, 2007).

3.1.1 Environmental benefits

A major problem in today´s society is the increasing amount of emissions from greenhouse gases. The construction industry contributes with 30 % of the total amount of greenhouse gas emissions. It also consumes around 40 % of the total energy use. In year 2009, the European Union decided that the percentage of emissions should be reduced by 80-95 % for the construction industry. This, among other things, can be achieved by reducing the use of energy (UNEP, 2009).

In the manufacturing process of many building materials like steel, concrete or brick, large amounts of energy are required. This leads to high emissions of carbon dioxide, CO_2. To be able to reach the goals decided by the European Union, the construction industry can influence and lower the emissions by implement more timber in the constructions. Since timber is a renewable building material, less amount of CO_2 would be emitted if timber was used instead of other common materials (Svenskt trä, 2015c). A comparison of different materials with regard to emissions from the manufacturing process is illustrated in Figure 4. It should be mentioned that the storage of CO_2 in the material is not taken into account.

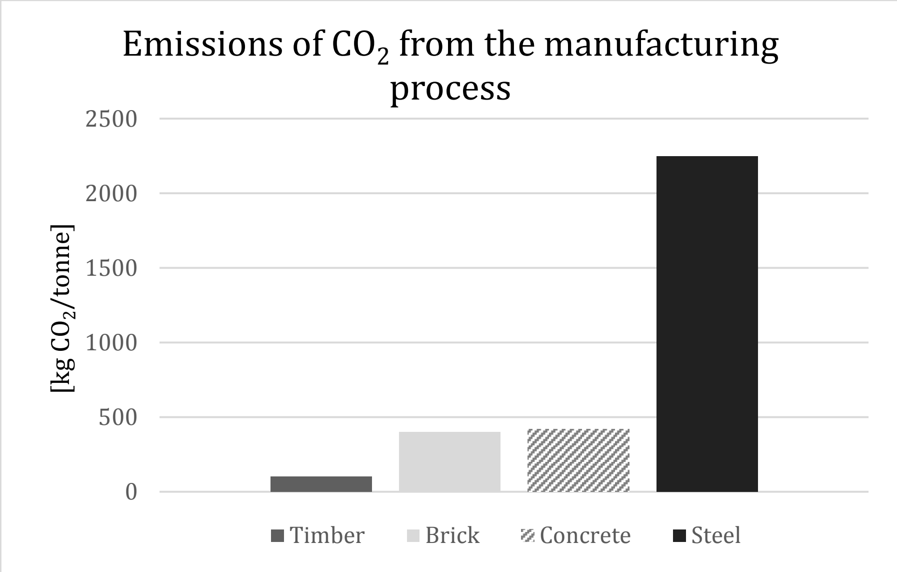

Figure 4 - Values of carbon dioxide emissions during the manufacturing of some common building materials. Data from (Svenskt trä, 2015c).

Another benefit of using timber is that during the manufacturing process, there is very little to no waste, since it can be used as an energy source. Also, timber has the benefit of being able to recycle or sometimes reuse after its lifespan (Beyer, et al., 2006).

3.1.2 Technical benefits

Timber is a lightweight material with a density of 300-600 kg/m^3. Due to this, the total weight of the building can be lowered by implementing timber (Crocetti, et al., 2011). The low density of timber makes it a suitable and useable material when adding floors to existing buildings[4]. Other benefits with the low density of timber are that it is a convenient material to work with and facilitates the transport of the material (Crocetti, et al., 2011).

Timber is a material with a high strength and load bearing capacity, both in tension and compression, in relation to its weight (Wik & Karlsson, 2007).

One specific advantage with timber in buildings is the ability to reduce the energy use. The reason is because timber's natural thermal insulation qualities. Timber constructions have a high insulation in relation to buildings with other conventional materials. To provide double thermal insulation values, an external wall in timber only needs half the thickness compared to a wall in concrete or brick (Beyer, et al., 2006).

In year 2010, the European standard, Eurocode, for fire safety in timber structures was revised in such a way that buildings should limit the risk of fire and the risk for the fire to spread. One of the main requirement in the new codes related to fire safety is that a timber building up to four storeys needs to be in safety class REI60. For higher timber buildings, the requirement is REI90. The

[4] Technical Director, Moelven, Interviewed 10 March 2016

number indicates the time, in minutes, that a structure should withstand a fire (Crocetti, et al., 2011).

Timber constructions have a high resistance against fire. The material burns immediately when it is exposed to fire, but it burns in a predictable and slow way. During a fire, a protective layer of coal is created on the surface. Under the layer of coal, the timber stays intact (Crocetti, et al., 2011). Timber remains the load bearing capacity during a fire, compared to steel that melts and collapse unexpectedly when a critical temperature is reached. The charring rate for timber is between 0.5-1 mm/min (Bergkvist & Fröbel, 2013).

3.1.3 Disadvantages

Due to the lightweight and low density, timber is very sensitive against vibrations compared to other building materials. When a lightweight material is exposed to live loads, for example on the floor structure, vibrations occur easily. These vibrations are perceived as unpleasant and should therefore be limited. The vibrations can be reduced by increasing the mass, stiffness or damping factor, but it is not always easy since all the parameters in a structure depends on each other. If one parameter is increased, another value might need to be increased to fulfil the requirements (Thorsson, 2016).

Another disadvantage with a lightweight material is that the low weight is unfavorable to resist large horizontal loads, such as wind loads. Due to this the structural system often needs to be strengthened against horizontal forces. Especially, in taller buildings since the wind loads increase with height (Crocetti, et al., 2015).

Since timber is a natural material, there are number of characteristics for structural timber that can be seen as defects. One of the most common defects is knots. Knots have a large influence on the strength of sawn timber, since the fiber orientation near and around the knots is distorted. As a result of the changes in orientation, the fibers sweep around the knots and are no longer continuous. This affects the strength negatively. Thus, an element with a lot of knots is categorized into a lower strength class (Crocetti, et al., 2011).

One of the external factors that have a large influence on timber is water. Since timber is a natural material, the moisture content is varying with the relative humidity in the surrounding air. Two consequences from this are shrinkage and swelling. When timber is exposed to a low degree of relative humidity, the moisture content in the material is reduced and it will shrink. Correspondingly for a high relative humidity, the material will swell. Thus, variation in moisture content causes geometrical changes, distortions, in the cross-section of the element, which affects the strength. The most common distortions are twist, spring, cup and bow, which are illustrated in Figure 5. The twist is the distortion mode that causes the largest problem due to the lack of load bearing ability in the structural system of buildings (Crocetti, et al., 2011).

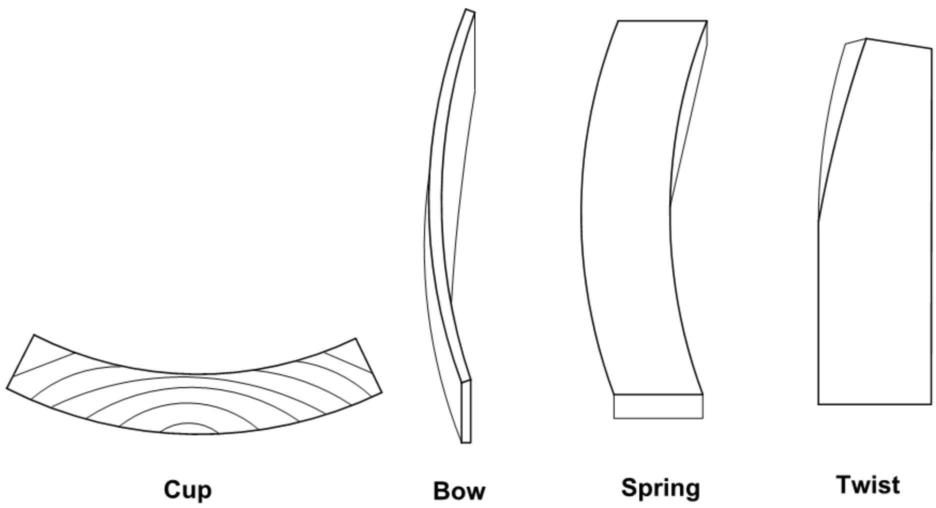

Figure 5 - The four different distortion of timber due to varying moisture content

Timber is an anisotropic material which means that the material has different properties in different directions. The three principle directions are longitudinal, radial and transversal. The strength differs between the directions, something that needs to be considered when designing timber structures. For example, when timber is loaded perpendicular to the grain, both the stiffness and strength are very low. The reason is because the forces to pull apart or break the fibers are much lower than if the timber is loaded in tension parallel to the grain (Crocetti, et al., 2011).

3.2 Fibre Reinforced Plastic

In civil engineering, Fibre-reinforced plastic (FRP) has been used for approximately 30 years. FRP is a composite material, which means that it is built up as a polymer matrix mixed with some reinforcing fibres. Thus, a material which consist of at least two different materials. The reinforcing fibres can vary between several materials, for example roving of glass-, carbon- or aramid fibres, chopped fibre mats or woven fabrics. The polymer matrix surrounds the fibre reinforcement and together they form a FRP composite (Friberg & Olsson, 2014).

The bonds between the polymer and the fibre, the interface, are of great importance since that is where the load transferring occurs in a structure. In addition, the angle between the fibres and the direction of loading governs the properties of the bonding. The angle and direction of loading are also related to the properties of the matrix in form of its strength and stiffness. Hence, the interaction between the fibres and the matrix has a large influence on the failure mode of a structure. The bond, called resin, is often combined with additives and fillers, and is classified as a thermosetting or thermoplastic resin (Friberg & Olsson, 2014).

There is a lack of knowledge about the performance when using a composite FRP structure regarding how the material behaves, compared to more common materials like steel, timber and concrete. However, since FRP is an anisotropic material, it is possible to arrange the fibres in the same direction as the principle stress. An arrangement of that kind generates an increased structural efficiency, compared to isotropic materials like steel and concrete, which do not have that possibility. This possibility provides FRP a great technical advantage (Friberg & Olsson, 2014).

3.2.1 Environmental benefits

The manufacturing of the polymers in FRP composites is in many cases produced by using waste products from the oil industry. Due to this, the manufacturing of the polymers generates very little waste material and less energy (Friberg & Olsson, 2014).

FRP composites are very durable materials and can resist for example chemicals, moisture and temperature in an acceptable way under appropriate loading conditions. This makes FRP very favourable to use for concrete and timber strengthening or use FRP in moist environment. But if the conditions are not acceptable or FRP is used in a harmful environment, the mechanical properties can be seriously disturbed (Friberg & Olsson, 2014).

Depending on what kind of material the FRP composite consists of, the amount of reused material in the composite varies. Therefore FRP can be produced with a relatively large amount of reused materials, which is an environmental benefit (Friberg & Olsson, 2014).

3.2.2 Technical benefits

FRP is a light and strong material and the density depends on what polymers and fibres that are used in the composite. The general value of the density is between 1200-1800 kg/m^3. Because of the relative low density and that FRP is a strong material, the strength to weight ratio is high (Friberg & Olsson, 2014).

Due to the fact that FRP can consist of several different materials, it is difficult to predict how FRP will behave when it is exposed to fire. But, it is known that the transition temperature for FRP is relatively low and the material is very sensitive to fire. The knowledge about the behaviour of the different materials in FRP during a fire are well known, for example, fibres that performs better can be used in the composite. Also, the orientation of the fibres influences the performance when exposed to fire and can therefore be taken into account to improve the fire resistance (Friberg & Olsson, 2014).

Another benefit is that FRP provides higher heat insulation than steel, but during a fire, the mechanical properties are lost at lower temperatures. The fire resistance of FRP can be increased by using fire protection like coating or additives, which are the most common techniques (Friberg & Olsson, 2014).

3.2.3 Disadvantages

When considering the fibres, the manufacturing process is worse than the polymers. Very high temperatures are required to produce the fibres, which lead to a large energy use and amount of materials. In addition, fossil fuels are used in the manufacturing process. However, due to the new demands by the European Union, research of implementing green composites has been initiated. These composites are resins consisting of recycled and biologically renewable resources. But it is worth mentioning that although this, the energy consumption in total for the FRP composite, is around one fourth of the manufacturing of steel today (Friberg & Olsson, 2014).

One large disadvantage with FRP is the possibility to recycle the material. Due to the bonding resins, thermosetting FRP is very hard to recycle. The reason is the lack of possibility to re-melt the material. For a thermoplastic FRP the possibility to recycle is higher, but requires a re-melting process, which in turn requires amounts of energy. Also the decommissioning of FRP cost money (Joâo, et al., 2011).

When FRP is exposed to fire, the matrix is very exposed because it burns very easy and softens fast when the temperature is increasing. This is because of the glass transition temperature is low and when this temperature is reached, the matrix goes from hard and brittle behaviour to viscous and rubbery. Also, most of the FRP composites are flammable which means that the spreading rate of the fire is high on the surface (Friberg & Olsson, 2014).

As mentioned earlier, FRP can be fire-protected with additives or coating to increase the fire resistance. The negative aspect with this is that the methods are very costly (Friberg & Olsson, 2014).

3.3 Steel

Steel is a widely used material in the construction industry, both in commercial and industrial buildings. Steel is a material with high strength in proportion to needed dimensions, as well as high stiffness, toughness and ductility. It is also a material that can be developed into nearly any shape. The different members in a steel structural system are either welded or bolted together (Steel construction, 2016).

The properties of structural steel depend on both the chemical composition and the method of manufacturing. Steel is an alloy that mainly consists of iron. A small addition of other materials can have a significant effect on the structural properties and strength. The degree of added materials in the alloy can vary and are governed by the limits in the product standard. The most common alloy material in steel is carbon. Other common alloy materials in steel are manganese, niobium and silicon (Steel construction, 2016).

During the manufacturing of steel construction products, the properties that need to be considered are:

- Strength
- Toughness
- Ductility
- Weldability
- Durability

Construction steel is presented in many different ways, where hot rolled and cold rolled are the most common ones. The density of construction steel is around 7800 kg/m^3, which is about 15 times heavier than timber (Al-Emrani, et al., 2013).

3.3.1 Environmental benefits

Steel is the most recycled building material in the world, with a global recycled degree over 60 percent. This corresponds in over 650 mega tones recycled steel ever year. Since steel has a long product life it can easily be recycled without any loss of quality. Recycling of steel saves great amount of energy, compared to newly produced steel. It also generates a significant saving of the raw material (Worldsteel association, 2014).

3.3.2 Technical benefits

Steel is a construction material that can be categorized as a strong material, which generates great advantages and possibilities during construction. Other important technical benefits with steel are that the material is flexible and ductile and can bend out of shape without cracking. When it is subjected to large forces, it will not suddenly crack. Instead, it will slowly bend out of shape. Due to these properties, a structural system in steel performs better than many other building materials when subjected to earthquakes (Understand construction, 2016).

3.3.3 Disadvantages

A process that affects the strength of steel is corrosion. Corrosion is an electrochemical process that occurs when the iron in steel is exposed to water and oxygen. The process starts at the surface of the material and during time the corrosion goes deeper into the material and cost a lot of damage (Tordoff, 2003).

Steel manufacturing is very energy intensive, even if progress is being made. The energy used during the manufacturing process has been reduced by nearly 60 % over the last 50 years. It is important for the competitiveness of the material to continue in this direction compared to other materials (Worldsteel association, 2014). As shown in Figure 4 the manufacturing of steel still generates very large amount of carbon dioxide emissions. The emissions are almost 23 times larger than the emissions for manufacturing timber (Svenskt trä, 2015c).

Since steel is a material that loses strength, change properties and shape during high temperatures, it is a material that is very sensitive against fire. When unprotected steel is exposed for fire it melts and collapse unexpectedly when the critical temperature is reached. The critical temperature is varying and depending on the structural element type, orientation and loading. It is often considered as the temperature where the yield stress of the exposed steel has been reduced to 60% of yield stress in room temperature. To fulfil the requirements of fire, steel needs to be protected. This can be made with gypsum board or fire resistance paint, which is costly (Bergkvist & Fröbel, 2013).

4 Reference building - Strömshuset

In this chapter an introduction to the reference building is given. The geographical location is described as well as the structural system of the building. Also, previous investigations that have been made on the reference building are defined. In the end of the chapter the capacity of the structural system is determined.

4.1 Introduction to Strömshuset

The building that has been used as a benchmark and reference building in this study is Strömshuset, which is located in the neighborhood with the same name. Strömshuset is located in the central part of Gothenburg. The building is especially recognized among the population in the city, since it is located next to the cathedral and has a thermometer along one of the façades.

The neighborhood Strömshuset consists of three different buildings, Strömshuset, Varuhuset number 17 and Varuhuset number 12. During the last years, the buildings have been connected to each other and are now built together.

The oldest part of the neighborhood, Strömshuset, was built in year 1935. Varuhuset number 17 was built in year 1970 and was extended by Varuhuset number 12 in year 1977. In Figure 6 the architectural plan is shown and the three buildings are marked out.

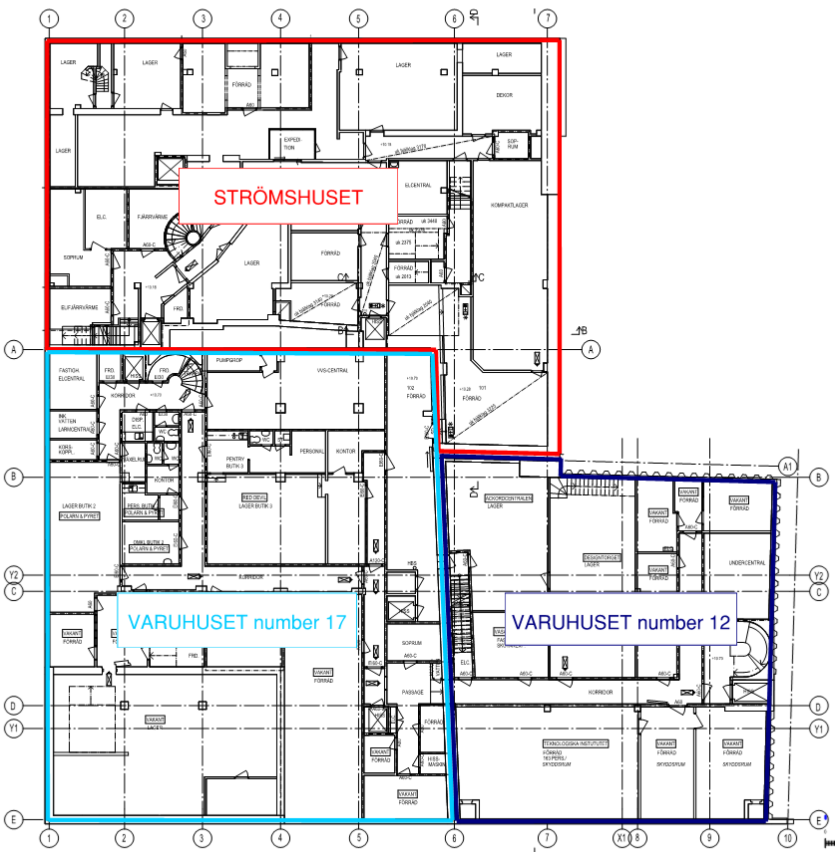

Figure 6 - Plan view of the neighborhood Strömshuset showing the three buildings

According to the architectural drawings, there are several different activities in the building. The basement contains inventories while floor 1 and 2 consist of shops and floor 3-8 contain office areas and lecture halls.

4.2 Structural system

Strömshuset has nine floors, where the bottom floor is a basement. The two top floors have a slightly smaller area compared to the rest of the floors. This is illustrated in the sectional sketch of the building in Figure 7. Each floor has a free height of 2.8 meters.

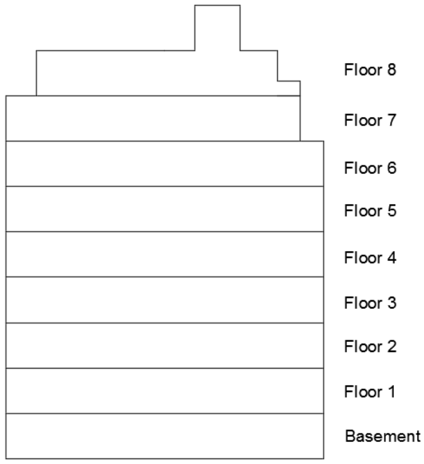

Figure 7 – Illustration of the building in section

The structural system consists of a beam-post system in steel. In the floor structure, the steel beams are embedded in concrete. The reason is because in the past, a common solution to protect the steel from fire was to surround it with concrete. The beams have a cross-section of the old type DIP, which is I-shaped and today replaced by the HE-profiles.

The columns are also of the DIP-profiles. The different identified DIP-profiles in Strömshuset are illustrated in Figure 8. The maximum distance between the columns are 5.5 m and 7.5 m respectively, which can be seen in Figure 9.

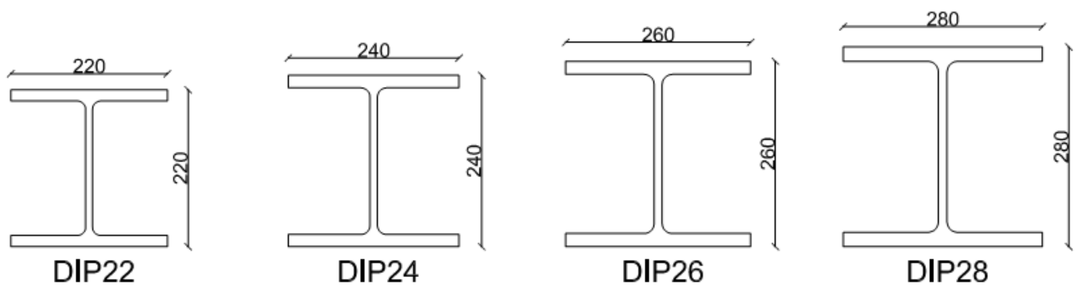

Figure 8 - Cross-section of the different DIP-profiles in Strömshuset [mm]

On top of the columns, the floor structure is placed. The floor structure consists of an 80 mm thin concrete slab, which is placed on top of the steel beams. Timber flooring is then used on top of the concrete layer. This type of floor structure was commonly used at the time Strömshuset was built. The total height for the floor is 0.4 m, except for the bottom slab, which has a thickness of 0.5 m.

In the structural system, there are also elevator shafts and stairwells, which provide horizontal stability in the building. These are in concrete and their positions are shown in the drawing in Figure 9. The thickness of the stabilizing walls and external walls are 0.3 m. Although, the external walls along line 6 and 7 are 0.5 m.

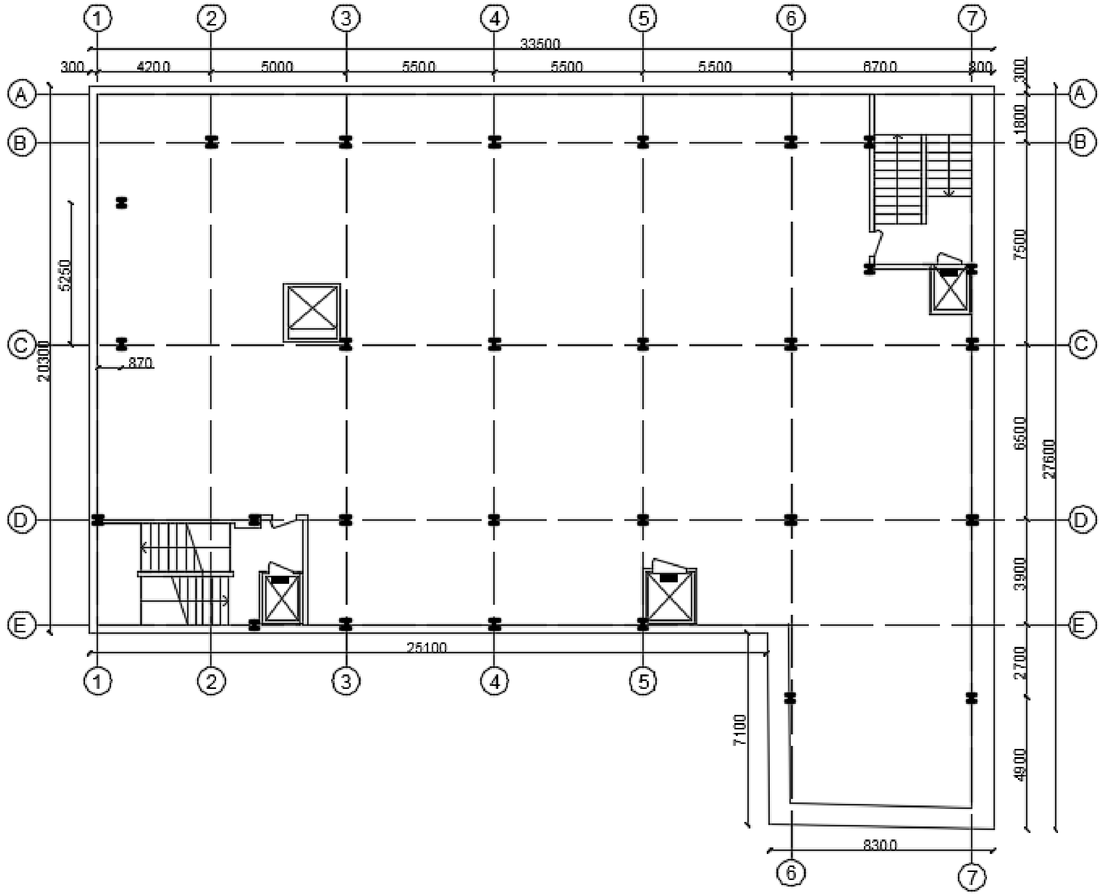

Figure 9 - Floor plan of Strömshuset with dimensions [mm]

4.3 Previous investigations

In the years 2010 and 2016, COWI performed investigations of the bearing capacity in the foundation of Strömshuset. The aim of the investigations was to analyse the capacity of the foundation and find out the possibility to add 1-3 new floors. The investigations contain only calculations of the vertical loads acting on the structural system on each floor. The results presented by COWI are attached in Appendix 17 and form the basis when determining the vertical load bearing capacity in this study.

The calculations of the vertical loads are based on the values shown in Table 1 and follow the regulations of BKR 13, which are the construction rules of the Swedish National Board of Housing. The regulations ceased to be valid in year 2011 and were then replaced by the European standards, Eurocode (Boverket, 2014).

Table 1 - Characteristic values for loads and combination factors used by COWI in previous investigations (COWI, 2010)

	Load	Fixed load	Ψ	Free load	Ψ
Permanent load					
Concrete slab	1.9–2.4 kN/m^2				
Timber cover	0.3 kN/m^2				
Steel beams	3.0 kN/m^2				
Installations	0.3 kN/m^2				
Partition walls	0.4 kN/m^2				
Basement slab	12.0 kN/m^2				
Haunch	6.0 kN/m				
Façade	10.7 kN/m				
Basement wall	22.6 kN/m				
Variable load					
Basement		1.0 kN/m^2	1.0	1.5 kN/m^2	0.5
Floor 1-2		0 kN/m^2	1.0	4.0 kN/m^2	0.5
Floor 3-8		1.0 kN/m^2	1.0	1.5 kN/m^2	0.5
Snow	1.2 kN/m^2		0.7		

4.4 Capacity of the existing columns

To determine the number of new floors that can be added due to vertical loads, the utilization ratio due to buckling in the columns had to be calculated. Figure 10 shows the position and type of columns according to the existing drawings. The most used type of column in the building is DIP28 and at some locations smaller profiles are used.

Since the drawings of Strömshuset are very old, necessary information like steel quality was not expressed. In the time the house was built, it was common to use the steel that was available and therefore the quality may differ. In a conversation with the supervisors at COWI, it was decided to assume a low steel quality in today´s standards. In this case S235 was chosen. The columns were also assumed to have the buckling length of 2.8 m, which is the same as the free height of each floor. The calculations due to buckling can be found in Appendix 2.

The most exposed column turned out to be P11 illustrated in Figure 10, with a utilization ratio of 88.7 %. The rest of the utilization ratios from the calculations can be found in Appendix 14.

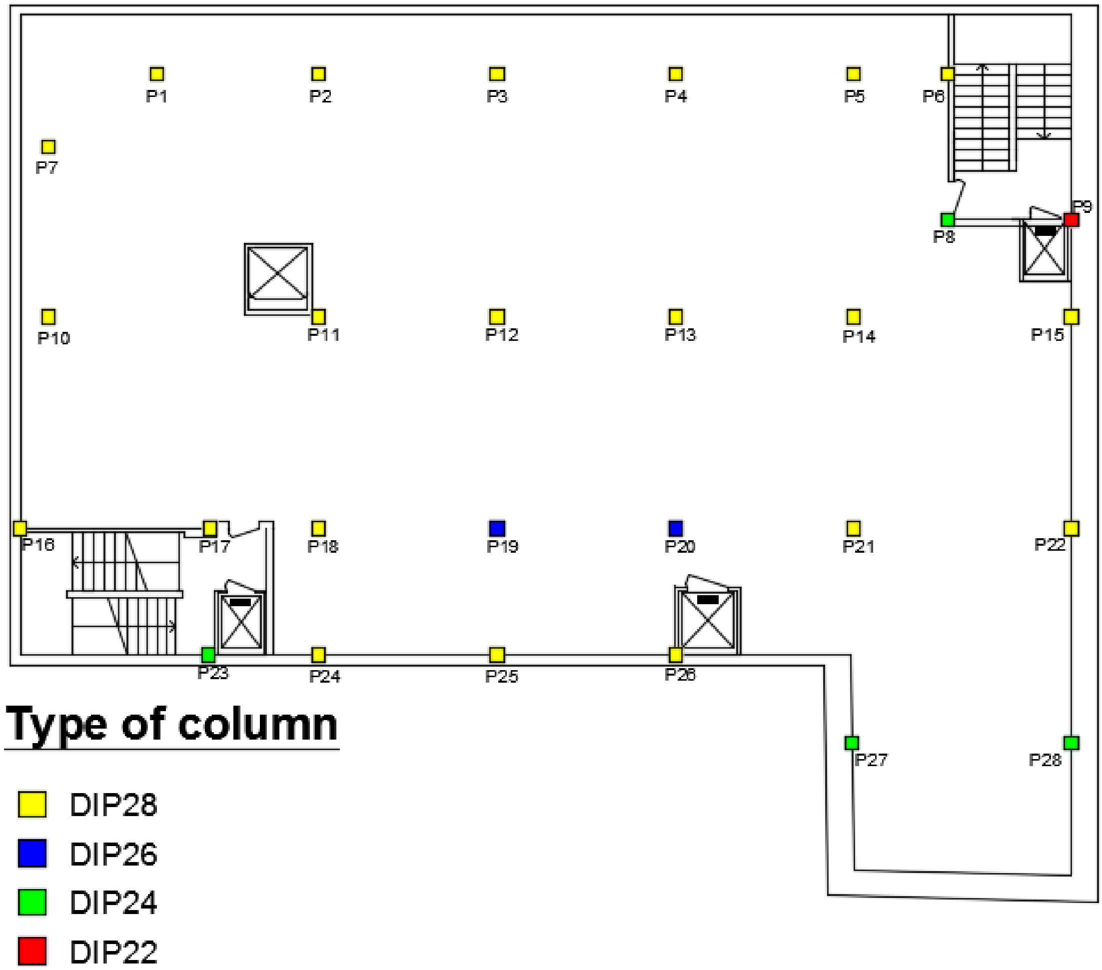

Figure 10 - Illustration of column types and numbering of the columns

5 The concepts for adding floors on Strömshuset

There are several building techniques that can be used when building with timber. In this study, four of them will be presented in this chapter. Further, in chapter 6 the proposals were evaluated against each other, to find the most suitable solution to use when adding floors on Strömshuset.

The reason that the four concepts presented in this chapter, were chosen was because all of them are suitable building methods. It was interesting to consider both methods for building on site and prefabricated options. Further, timber in combination with other materials was also considered to be interesting, in this case steel and FRP as strengthening. All concepts were predefined in the initial stage and decided by the authors, together with the involved supervisors.

5.1 Timber built on site

Timber built on site is one of the standard design method regarding timber structures. When constructing timber buildings on site there are two options, one is that pre-cutted timber arrives to the construction site and the other alternative is that timber is cut and adapted on site. With the design method built on site the bearing walls mounts together down on the foundation slab or at a storey and then raised, often by handcraft, and placed into position. This design method is built floor by floor and the bearing system is often constructed as an open stud frame without any insulation and covering layer. The next step during the construction phase contains the roof design and finally the insulation and covering layer are installed (Bergkvist & Fröbel, 2013).

5.1.1 Panel systems

Panel systems are a common method within timber built on site that are based on planar building elements. The basic technology for panel systems consists of either light frames or solid timber elements (Crocetti, et al., 2011). One advantage by using light frame constructions is that it provides great opportunities to integrate the technical equipment in the cavity (TräGuiden, 2003c). The maximum span length for a panel system is around 8-10 meters but the choices of floor structures will be limited when the span is longer than 6 meters (TräGuiden, 2003b).

5.1.2 Beam-post system

Beam-post system is another common stuctural system for timber structures built on site and can have various structural design. The beam-post system mostly occurrs in structures with large span, such as an industry building and arenas (Crocetti, et al., 2015).

The system can also be used in multi-storey buildings with smaller span lengths. For smaller spans the system is based on rectangular modulus with a maximum span length of 8 meters. The limit is based on the relation between the span length and the floor element height, which can be of great importance for timber structures since the dimensions can be very large otherwise (Crocetti, et al., 2015).

5.2 Prefabricated timber modules

During the last decade, it has become more popular to use prefabricated industrially manufactured timber modules in construction projects in Sweden. This means that most of the production process takes places in a factory and not in direct connection to the building site. The degree of prefabrication can be very high, up to 80 percent. This leads to a reduced construction time, compared to a frame built on site. According to a Technical Manager at Moelven, it is possible to shorten the construction time with around 30-35 %[5].

The degree of prefabrication depends on the specific situation. For the case when floors are added on an existing building, the degree of prefabrication are less, compared to a new building. One negative aspect with modules is that it is difficult to adapt elevators, stairwells and shafts for technical installations between the existing and new building. Therefore, modules are preferred when building new constructions compared to adding floors[5].

Since most of the manufacturing takes place in a factory, the environment is controlled and the use of energy can be handled in a better way. Also, the waste can easily be taken care of and be used as an energy source (Bergkvist & Fröbel, 2013).

The modules form a self-bearing system and are connected to other modules during construction. In the construction procedure, the modules are first stacked on top of each other. After that, audio blocks are placed between the modules to minimize the sound transmission and steel sheets are placed in the corners to counteract horizontal loads. The modules usually have no problem with vertical loads[5]. The system of building with modules can be made by using a light-frame system or a solid wood modular system. The main difference is that a solid wood system opens up for the possibility to achieve stiffer stabilizing walls (Crocetti, et al., 2011).

The size depends on the possibility for transportation of the elements, but the standard limitation is around 4.15 meters in width. Thus, the free maximum span width is close to 4 meters and the length can vary up to 13 meters (Crocetti, et al., 2015). The weight of the modules is assumed to correspond to a timber frame of the same size[5].

The demands for protecting the modules against fire are high and must fulfill demands of 60 minutes for the fire to spread between the modules. If the building consists of more than four stories, the demand is 90 minutes. For a project of adding floors, the demand is automatically 90 minutes. The most

[5] Technical Director, Moelven, Interviewed 10 March 2016

common fire protection for the modules is gypsum boards, but also mineral wool can be used[6].

5.3 Timber with FRP

In a timber construction, FRP can be used to increase the strength and stiffness of a structural element by reducing the cross sectional area. Today FRP is more frequently used in buildings as a reinforcing material in timber or concrete elements (Friberg & Olsson, 2014). In timber elements, there are several application areas for FRP reinforcement. For example to strengthen beam-ends, reinforcement perpendicular to the grain and reinforcement in bending zones (Schober, et al., 2015).

5.3.1 Beam end reinforcement

FRP can be used in the beam-ends to restore the capacity in a decayed end. The decayed parts are cut off and new holes are drilled, which are filled with FRP reinforcement. This method is more common for inhabited floors or to restore elements in older buildings with complicated timber joints. The principle for beam end reinforcement is shown in Figure 11 (Schober, et al., 2015).

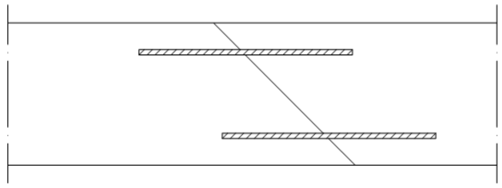

Figure 11 - FRP reinforcement for strengthening in the connection between two beam ends

5.3.2 Tensile reinforcement perpendicular to the grain

The tensile strength of timber perpendicular to the grain is significantly lower than the strength parallel to the grain. Due to this, FRP can be used to increase or maintain the load-carrying capacity of structures loaded perpendicular to the grain. Examples of where these stresses might occur are notches, holes or curved beams. The purpose of involving FRP is to increase the strength and stiffness of the element and also lead to a more plastic failure. In a curved beam, the reinforcement is placed in the apex zone and in the transverse direction of the grains. The example of the curved beam is illustrated in Figure 12 (Schober, et al., 2015).

[6] Technical Director, Moelven, Interviewed 10 March 2016

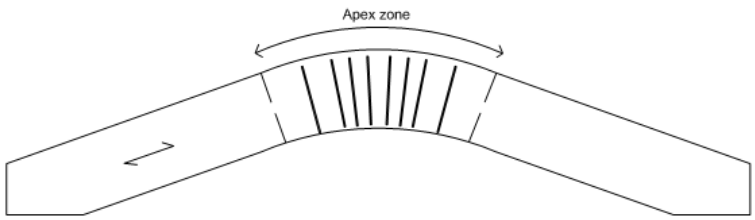

Figure 12 - FRP reinforcement for a curved beam to increase the resistance against tension perpendicular to the grain

5.3.3 Bending reinforcement

Reinforcing a member subjected to bending can be made in two different ways, internal or external reinforcement. Internal reinforcement can be bonding rods or strips, placed in grooves in the tension and compression zones of the element subjected to bending. External reinforcement can be FRP plates bonded to the tension side. Experiments have shown that adding small plates of FRP, can have significant effect on the bending stress. For example, a beam with FRP plates can increase the strength and stiffness with up to 100 percent. A beam with external FRP plate bonded to the tension side is illustrated in Figure 13 (Schober, et al., 2015).

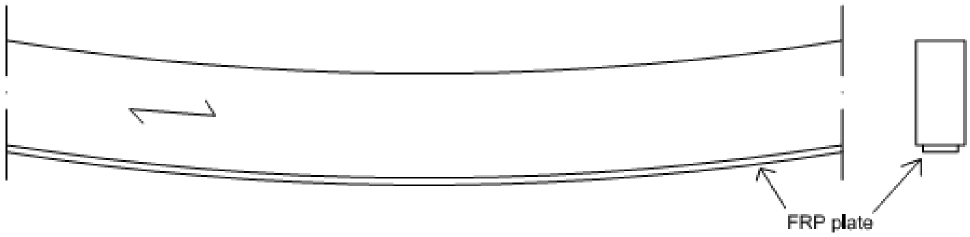

Figure 13 - External FRP plate bonded to a beam on the tension side to increase the bending resistance

5.4 Timber and steel structure

One way to build with a combination of steel and timber is to use steel for the columns and beams and timber for walls, diaphragms and floors. A mixed structure of this kind makes it possible to utilize the advantages of both materials. In this combination, the steel handles the vertical gravity loads like permanent loads and variable loads. The timber takes the horizontal loads like wind loads. These proposals are a variety of the beam and post system, described in chapter 5.1.2 and is suitable for structures with larger span length and open spaces (TräGuiden, 2003a).

Another effective combination of timber and steel is to replace the timber beams with steel beams when the loads are large. This leads to a decrease in the height of the beam compared to if timber would has been used instead (TräGuiden, 2003a).

6 Evaluation of the concepts

To find the most suitable concept for adding floors on the existing building, the four concepts were evaluated with regard to five criteria. The criteria were selected to be the most relevant for this kind of building and project. Further, the criteria were evaluated against each other in order to determine the weighting of the criteria in the evaluation process.

The four concepts were then ranked against one another and multiplied with a weighting factor. The weighting factor was based on the percentage of importance that the different criteria were given, after deciding how important the criterion was regarded in correlation with the other criterion. The workflow of the evaluation phase is illustrated in Figure 14.

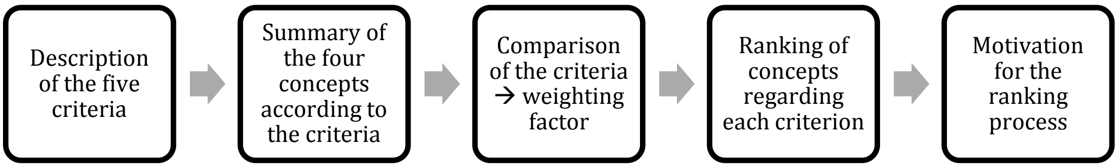

Figure 14 - Illustration of the workflow of the evaluation phase

6.1 Description of the evaluation criteria

In this section a description of each criterion is given and there meaning is presented.

- **Fire** considers how all the materials in the concepts acts when exposed to fire. All materials must fulfill the demands for the fire safety. Also, the need for fire protection is taken into consideration.
- **Production time** refers to the expected production time on site. This time will differ between the different concepts, since the degree of prefabrication varies.
- **Environmental impact** considers the amount of emissions during manufacturing. This criterion also considers the use of energy during manufacturing and the possibility to recycle or reuse the building materials after the life span of the building.
- **Adaption to the existing building** refers to the degree of customization between the structural system in the different concepts and reference building. The suitability of the structural system to the proposed activity of residences are also considered in this criterion.
- **Self-weight** considers the density of each material in the concept together with the dimensions of the material that are needed for the structural system. The total self-weight of the new floors is considered to reduce the impact on the reference building.

6.1.1 Motivation for the choice of criteria

Since the five criteria were chosen by the authors, motivations for why each of them were regarded as important for this study are presented in this section.

Fire

Fire is interesting to evaluate since the concept includes different materials and the behavior during a fire differ. Also, the need for fire protection varies between the concepts.

Production time

The production time was chosen as one of the criterion since Strömshuset is located in inner city. Due to this it is important that things like the surrounding buildings, traffic and activities in the building, are not affected during a longer period of time.

Environmental impact

The impact on the environment is a very timely topic. Therefore, it is important to determine how the specific concept will affect the environment into consideration.

Adaption

When adding floors it is important that the new construction is flexible and possible to adapt to the existing building. Especially since the existing building might have been renovated during the years, which could not be seen in the drawings. Therefore, the adaption criterion was chosen to be considerd.

Self-weight

The self-weight is also of interest since it is desirable to reduce the impact on the existing load bearing system and building. Due to the fact that the four concepts consist of different building materials, it is interesting to consider the self-weight.

6.2 Summary of the four concepts

In Table 2 the information of the four concepts regard the five stated criteria are summarized. The summary has been made to more easily understand the comparison between the concepts in chapter 6.4.

The different concepts in Table 2 are represented in the following order:
A – Timber built on site
B – Prefabricated timber modules
C – Timber reinforced with FRP
D – Timber and steel structure

Table 2- Summary of advantages and disadvantages for the four different concepts due to the criteria

	A	B	C	D
Fire	- High resistance - Predictable and slow burning - Remain bearing capacity	- High resistance - Controlled and slow burning - Remain bearing capacity	Timber – High resistance FRP – Sensitive and fast burning - Need for fire protection	Timber – High resistance Steel –Melts and collapse unexpectedly - Need for fire protection
Production time	- Longer construction time - Floor by floor construction	- High degree of prefabrication - Short construction time	- Longer construction time - Floor by floor construction	- Longer construction time - Floor by floor construction
Environ-mental impact	- Less emissions of CO_2 during manufacturing - Waste used as energy source - Recyclable	- Controlled use of energy during production and less emissions - Waste used as energy source - Recyclable in some extent	Timber – Same as alternative A FRP -Requires much energy to produce - High amount of emissions	Timber – Same as alternative A Steel – High amount of emissions of CO_2 - Recyclable and reusable
Adaptation	- Easy customization - Span length ~ 8 meters	- Limited customization -Span length 4.15 meters	- Easy customization	- Easy customization
Self-weight	- Lightweight material - Low density ~ 300-600 kg/m^3	- Lightweight material - Low density ~ 300-600 kg/m	- Density FRP ~ 1100-1300 kg/m^3	- Density steel 7800 kg/m^3

6.3 Ranking of the evaluation criteria

The evaluation of the criteria consisted of a comparison between all of the criteria to each other by a ranking system. The aim of the evaluation was to determine different weighting factors for all criteria, to determine which the most important one is. The ranking system is based on a scoring system from one to three, where the numbers are defined as follows:

1 = the criterion is less important than the other
2 = both criteria are equally important
3 = the criterion is more important than the other

Thus, the criterion with the highest total sum generates the highest weighting factor and is assumed most relevant for this case. The weighting factor was calculated by the dividing the sum of points for each criterion with the total sum of the given points. In Table 3 the results from the ranking are presented.

Table 3 - Ranking of the evaluation criteria

	Fire	**Production time**	**Environ-mental**	**Adaptation**	**Self-weight**	**SUM**	**Weight**
Fire	-	1	2	1	1	5	12.5%
Production time	3	-	2	1	1	7	17.5%
Environmental impact	2	2	-	1	1	6	15%
Adaptation	3	3	3	-	3	12	30%
Self-weight	3	3	3	1	-	10	25%
						40	100%

Adaption

The highest ranked criterion is adaption since it is important that the new floors can be adapted to the existing structural system with regard to different span lengths and planned activities. Because that the existing structural system is in steel with large spans, it is required that the new structural system can be adapted without a high degree of rebuilding.

Self-weight

Another important criterion is the self-weight due to the fact that the impact on the existing building should be minimized. A low self-weight enables the possibility of adding more floors on the existing building, since the effect of the load bearing capacity is reduced.

Production

Production time is in this case also among the high-ranked criterion since, when adding more floors, it is important that the ongoing activities in the building are not affected a longer period of time. Also, since the production in this case is located in the central part of Gothenburg a short production time is preferred to minimized the influence on the surrounding area.

Environmental impact
The environmental aspect is less important when looking at the structural system compared to if the entire building would be considered. In this case the environmental criterion was ranked as number four since timber is more or less included in all concepts. Therefore, the environmental impact will thus be lower than using other building materials.

Fire
The lowest ranked criterion is fire. Irrespective of which the most suitable concept will be, fire safety can be achieved easy in all of them.

6.4 Ranking of the concepts according to the criteria

To find the most suitable concept for adding more floors on Strömshuset, the four concepts were ranked against one another according to the different criterion. The concepts were given a score from one to five depending on how well the concept was expected to perform according to the individual criteria. A higher value means that the concept was more favorable and predicted to perform well regarding the criteria. The different scores are defined as:

1 = the concept performs very bad according to the criterion
2 = the concept performs bad according to the criterion
3 = the concept performs good according to the criterion
4 = the concept performs very good according to the criterion
5 = the concept performs excellent according to the criterion

After deciding how important the criteria are regarded the correlation with the other criteria, the given score was then multiplied with the weighting factor, developed in Table 3. This was made to give the final score for the concept due to the individual criterion. The result of the ranking of the concepts with regard to the criteria, are shown in Table 4.

The concepts in Table 4 are presented the following order:
A – Timber built on site
B – Prefabricated timber modules
C – Timber reinforced with FRP
D – Timber and steel structure

Table 4 - Result of the ranking of the concepts

	Weighting factor	A		B		C		D	
Fire	12.5%	4	0.5	4	0.5	1	0.13	2	0.25
Production time	17.5%	2	0.35	5	0.88	2	0.35	2	0.35
Environmental impact	15%	5	0.75	5	0.75	2	0.3	3	0.45
Adaptation	30%	4	1.2	2	0.6	4	1.2	4	1.2
Self-weight	25%	5	1.25	5	1.25	4	1	3	0.75
SUM	100 %	-	**4.05**	-	**3.98**	-	**2.98**	-	**3.00**

The results from the ranking process showed that concept A, timber built on site scored the highest. Motivations for the given scores are presented in the next section.

6.4.1 Motivation for the ranking process

Fire

When comparing the four concepts with regard to fire, concept A and B were assigned a score of four since timber has a more controlled and slow burning process than both steel and FRP, which concept C and D partly consist of. Therefore, the two later concepts were assigned a much lower score, even though steel and FRP can be fire protected, the burning process is faster and unexpected compared to timber.

Production time

According to the fact that the production time can be reduced up to 30 % when building with prefabricated modules compared to building at site, concept B was ranked with a score of five. Since the three other concepts intend to build at the construction site, they were ranked as a two and therefore concluded to not differ very much in production time.

Environmental impact

The motivation for assigning concept A and B with a score of five due to the environmental criterion is because the manufacturing process of timber components and modules generates less emissions, compared to steel and FRP. Among the later ones, FRP has a worse energy use and higher emissions than steel, which is in favor for concept D in a comparison between concept C and D. Another favorable aspect for concept B is that since the manufacturing process occurs mostly in the factory, the amount of emissions can be more controlled. Considering the possibility to recycle the building materials in the concepts after the lifecycle, all timber can be used as energy source. The steel can be both reused and recycled after its lifetime.

Adaption

Adding floors is, compared to building from scratch, more complex since the additional construction must be adapted to the existing elevator shafts and stairwells for example. Due to this fact, concept B is more limited because of the limitations in sizes and all modules for the different shafts must be adjusted according to the structural system of Strömshuset. Therefore, concept B was assigned a score of two. For the others, the assigned score was four since the degree of customization when building the new floors on site is higher compared to if most of the assembling occurs in a factory. Also, a system consisting of load bearing beam-column system or load bearing walls can have larger span lengths. Thus, the possibilities for adaption are higher for concept A, C and D.

Self-weight

The fifth criterion in the ranking process was self-weight. Since timber is a lightweight material the score five was assigned to concept A and B, since prefabricated modules were assumed to have the same self-weight as timber built on site. Concept C and D also consist of heavier materials, the score was lower for these concepts. Since the self-weight criterion is hard to compare, rough calculations were made for steel and timber to make the comparison as equal as possible. Since FRP is used as reinforcement it will only reduce the dimension of the timber elements. Therefore no calculations were performed. The calculations are further explained in the following chapter.

6.4.1.1 Comparison of the self-weight criteria

To obtain an as equal comparison as possible between the four concepts due to self-weight, rough calculations were performed on one column with one floor in Strömshuset. The reason for the calculations was to estimate the weight for one column in both steel and timber, and use the results to back up the scoring due to the self-weight criterion.

The used load combination in the calculation was according to Eurocode 1990 and more specific equation 6.10a, represented in equation (6.1). Two different cases were used, the first one with snow as main load and the second one with imposed load as main load.

$$Q_d = \max \begin{cases} \gamma S + \gamma \psi_0 Q_k \\ \gamma Q_k + \gamma \psi_0 S \end{cases} \tag{6.1}$$

S Snow load

Q_k Variable action

Imposed load for the first combination and snow in the second case

γ Partial safety factor for variable load. Equal 1.5 for both combinations

ψ_0 Coefficient for variable load, different for snow and imposed load

From the load combinations, the maximum load was calculated as 168.3 kN.

In the calculations, common profiles were assumed. For the timber column, glulam was assumed with strength class GL30c. For the steel column, a rectangular Swedish VKR-profile. The calculations followed the principles of Eurocode 1991. The tributary area was assumed to be equal to area for the most loaded column, which are 5.5x7.0 meters according to the drawings. In Table 5 below, the needed dimensions to handle the calculated load and total weight for the two types of columns are presented.

Table 5 - Results from estimation of the weight of one steel and one timber column for one floor in Strömshuset for a maximum load of 168.3 kN. The dimensions are taken from Swedish Wood homepage and tables from the steel manufacturer Tibnor.

Column material	Needed dimension	Total weight
Timber	140x135 mm	72.3 N/m
Steel	80x80x4 mm	92.3 N/m

The results in Table 5 are for the smallest needed dimension to resist the load. Since Strömshuset does not consist of only one floor in reality, the calculated load was assumed to be doubled to 336.6 kN. The reason was because it was assumed to be interesting to compare the total weight for larger dimensions as well. The results from the increased dimensions are presented in Table 6.

Table 6 - Results from estimation of the weight of one steel and timber column for one floor in Strömshuset for a doubled load of 336.6 kN.

Column material	Needed dimension	Total weight
Timber	165x180 mm	113.6 N/m
Steel	100x100x5 mm	144.2 N/m

The difference in total weight due to an increased load is larger, compared to the needed sizes. According to the calculations in Appendix 13, the difference in weight increase with the increase in dimensions.

6.4.1.2 Motivation for choosing the final concept

According to the result presented in Table 4, the concept that scored slightly higher was concept A with timber built on site, but the difference between concept A and B are almost negligible. The reason that A scored slightly higher than B was due to the adaption criterion, which are the highest weighted among the criteria. Due to the equal scoring, it would technically speaking, be possible to go ahead with whichever of the proposals.

For this specific case with a floor addition on Strömshuset, concept A was chosen as the concept to move forward with. The reason was because it was concluded to be more flexible to adapt to Strömshuset compared to if modules would have been chosen. Modules are most suitable for building new residential or hotels, since the modules does not have to be customized in a high degree. In this case, the modules would have to be customized in a high degree which is hard and not preferable. For example, to fit the existing elevator shafts, stairwells and shafts for installation. There might also be problematic with the differences in span-lengths in the existing building compared to the very fixed sizes of a module. Due to this, concept A was the most suitable solution for adding floors on Strömshuset.

Since the existing structural system is a beam-post system, is was decided to be suitable to design the new floors as a beam-post system as well, instead of a panel system.

If it turns out that very large dimensions have to be used for the timber construction, FRP might be used as strengthening to reduce the sizes of the elements.

7 Description of the added floors

From the evaluation of the four different concepts, the final concept for the added floors are going to be a beam-post system. In this chapter the new structure will be described and motivated according to choice of material and structural layout for the elements. Further, the assumptions made in the calculations are presented.

7.1 Material and structural system for the added floors

For the beams and columns for the new floors, glulam was chosen. The reason is because glulam provides an effective utilization of the material. Also, by using the technique of glulam with gluing lamellas to each other, the usage of material is less and the technical benefits that timber provides are optimized (Gross, 2016).

The floor elements were chosen as a cassette floor from the Swedish manufacturer Martinsons. The top flange for this floor is in cross laminated timber, CLT and the web and bottom flange are in glulam. One advantage by choosing this floor is that the ceiling can easily be attached and act as fire protection and improve the sound-proofing. Also, the floor can easily be adjusted for different flooring that are used in a residential building (Martinsons, 2006). Another advantage with this floor is that the construction time is lowered since the elements are prefabricated.

From the floor plan in Figure 9 of the existing building, there are two shafts located in the corners of the building with both an elevator and stairwell. Also, there is one single elevator. For the new floors, the two shafts in the corners are designed to reach to up all the way through the new floors. Since there is a store on the ground floor, it is not practical for the people that are going up to the new residential to cross whole ground floor to reach the single elevator. Therefore, this elevator is not designed to reach the new floors.

The shafts form the stabilizing system and the capacity of the walls should be able to resist the torsion moment for the building.

In the connection between the existing building and the added floors, the existing floor with beams casted in concrete and a thin concrete slab are kept.

The plan drawing in Figure 15 shows the load-bearing elements of the added floors. The shear walls are the thick black lines and the beams are illustrated with dashed lines. The arrows shows the span lengths of the floor structure.

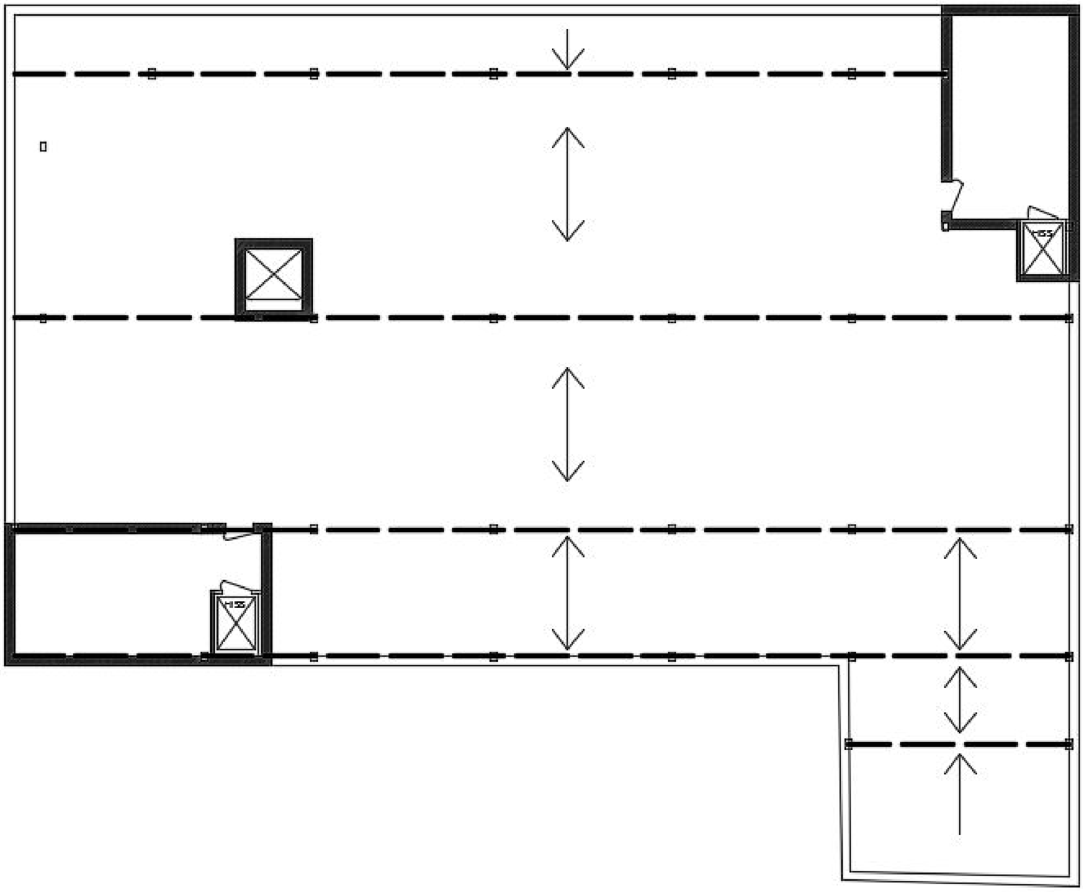

Figure 15 - The figure shows the new added floors where the shear walls, beams, columns and span of the floor are specially marked out. The shear walls are marked with thick black lines and the beams with dashed lines.

7.2 Assumptions in the calculations

When designing the new floors, some assumptions were made to simplify the calculations:

- The two top floors of Strömshuset, floor 7 and 8, were removed to simplify the calculations and the geometry of the building. Due to this, new utilization factors for the columns on the bottom floor were determined.
- The wind loads were calculated for such a case when the building stands by itself, which is not entirely true since there are other buildings in connection to Strömshuset. The reason this assumption was made, is because when floors are added to the existing building, the total height will be above the other rooftops. Thus, the calculations for the wind load are on the safe side.

Because two floors were removed, the utilization ratio for the columns were changed, since the loads from the two top floors could be subtracted. The new utilization ratio for the most loaded column was calculated to 65.6 %. The new utilization ratios are presented in Appendix 15, for the case where the two top floors were removed. The calculations in Appendix 15 are based on the results from previous investigations made by COWI.

7.3 Column based connections

In this chapter, possible details that can be used in the transition between the existing building and the new floor are presented. Due to the fact that the floor in the transition consist of the existing floor with a thin concrete slab, the studied details are timber columns with possible attachments to a concrete slab.

A column based connection can either be simply supported or fixed end. If the connection is fixed, bending moments are resisted together with the horizontal and vertical loads as opposed to a simply supported connection where no bending moment is transferred. This project will only focus on connections with fixed end.

Structural elements must be connected to each other to function as a system. A fixed connection can be connected to the concrete in different ways, some examples are listed below:

- Steel plate cast in concrete
- Steel plates welded together with a cast in steel plate
- Anchoring with expandable screws

If a timber column is placed directly on a hygroscopic material like concrete a moisture barrier has to be used to prevent damaged on the column (Crocetti, et al., 2016).

7.3.1 Nailed steel plates

The most common type of fixed connection consists of two steel plates connected to each side of the column, presented in Figure 16. The steel plates are connected to the slender side of the column by using nails or wood screws. The steel plates are then cast into the concrete or welded together with a cast in steel plate. This type of connection is suitable for a column exposed for both small and large loads (Crocetti, et al., 2016).

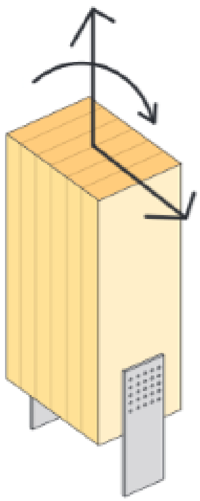

Figure 16 - Fixed connection with two steel plates connected to each side of the column (Svenskt Trä, Träguiden 2016)

7.3.2 Slotted-in steel plates

Another common type of fixed connection is slotted-in steel plates, illustrated in Figure 17. Embedding the steel plates in the timber column is beneficial both in an esthetic point of view and due to fire safety. The number of steel plates differ due to the acting loads. The steel plates are installed into the column by pre-drilled holes and the fastening by dowels. However, when the steel plates are installed in the column the cross-section is reduced and the stresses need to be checked (Crocetti, et al., 2016).

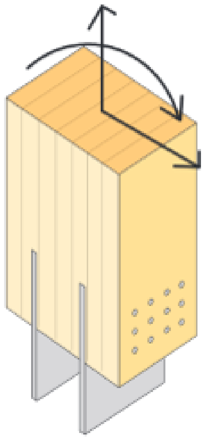

Figure 17 - Fixed connection with slotted-in steel plates (Svenskt Trä, Träguiden 2016)

The steel plates are connected to the concrete by a base plate that are attached to the steel plates and then cast in the concrete. A great benefit with slotted-in steel plates is that these type of connections are sustainable due to its ability to keep water away and therefore no extra protection is needed (Crocetti, et al., 2016).

7.3.3 Glued-in rods

Glued-in rods, illustrated in Figure 18, is another way of embedding the rods in the timber column and therefore increases the fire safety and strength of the connection when the structural system is exposed to fire. The rods are glued into pre-drilled holes of the column and the connected to a steel plate that is welded to the foundation or screwed by screws.

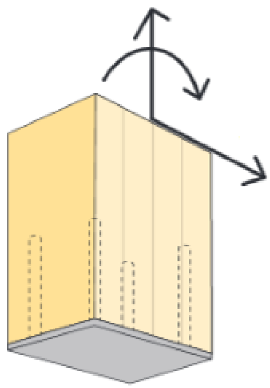

Figure 18 - Glued-in rods with a steel plate (Svenskt Trä, Träguiden 2016)

Glued-in rods is an appropriate solution for a structural system with small vertical loads and should be avoided in structures with dynamic loads (Crocetti, et al., 2016).

7.4 Loads

In this section, both the vertical and lateral loads acting on the building and how the loads are transmitted are presented.

7.4.1 Vertical loads

The vertical loads acting on a structural system in a building are permanent loads, imposed loads and snow loads. To transfer the vertical loads through the building down to the foundation the loads are transmitted to the vertical load bearing system by the horizontal load bearing system. The vertical load bearing elements are columns, load bearing walls and core while the horizontal load bearing elements consist of beams and floor slab (Andersson & Hammarberg, 2015).

The vertical loads accumulate through the building from the top of the roof structure down to the foundation. This results in that the vertical elements on the bottom floor need to be design for the total vertical load acting on all the floors in the building. Also the foundation need to be designed to carried the total vertical design loads. Since the vertical loads consist of both permanent loads and different live loads which accumulates through the building, different loads combinations need to be defined to find the design load (Andersson & Hammarberg, 2015). Figure 19 illustrate a principle sketch over a vertical load bearing system.

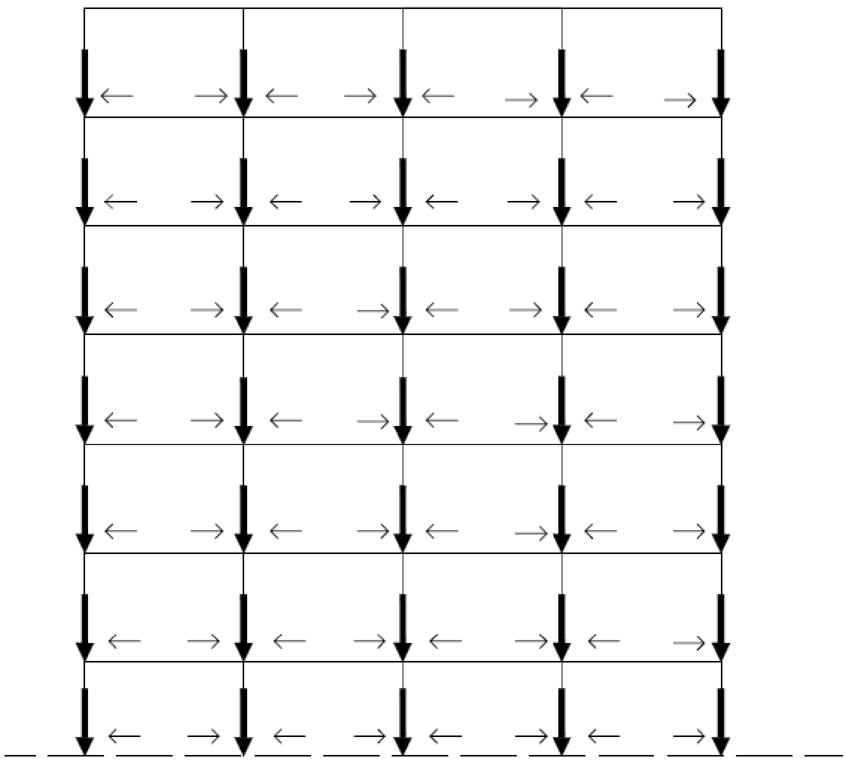

Figure 19- Principle sketch of a vertical loads system

7.4.2 Lateral loads

Lateral loads, or horizontal loads, are of great importance for the lateral stability of a structural system. The loads acts parallel to the plan of the building and are for example caused by wind loads, seismic activity or unintended inclination of the building. Therefore, it is important to make sure that the building is provided with some kind of bracing system to handle the horizontal loads both locally and globally. Three ways to stabilize the system are to use diagonal bracing, shear walls or rigid joints between the elements, which is illustrated in Figure 20 below (Crocetti, et al., 2016). In this study, the loads from seismic impact are not considered.

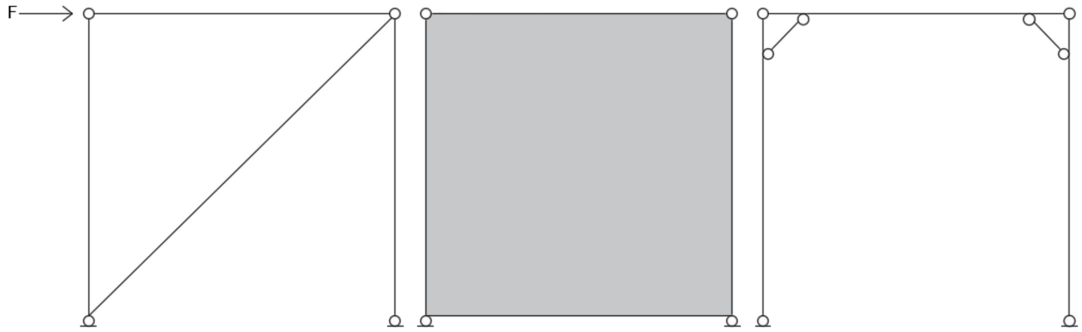

Figure 20 - The picture to the left illustrates diagonal bracing, the picture in the middle shear walls and the one to the right is one example with rigid joints

The wind pressure increases with increasing height. Therefore, the wind load affects taller building the most. The loads act on the walls and roof which then transfer them to the stabilizing members (Crocetti, et al., 2011). The wind generates both shear forces and moments in the load-bearing elements. If these cannot resist the acting load both locally and globally, the whole structure runs the risk of tilt or slide. Sliding can occur if the shear force between the foundation and building is too large (Andersson & Hammarberg, 2015). In this study the foundation was neglected and therefore sliding were not controlled. Tilting on the other hand should be controlled, both for the added floors and for the whole building with the new floors.

Horizontal loads not only cause horizontal deformations like tilting. In addition, the loads can cause torsional deformations as well. The torsional moment is resisted by the stiffness of the stabilizing elements, which examples of were illustrated in Figure 20. The closer the mass center of the building and rotational center coincide, the less is the chance of the building to collapse (Crocetti, et al., 2011).

Short periods of wind loads and machinery, for example, can cause dynamic effects in buildings in form of vibrations. These vibrations are perceived as uncomfortable for the people inside the building and are therefore taken into account during design calculations (Andersson & Hammarberg, 2015).

7.5 Design principles

In this section, the principles for the design of the new floors are presented. Also, the methods of the calculations and design criteria are presented.

7.5.1 Design of floor structure

Since the cassette floor structure consists of two different materials with different strength the effective bending stiffness was calculated. When the effective bending stiffness was determined the floor structure was controlled against compression, tension and panel stresses in the ultimate limit state. Figure 21 illustrate where the different stresses in the floor structure occur. In the middle of the two flanges the floor structure has been controlled against bending. In the transition between the web and the flanges control against panel shear has been performed and finally panel shear in the neutral axis in the web has been checked.

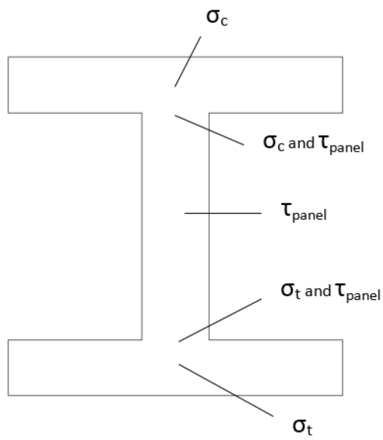

Figure 21 - Illustration of the different stresses in the floor structure that has been controlled

The following criteria have to be fulfilled in the design of the floor structure:

- $\sigma_c \leq f_{c.m.d}$
- $\sigma_t \leq f_{t.m.d}$
- $\tau_t \leq f_{t.v.d}$
- $\tau_t \leq f_{pv.d}$

The results for the calculations are presented in both Appendix 3 and 17.

7.5.1.1 Dynamic analysis of the floor structure

According to Eurocode 1995-1-1 section seven, a dynamic analysis of the floor structure should be performed. The dynamic respons of the floor is checked with calculating:

- Static deflection due to point load
- First natural frequency [Hz]
- Number of modes below 40 Hz
- Unit impulse velocity [m/s]

The first check regarding static deflection due to point load has been performed regarding equation (7.1).

$$\frac{w_{floor}}{P} \leq a \tag{7.1}$$

w_{floor} Static deflection of the beam
P Point load due to human response
a Static deflection criterion

In the next step the floor structure was controlled against velocity response according equation (7.2). The criterion for the velocity response was determined that the first natural frequency needs to be higher than 8 Hz, since it is a timber floor.

$$f_1 = \left(\frac{\pi}{2*L^2}\right) * \sqrt{\left(\frac{EI_l}{m_{floor}}\right)} \tag{7.2}$$

f_1 First natural frequency, ≥ 8 Hz
L Span length of one floor element
m_{floor} Self-weight of one floor element
EI_l Elastic modulus

The final check of the floor structure consisted of calculating the number of modes below 40 Hz, to fulfil the criterion regarding unit impulse velocity of the floor structure. The criterion due to unit impulse velocity is presented in equation (7.3).

$$v_{velocity} \leq b^{(f_1 * \xi - 1)} \tag{7.3}$$

$v_{velocity}$ Peak velocity
ξ Modal damping factor

7.5.2 Design of beams

The design calculations of the beams included control of bending resistance moment, maximum shear stress, deflections and compression perpendicular to the grain. The calculations followed the principles of Eurocode 1995-1-1. In the control of the bending resistance moment equation (7.4) were used. The criterion implicate that the bending capacity of the beam should be larger than the acting bending moment. The maximum bending moment was calculated according to the elementary case for simply supported beam and is in the middle of the span.

$$M_{Ed} \leq M_{Rd} \tag{7.4}$$

M_{Ed} Maximum bending moment
M_{Rd} Resisting bending moment

In the design of the shear stress the criterion illustrated in equation (7.5) needs to be fulfilled.

$$\tau_d \leq f_{v.g.d} \tag{7.5}$$

τ_d Maximum shear stress
$f_{v.g.d}$ Shear resistance in the beam

The next check of the beams was performed on deflection in the beam due to permanent and variable loads. The criterion for the calculation regarding deflection is stated in equation (7.6). In this project the criterion was set to 20 mm for this floor structure, which is a common criterion for residential buildings according to the manufacturer (Martinsons, 2006).

$$w_{fin} \leq 20\ mm \tag{7.6}$$

w_{fin} Final deflection of the beam

The final deflection of the beam is time depending and therefore the initial deflections for both permanent and variable loads were multiplied by a factor $1+k_{def}$. The factor $1+k_{def}$ regards the creep deflections which occurs in a timber beam over time. The value on the deformation modification factor, k_{def} depends on the service class of the beam.

The final check of the beams was control of compression perpendicular to the grain. In the design of compression perpendicular to the grain, the criterion in equation (7.7) needs to be fulfilled. The design compressive stresses in the effective contact area needs to be lower than the design compressive strength perpendicular to the grain.

$$\sigma_{c.90.d} \leq k_{c.90} * f_{c.90.d} \tag{7.7}$$

$\sigma_{c.90.d}$ Design compressive stress
$k_{c.90}$ Instability factor perpendicular to the grain
$f_{c.90.d}$ Design compressive strength perpendicular to the grain

7.5.3 Design of columns

When designing a column according to Eurocode, the column can be considered as slender or non-slender. In general when a column is exposed to vertical loads, the load effect increases due to structural deformations, so called second order effects. The design of columns in this study includes both steel and timber columns according to Eurocode 1993-1-1 for the steel columns, and Eurocode 1995-1-1 for the timber columns. The calculations are similar for both cases, and the presented formulas in this section are applied on both types of columns.

The slenderness ratio of a column is calculated according to equation (7.8).

$$\lambda_c = \frac{L_c}{\pi * i} * \sqrt{\frac{f_y}{E}} \tag{7.8}$$

L_c Buckling length of a column
i Radius of gyration
f_y Compressive strength
E Modulus of elasticity

A column should be designed with a higher load bearing capacity than the resulting axial force and thereby fulfil the criteria in equation (7.9).

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1 \tag{7.9}$$

N_{Ed} Vertical load acting on a column
$N_{b,Rd}$ Load bearing capacity of a column

The load bearing capacity, $N_{b.Rd}$ is calculated according to equation (7.10) for cross section class 1-3.

$$N_{b.Rd} = \frac{\chi * A * f_y}{\gamma_{M1}} \tag{7.10}$$

χ Reduction factor due to buckling
A Cross-sectional area of column
Υ_{M1} Partial safety factor, taken as 1.0

Where the reduction factor for a steel column is calculated using the expression stated in equation (7.11) and the relative slenderness, λ_c, is calculated according equation (7.12). The reduction factor for a timber column is calculated in a similar way and is expressed as k_c.

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda_c{}^2}} \qquad \text{where } \chi \leq 1 \tag{7.11}$$

The reduction factor due to buckling, χ, should be less or equal to one since the stresses is not allowed to exceed the compressive strength, f_y.

$$\lambda_c = \frac{\lambda}{\pi} * \sqrt{\frac{f_y}{E}} \tag{7.12}$$

λ_c Relative slenderness ratio
λ Slenderness ratio

Where the slendernaee ratio, λ, is calculated as expressed in equation (7.13).

$$\lambda = \frac{L*\beta}{i_z} \tag{7.13}$$

L Length of column
β Factor depending on boundary condition
i_z Radius of gyration in z-direction

And the factor Φ for a steel column is calculated according to equation (7.14). For a timber column the factor is expressed as k.

$$\Phi = 0.5 * [1 + \alpha * (\lambda - 0.2) + \lambda^2] \tag{7.14}$$

Where α is the buckling curve depending on the imperfection class of the steel member. The imperfection class is determined depending on several parameters, like shape of the cross section and manufacturing method. All the parameters have an effect on the buckling resistance in the column. The value of α can vary between 0.13 -0.76. A lower number of α results in a larger reduction of the capacity of the column, since the cross section contains large initial stresses. For a timber column the β-factor is used instead of α. Also, the value of 0.2 is changed to 0.3 in equation (7-14) when calculating for a timber column.

7.5.4 Design with regard to fire

In the design of a building regarding fire in Sweden there are two different structural fire requirements that needs to be fulfil. The first requirement involves that an individual load bearing structural member fulfil the requirements regarding strength. The other requirement is that the whole building needs to fulfil the ruled and regulations regarding escape routes and the risk of personal injury in case of fire. When designing a building with regard to fire the size of the building, number of floors, the prospective activities in the building and required need of protection determines the fire requirements of an individual structural member and fire class of the building (Gross, 2016).

The different fire classes that a building in Sweden are divided into are Br0, Br1, Br2 and Br3 where Br0 requires the highest requirements on a building and Br3 the lowest. Br3 is often used in the design of building with only one floor.
In the design of the load bearing system the different members should be designed to fulfil the requirements in Eurocode against collapse while exposed to fire. There are seven fire classes from A1-F which replace the previous Swedish classes I, II and III. The different classes, combined with the requirements of expansion of smoke and drop class, determines the final fire class of an individual load bearing member. Table 7 present the fire classes for a load bearing structural member (Gross, 2016).

Table 7 - The different fire classes for a load bearing structural member

Fire class	Expansion of smoke	Drop class	Previous Swedish class	Example of material
A1	-	-	Non-combustible	Concrete
A1	s1-s3	d0-d2	Non- combustible	Gypsum
B	S1-s3	d0-d2	Class I	Fire protected timber
C	S1-s3	d0-d2	Class II	Wallpaper on gypsum
D	S1-s3	d0-d2	Class III	Timber
E	-	-	Non class	Plastic
F	-	-	Non class	Not tested

Independent on what material the structural members consist of the fire technical class, for a load bearing or separation structural member, designated with R15, R30, R60, R90 or EI15, EI30, EI60, EI90. Where R stands for strength, E for integrity due to smoke and flames and I stands for insulation due to rising temperatures. The number indicate the time in minutes that a fire separation member or wall in a building needs to resist fire without lose strength and enable that the fire spreads to another part of the building (Gross, 2016).

7.5.4.1 Design principles of timber members with regard to fire

The load bearing capacity of a timber member exposed to fire is designed according to Eurocode 1995-1-2. The structural members can be designed with three different design methods:

- Reduce the cross-section due to charring
- Reduce the characteristic values with regard to strength and elastic modulus
- Advanced design methods where temperature changes and moisture content are considered

In this study, the second mentioned method has been used. In the design calculation regarding fire the design load will be reduced by a factor η that vary depending on the imposed loads, but a recommended value is 0.6 (Crocetti, et al., 2016).

The depth of charring for an unprotected timber member is calculated according equation (7.15). In the equation both fire exposure time and charring rate for the material are considered. In Figure 22 the charring depth for a timber element is illustrated.

$$d_{char.n} = \beta_n * t \qquad (7.15)$$

β_n Design charring rate [mm/min]
t Exposure time [min]

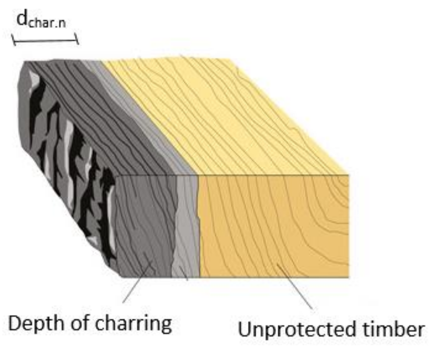

Figure 22 - Illustration of charring depth for an unprotected timber element (Svenskt Trä, Träguiden 2016)

To reduce the characteristics values due to strength and elastic module equation (7.16) and (7.17) have been used. According to Eurocode 1995-1-2, the value of 1/125 indicates that the structural member is exposed to compression stresses.

$$k_{mod} = \frac{1}{125} * \frac{P}{A} \tag{7.16}$$

k_{mod} Modification factor due to fire for compression stress
P Circumference of the reduced cross-section due to charring
A Area of the reduced cross-section due to charring

$$E = k_{mod} * k_{fi} * \frac{E_{0.05}}{\gamma_{M.fi}} \tag{7.17}$$

E Elastic modulus due to fire
k_{fi} Modification factor for timber in fire design
$E_{0.05}$ Elastic modulus parallel to the grain
$\Upsilon_{M.fi}$ Partial factor for timber exposed to fire

The design strength value of the structural timber member exposed to fire was the determined by equation (7.18).

$$f_{d.fi} = k_{mod} * k_{fi} \frac{f_{c.0.k}}{\gamma_{M.fi}} \tag{7.18}$$

$f_{c.0.k}$ Characteristic strength due to compression parallel to the grain

7.5.5 Design due to unintended inclination

In a building, the vertical elements are never exactly straight because of unintended inclination. For the global system, the horizontal component due to unintended inclination is calculated by assuming that the total vertical loads are transmitted through the columns. The total angle for inclination is the sum of the unintended inclination from all vertical elements through the whole building. The angle for an inclined system is calculated according to equation (7.19).

$$\propto_{md} = \propto_0 + \frac{\propto_d}{\sqrt{n}} \tag{7.19}$$

$\propto_{md}$	Total inclination angle
$\propto_0$	Systematic part of inclination angle
$\propto_d$	Random part of inclination angle
n	Number of supporting walls/columns in the system loaded with vertical loads

As mentioned above, the resulting vertical forces generate equivalent horizontal forces on each floor due to the unintended inclination angle, which is based on the mean angle for all the floors. The used load combinations to determine the vertical loads were according to the National Standards in Sweden for equations 6.10a and 6.10b, in Eurocode 1990. The vertical loads were calculated according to the two expressions in equation (7.20) for two different cases. The first case with an unfavorable self-weight and the second case with the self-weight as favorable.

$$V_d = \max \begin{cases} \sum_{j\geq 1} \gamma_d \gamma_{G,j} G_{k,j} + \gamma_d \gamma_{Q,j} \psi_0 Q_k \\ \sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_d \gamma_{Q,j} \psi_0 Q_k \end{cases} \tag{7.20}$$

$G_{k,j}$	Permanent actions
Q_k	Variable action Snow for the top floor and imposed load for the rest of the floors
γ_d	Partial safety factor for safety class 2 which is equal to 0.91
$\gamma_{G,j}$	Partial safety factor for permanent load. Equal 1.1 for unfavorable and 0.9 for the favorable case
$\gamma_{Q,j}$	Partial safety factor for variable load. Equal 1.5 for unfavorable and 0 for the favorable case
ψ_0	Coefficient for variable load, different for snow and imposed load

The equivalent forces are calculated according to equation (7.21).

$$H_{d.ui} = V_d * \propto_{md} * n \tag{7.21}$$

$H_{d.ui}$	Equivalent horizontal force for the specific floor
V_d	Vertical force from the specific floor

The principle for unintended inclination is illustrated in Figure 23. The illustration shows the vertical loads on each floor together with the total inclination angle, and the contribution from this to the horizontal load on the building.

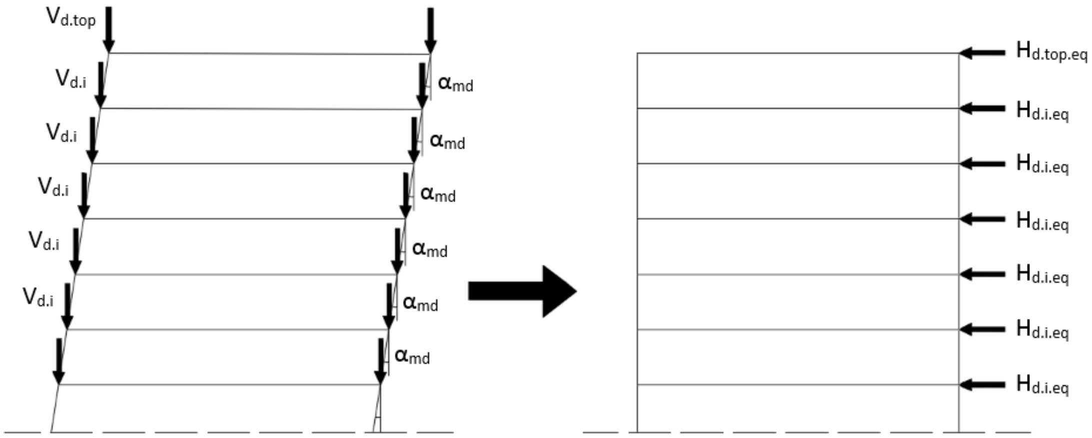

Figure 23 – Illustration of the vertical loads acting on the building and give arise to equivalent horizontal forces due to the inclination angle

7.5.6 Design due to tilting

As mentioned above, tilting of the building should be controlled. Since the additional floors are in timber and are five floors high, it was of interest to check tilting for both the whole building and the added floors. The tilting moment was therefore calculated in the bottom of the added floors and between the bottom slab and the building. The building is safe against tilting if the total moment due to horizontal loads from wind and unintended inclination are smaller than the resisting moment. The criterion that should be fulfilled is presented in equation (7.22).

$$M_{Rd} \geq M_{Ed} \tag{7.22}$$

M_{Rd} Resisting moment for the building
M_{Ed} Tilting moment from horizontal loads

In the same way as for unintended inclination, tilting was calculated for the two cases when the self-weight is unfavorable and favorable. The resisting moment is the sum of the vertical loads in the building from self-weight, imposed load and snow-load. When calculating the resisting moment, the distance to the rotation center should be taken into account, see equation (7.23). In this case, it was assumed to be the length of the façade parallel to the wind direction divided by six. The reason was because it was assumed to be on the safe side.

$$M_{Rd} = e_{RC} * \sum V_d \tag{7.23}$$

e_{RC} Distance to the rotation center

The principle for tilting is illustrated in Figure 24.

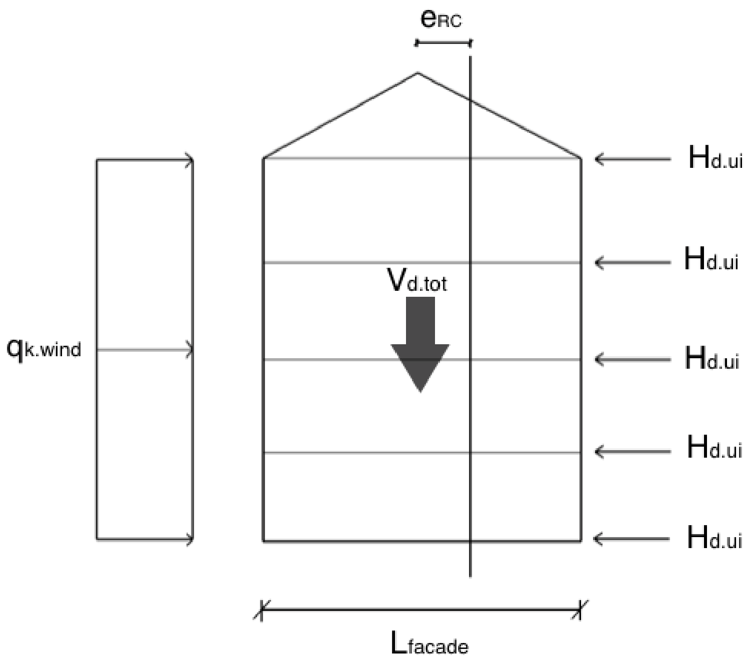

Figure 24 - Illustration of the principle of tilting

7.5.7 Design due to torsion

To ensure horizontal stability in the building, the shear walls parallel to the wind direction needs to be checked against the sum of the moment caused by wind and torsional moment. Torsion moment is the twisting of an object due to loads that occur if the shear walls are placed asymmetrically in a building.

In the design of the shear walls, the relative stiffness, *EI*, of the existing shear walls was calculated. The elastic modulus was assumed to be same for all the shear walls and the second moment of inertia depended on the geometry of each wall.

Also, the depth and position of the walls have been considered in the calculations to determine the center of rotation for the building. Figure 25 illustrates the shear walls when façade L2 is exposed to wind load. Only the walls parallel to the wind direction was considered to resist the wind loads. The reason was because the walls perpendicular to the wind direction were considered not to contribute to the resistance, since the calculated relative slenderness's were very small. The design principle was equivalent for the calculations on façade L3.

The upper left corner of the bulidng was decided to be the starting point when determining the location of the shear walls, shown in Figure 25.

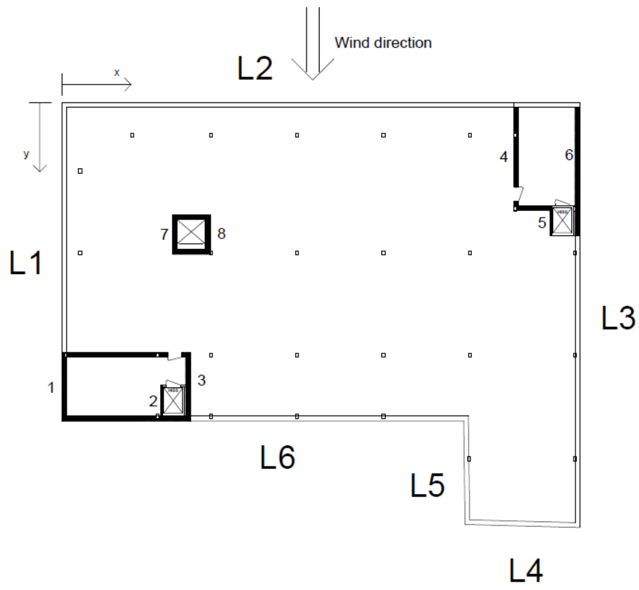

Figure 25 - Illustration of the shear walls and starting point when facade L2 is exposed to wind load

The rotation center was calculated according equation (7.24).

$$x_{rot} = \frac{EI_{tot}}{x * EI_{tot}} \tag{7.24}$$

x_{rot} Distance to rotation center from the edge in x-direction
EI_{tot} Total sum of the relative stiffness for all shear walls parallel the wind direction
x Distance of each shear wall to the edge in x-direction

The torsional moment is calculated due to the location of the shear walls relative the rotation center. How much of the total load that is resisted in each wall, depends on the position and depth. Therefore, all walls have been controlled individually due to the resisting capacity, which in turn, depends on the relative slenderness and position of the wall.

The torsional moment is summarized with the moment from the horizontal forces and then compared with the capacity of the shear walls. The criterion in equation (7.25) needs to be fulfilled.

$$M_{H+T} \leq M_{Rd} \tag{7.25}$$

M_{H+T} Total moment due to horizontal loads and torsion
M_{Rd} Resisting moment of the shear wall

7.5.8 Design of connection

The designed connection in this case is one possible connection to use in the transition between the existing floor structure and the timber columns. The connection is a mix of a nailed steel plates and bolted base plate with expander bolts into the concrete, shown in Figure 26. The design follows the principles in Eurocode 1995-1-1.

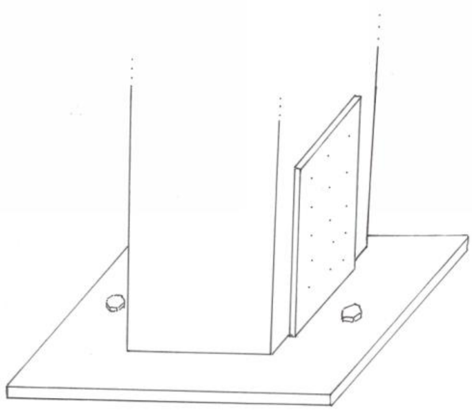

Figure 26 - Illustration of the connection in the transition. Nailed steel plates bolted with expander bolts into the concrete

The design of the nailed plates consisted of controlling the distance between the nails, the load bearing capacity of the steel plate and in the nails. Finally, the capacity due to block tearing was checked. The distance between the nails was checked according to Eurocode 1995-1-1. For the control of the load bearing capacity in the steel plate, the condition in equation (7.26) should be fulfilled.

$$F_{Ed} \leq N_{Rd} \tag{7.26}$$

F_{Ed} Resulting vertical force in the steel plate caused by the moment
N_{Rd} Capacity in the steel plate

When checking load bearing capacity of the nails, the same condition as for the steel plate in equation (7.26) above should be fulfilled. But in this case, the total capacity for all nails were defined as the sum of the capacity for one single nail and shear plane.

Block tearing, which means that the timber block surrounded by the nails breaks, were controlled as well. The design capacity was the governing value between shear and tension failure. Equation (7.27) shows the criterion that should be fulfilled due to block tearing.

$$F_{Ed} \leq N_{Rd} \tag{7.27}$$

F_{Ed} Resulting vertical force in the steel plate caused by the moment
N_{Rd} Capacity of the connection due to block tearing

For the anchoring expander bolts, the total number of bolts needed were calculated and the combined action of moment and shear were controlled. The number of bolts were calculated as the ratio between the shear force and the shear capacity of one bolt. When controlling the combined action, the criterion in equation (7.28) needed to be fulfilled. Since the bolts were assumed to be yielding, the combined action should be lower than the yield stress.

$$\sigma + \tau \leq f_y \tag{7.28}$$

σ Bending stress in the anchoring
τ Shear stress in the anchoring
f_y Yield stress for the steel bolts

8 Results from calculations

In this chapter, the results from the calculations and design of the new floors are presented. The calculated loads are presented and for design checks the utilization ratio also are presented.

8.1 Result from design of floor structure

The floor structure comprises of a cassette floors according standard dimensions from the manual Massivträ from Martinssons. The cassette floors are manufactured with a maximum span length up to 12 meters and a construction high varying between 0.3 -0.65 meters. The cassette floors consist of a cross laminated upper flanges combined with a web and bottom flanges in glulam timber, shown in Figure 27.

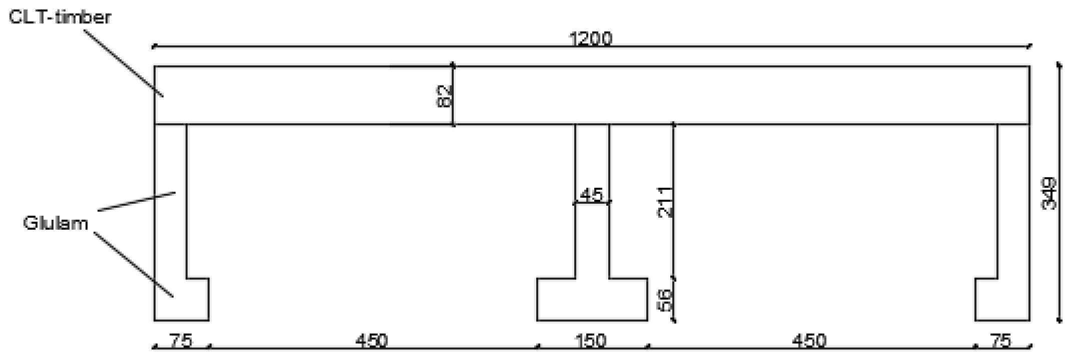

Figure 27 - Illustration of the cassette floor structure

The hight of the floor structure was determined by a standard dimension to manage the acting load. The resulting dimensions of the floor structure are presented in Figure 27, with a total hight of 349 mm and a web hight of 211 mm. The design calculations regarding the strength of the floor structure were performed according to the method described in chapter 7.5.1 and the results are presented in Table 8. The design calculations are presented in Appendix 3.

Table 8 - Results from the design calculations due to strength of the floor structure

	σ_d	τ_d	f_d	Utilization
Top flange compression	2.1 MPa	-	12.3 MPa	16.8 %
Top flange panel shear	-	0.6 MPa	2.3 MPa	25.8 %
Top web bending	2.5 MPa	-	14.2 MPa	17.7 %
Top web panel shear	-	1.1 MPa	1.5 MPa	73.7 %
Web, neutral axis, panel shear	-	0.7 MPa	1.5 MPa	43.2 %
Bottom web tension	4.2 MPa	-	9.9 MPa	42.5 %
Bottom web, panel shear	-	0.5 MPa	1.5 MPa	33.5 %
Bottom flange, panel shear	-	0.5 MPa	1.5 MPa	33.5 %
Bottom flange, tension	5.1 MPa	-	9.9 MPa	51.5 %

Another aspect that need to be in mind when designing floor structures is the extra height needed for the installations, but since the cassette floor structure is formed as a TT-section the installations can be placed in the space between the upper and bottom flange.

The cassette floor structure was design with a limit of the fundamental frequency of 8 Hz, with regard to Eurocode 1995-1-1 for timber floors. The limit for the maximum static deflection was determined to the recommended value 1.5 kN/mm

The design calculations of the floor structure regarding deflections have been made both in ULS and SLS. Regarding the manual Massivträ from Martinsson it is stated that the initial check of deflections in ULS and the final deflection in SLS are limited to maximum 20 mm respectively. The results for the calculations of deflections are presented in Table 9.

Table 9 - Results from calculations of deflections

	Bending [mm]	Shear [mm]	Total [mm]	Limit [mm]	Utilization
Deflection initial	0.013	0.002	0.015	0.020	75.8%
Deflection final	0.016	0.002	0.018	0.020	91.3%

8.1.1 Results from dynamic analysis of the floor structure

The floor structure has also been designed to fulfil the requirements regarding dynamic response, described in chapter 7.5.1.1. Table 10 shows the results from the calculations due to dynamic response of the floor structure. It turned out that the chosen hight of the floor structure on 349 mm managed all the criteria regard dynamic response.

Table 10 - Results from calculations due to dynamic response of the floor structure

	Calculated value	Criterion	
Static deflection	1.48 mm	1.50 mm	**OK**
First natural frequency	12.1	≥ 8.0 Hz	**OK**
Number of modes below 40 Hz	2.52	-	**-**
Unit impulse velocity	0.02	0.03	**OK**

8.2 Results from design of beams

The beams were designed to resist the design horizontal loads acting on the largest tributary area of a span length of 7.0 meters respectively 5.5 meters, shown in Figure 28. The beams were controlled against moment, shear force, deflections and compression perpendicular to the grain. The calculations can be found in Appendix 4.

To obtain the worst case regarding these components the beams were assumed to be simply supported. The beams were also assigned with the boundary conditions to be continuous over the span length equal to the total length of the building, illustrated in Figure 15.

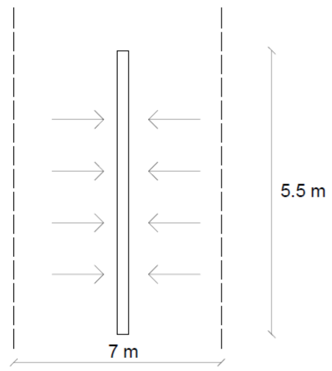

Figure 28 – Largest tributary area for the beams

The beams in the building were designed as glulam beams with the strength class GL30c exposed to long term load and in service class 2. Standard rectangular cross-section dimensions were used regarding Limträhandboken. The dimensions of the beam were calculated with regard to the principles described in chapter 7.2.5. The beams were designed to handle the design horizontal loads from permanent loads and imposed loads.

According to Eurocode, the requirement of the deflection for all the beams in the building were specified to the total span length of the beam in meters divided by 200. The results from the calculation are presented in Table 11 and the needed dimensions for the beams were 225x585 mm.

Table 11 - Results from design calculations of the beams

Moment	**Bending moment**	**Moment capacity**	**Utilization**
	114.1 kNm	221.9 kNm	51.4 %
Shear	**Shear force**	**Shear capacity**	**Utilization**
	1411.0 kN	1512.0 kN	93.3 %
Deflection	**Deflection beam**	**Limit deflection**	**Utilization**
	21 mm	28 mm	75.6 %

Also the beams were controlled against fire to fulfil the requirement of 90 minutes. The beams were assumed to be exposed against fire on three sides since the fourth and upper side of the beam is protected against fire in a different fire zone. The beam dimensions of 225x585 mm fulfilled and managed the criterion regard fire.

The final check of compression perpendicular to the grain was calculated after the design of the columns and the results are presented in chapter 8.3.1.

8.3 Results from design of columns

The columns in the building were designed to manage the total vertical loads acting on the column with the largest tributary area. Figure 29 illustrates the most loaded column on the bottom floor in the building for the new added floors, with a tributary area of 7.0 x 5.5 m^2. In the design of the columns the vertical loads were calculated from load combination 6.10a and 6.10b according National Standards in Sweden.

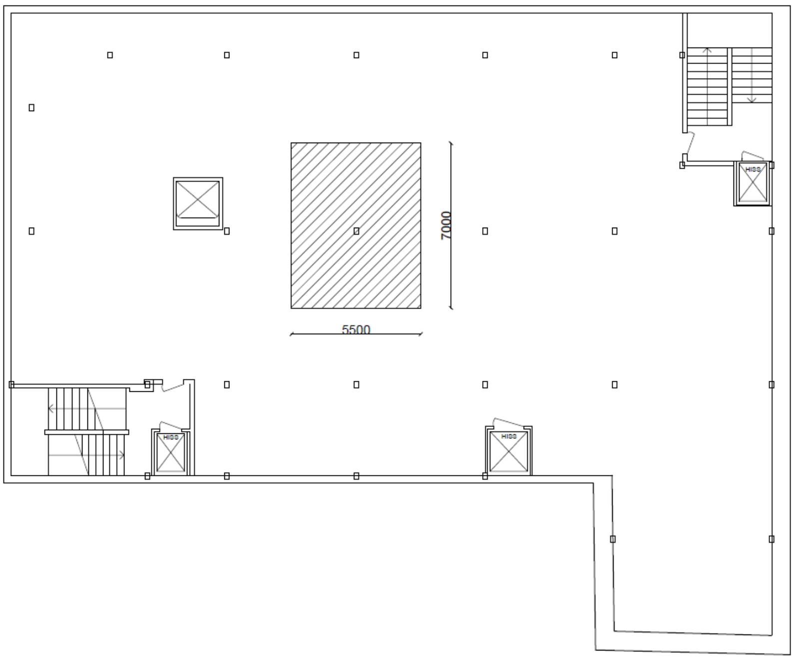

Figure 29 - The area illustrates the tributary area for the most loaded column on the bottom floor

The columns were decided to be designed in glulam with a strength class GL30c, which is a common material for vertical load bearing elements in a structural system with large vertical loads. Standard rectangular cross-section dimensions were used where the height of a lamella is 45mm. Figure 30 illustrates a general cross-section for the glulam column and also the defined slenderness directions x-and y. The columns were designed with fixed end with a combined bending moment and axial force. The columns were also assumed to be exposed to long term loads and to be in service class 2.

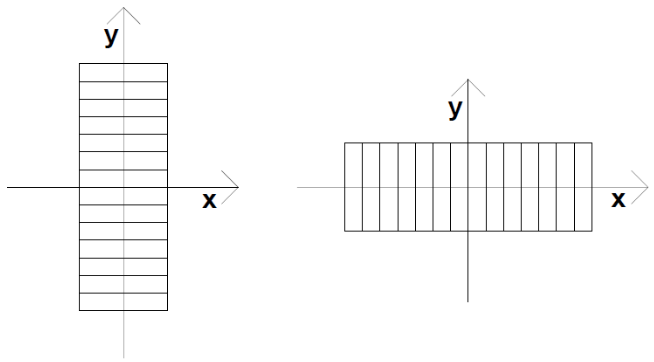

Figure 30 – Cross-section and slenderness directions of the columns

In the design calculations of the columns five different cases were analyzed depending on how many floors that were added on the existing building. The total vertical loads, due to axial force and bending moment, were determined for the different cases. The total loads differ for the different cases, since both the permanent loads and imposed loads were increased with the number of floors. The result form the calculations in Appendix 5, of the total amount of vertical loads, bending stresses and the needed dimensions for the columns are presented in Table 12.

Table 12- The needed dimensions for the columns in the different cases of floors addition

	GL30c [mm]	Vertical loads	Vertical capacity	Bending stresses	Stress capacity	Utilization
One added floor	165x180	220.5 kN	241.1 kN	7.8 MPa	19.0MPa	96.1 %
Two added floors	165x225	386.9 kN	422.2 kN	5.0 MPa	19.0MPa	99.3 %
Three added floors	190x270	553.4 kN	587.7 kN	4.4 MPa	19.0MPa	91.1 %
Four added floors	190x315	719.8 kN	729.3 kN	3.0 MPa	18.7MPa	95.6 %
Five added floors	215x360	886.2 kN	1021 kN	2.2 MPa	18.4MPa	88.5 %

The columns have also been controlled against fire to manage the fire requirement of 90 minutes according the equations described in chapter 7.5.4. The columns were assumed to be exposed to fire on all four sides and the vertical loads were reduced according to Eurocode 1995-1-2. The results from the fire calculations from Appendix 10 presented in Table 13 show that the smallest dimension of the cross-section for the columns did not fulfil the requirements against fire and had to be increased. To fulfil the requirements it turned out that the width of the two cross-sections were the weak point and had to be increased.

Table 13 - Utilization ratios of columns exposed to fire in 90 minutes

FIRE	GL30c [mm]	Vertical loads	Vertical capacity	Bending stresses	Stress capacity	Utilization
One added floor	165x225	132.3 kN	360.2 kN	7.8 MPa	15.2MPa	94.4 %
Two added floors	165x225	232.2 kN	450.3 kN	5.0 MPa	15.2MPa	88.5 %
Three added floors	190x270	332.0 kN	796.5 kN	4.4 MPa	20.8MPa	50.6 %
Four added floors	190x315	431.9 kN	1042 kN	3.0 MPa	22.6MPa	48.8 %
Five added floors	215x360	531.7 kN	1269 kN	2.2 MPa	23.6MPa	38.5 %

8.3.1 Compression perpendicular to the grain

In the check regard compression perpendicular to the grain the design compressive stresses in the effective contact area were compared to the design compressive strength perpendicular to the grain. The check was performed for the different cases where one to five floors were added and the results are presented in Table 14.

Table 14- Results from control of compression perpendicular to the grain

	GL30c [mm]	$\sigma_{c.90.d}$	$f_{c.90.d}$	Utilization
One added floor	165x180	2.8 MPa	1.4 MPa	199.6 %
Two added floors	165x225	2.2 MPa	1.4 MPa	159.7 %
Three added floors	190x270	1.6 MPa	1.4 MPa	115.5 %
Four added floors	190x315	1.4 MPa	1.4 MPa	99.0 %
Five added floors	215x360	1.1 MPa	1.4 MPa	76.6 %

As the results show in Table 14 there are only the two larger dimensions of the columns, for four and five added floors, that managed to resist the compression perpendicular to the grain. How to solve this are further discussed in chapter 9.

8.4 Results from unintended inclination

The calculations of the unintended inclination were performed for two different cases. In the first case, the vertical loads for the new added floors were calculated. In the second case, the vertical loads for the existing building.
At first, the needed loads and areas were calculated and put together in the load combination (7.20) to determine the vertical loads on each floor. The vertical loads were the same for all the floors in the existing building. In the added floors there were some differences for the top floor and in the transition between the new and existing floors. Thus, the equivalent horizontal forces were different for these locations as well. The calculated horizontal forces are presented in Table 15 below, for the two cases with self-weight unfavorable and favorable.

Table 15 - Equivalent horizontal loads for the two load-cases

Horizontal load on each floor	Self-weight unfavorable	Self-weight favorable
Top floor	5.1 kN	1.3 kN
Floor 8-11	12.8 kN	5.2 kN
Floor 7	27.5 kN	18.4 kN
Floor 1-6	36.2 kN	23.1 kN

The equivalent horizontal forces on floor 1-6 are much larger than in the added floor. The reason is because timber is used instead of steel, which lowers the self-weight and thus the horizontal forces become lower. Another thing that can be stated is that the contribution when the self-weight is unfavorable is larger than when it is favorable.

8.5 Results from tilting

As mentioned above tilting was checked for both the added floors and the whole building and therefore the results for these both cases are presented separately. The calculations started with determining the moment from unintended inclination and wind loads. The resulting moment from these were then added to a total moment. In the next step, the resisting moment was calculated. Finally, the moments were compared and a utilization ratio was calculated.
The calculations were then repeated for both load cases and for both the façades that are exposed to wind loads. Table 16 below presents the results for tilting of the new added floors.

Table 16 - Results from tilting for the new added floors for both facades and for the two load cases

Façade L2	Tilting moment	Resisting moment	Utilization ratio
Self-weight unfavorable	27.5 MNm	77.3 MNm	35.5 %
Self-weight favorable	27.1 MNm	37.3 MNm	72.6 %
Façade L3	**Tilting moment**	**Resisting moment**	**Utilization ratio**
Self-weight unfavorable	25.3 MNm	93.9 MNm	27.0 %
Self-weight favorable	25.0 MNm	45.3 MNm	55.2 %

Table 17 shows the results for checking tilting for the whole building.

Table 17 - Results from tilting for the whole building for both facades and for the two load cases

Façade L2	Tilting moment	Resisting moment	Utilization ratio
Self-weight unfavorable	112.2 MNm	278.5 MNm	40.3 %
Self-weight favorable	111.2 MNm	165.6 MNm	67.1 %
Façade L3	**Tilting moment**	**Resisting moment**	**Utilization ratio**
Self-weight unfavorable	891.7 MNm	338.0 MNm	26.4 %
Self-weight favorable	881.5 MNm	201.0 MNm	43.9 %

8.6 Results from torsion

The calculations of the torsion moment were at first performed for the building with five added floors and façade L2 was exposed to wind loads. If it turned out that the shear walls could not resist the torsion moment the calculations were repeated for less floors. The principle was the same for façade L3.

The design calculations can be found in Appendix 11 and are calculated regard the principle described in chapter 7.5.7.

Figure 31 illustrate the shear walls for the new added floors when façade L2 was exposed to wind load as well as the calculated rotation center. The location of the rotation centre was calculated to 26.35 meter from the starting point in x-direction and 10.15 meter from the starting point in y-direction.

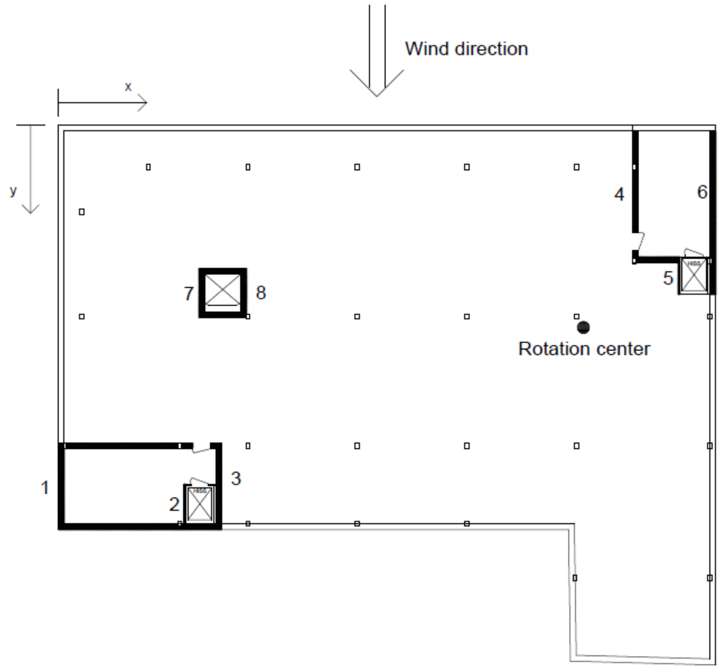

Figure 31 - Illustration of the shear walls when wind acting on facade L2 and also the calculated rotation center

The calculations started with determining the torsion moment caused by asymmetrically in the building. The calculated torsion moment was then added to the wind load. In the next step, the resisting moment was calculated. Then the total calculated moment due to wind was compared to the resisting capacity and the utilization ratio was calculated.

Table 18 presents the results from Appendix 18 for the case when five new floors were added on the existing building and façade L2 was exposed to wind load.

Table 18 - Results from the control of capacity in shear walls when five floors were added and façade L2 was exposed to wind load

Shear wall	Total wind moment	Resisting capacity	Utilization
1	21.2 MNm	2.2 MNm	966.2 %
2	2.1 MNm	0.5 MNm	423.5 %
3	16.8 MNm	2.2 MNm	764.1 %
4	21.5 MNm	5.7 MNm	381.2 %
5	0.5 MNm	0.5 MNm	99.5 %
6	19.0 MNm	9.3 MNm	203.5 %
7	3.9 MNm	0.8 MNm	504.5 %
8	3.7 MNm	0.8 MNm	470.9 %

As Table 18 shows only shear wall 5 managed to resist the wind load. The rest of the walls are exposed to much larger wind loads than the resisting capacity. Therefore, new calculations regard the control of the capacity in shear walls were made for a case when three new floors were added. The calculations can be found in Appendix 18. Table 19 below shows the results of the calculated wind moment, resisting capacity and the utilization ratio, for each shear wall when three floors were added.

Table 19 - Results from the control of capacity in shear walls when three floors were added and façade L2 was exposed to wind load

Shear wall	Total wind moment	Resisting capacity	Utilization
1	13.0 MNm	1.8 MNm	724.5 %
2	1.3 MNm	0.4 MNm	321.8 %
3	10.3 MNm	1.8 MNm	572.8 %
4	13.4 MNm	4.6 MNm	289.4 %
5	0.3 MNm	0.4 MNm	75.5 %
6	1.8 MNm	7.7 MNm	152.8 %
7	1.0 MNm	0.6 MNm	379.6 %
8	0.9 MNm	0.6 MNm	354.3 %

Also in this case, only shear wall 5 managed to resist the wind load.

The design of capacity of shear walls were also calculated for the case when façade L3 is exposed to wind load. Figure 32 shows the shear walls, number 9-14, and the calculated rotation center when façade L3 is exposed to wind. The rotation center is located 16.72 meter from the starting point in y-direction and 16.75 meter from the starting point in x-direction. The starting point is located at the same point if five or three floors were added.

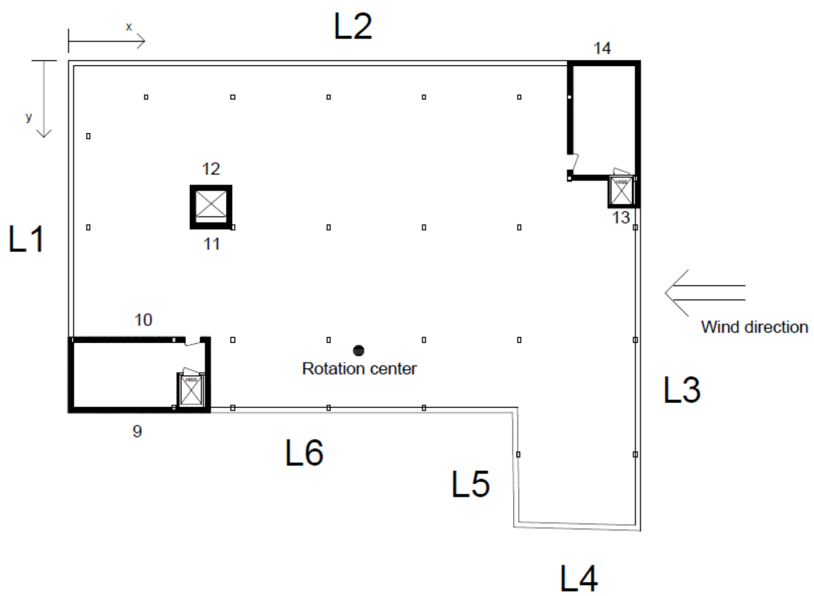

Figure 32 - Illustration of the shear walls when wind acting on facade L2 and also the calculated rotation center

Table 20 shows the results from the calculations according to control of capacity in shear walls when five floors were added and façade L3 was exposed to wind load. As Table 20 shows only shear wall 13 managed to resist the horizontal loads. The rest of the walls are exposed too much larger horizontal loads than the resisting capacity.

Table 20- Results from the control of capacity in shear walls when five floors were added and facade L3 was exposed to wind load

Shear wall	Total wind moment	Resisting capacity	Utilization
9	33.4 MNm	9.1 MNm	368.7 %
10	33.4 MNm	9.1 MNm	369.1 %
11	0.9 MNm	0.7 MNm	118.6 %
12	0.9 MNm	0.7 MNm	118.6 %
13	0.4 MNm	0.4 MNm	94.9 %
14	4.5 MNm	2.3 MNm	195.5 %

Table 21 shows the results from checking capacity of shear walls when three floors were added and façade L3 was exposed to wind load. When only three floors were added and the wind load acted on façade L3. It turned out that shear wall 11, 12 and 13 managed to resist the horizontal loads. The rest of the walls are exposed to larger wind loads than the resisting capacity.

Table 21 - Results from the control of capacity in shear walls when three floors were added and facade L3 was exposed to wind load

Shear wall	Total wind moment	Resisting capacity	Utilization
9	21.8 MNm	7.4 MNm	294.0 %
10	21.8 MNm	7.4 MNm	294.4 %
11	0.6 MNm	0.6 MNm	94.7 %
12	0.6 MNm	0.6 MNm	94.7 %
13	0.3 MNm	0.4 MNm	75.7 %
14	2.9 MNm	1.9 MNm	156.1 %

8.7 Results from design of connection

For the nailed steel plate, the nails were assumed to be quadratic and grooved. The required dimensions for the plate and loads due to tension and shear are shown in Figure 33.

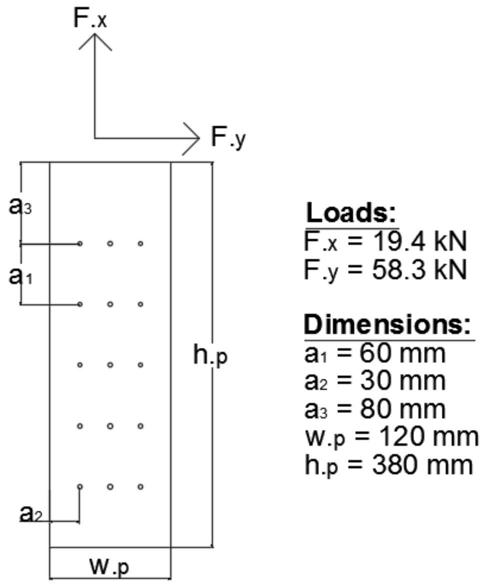

Figure 33 - Needed dimensions and loads due to tension and shear for the nailed steel plate

The needed thickness of the steel plate was calculated to 5 mm and the diameter of the nails to 4 mm. The results from the control of the capacity for the steel plate, the nails and block tearing are presented in Table 22.

Table 22 - Results from design of nailed steel plate

Check	Load	Capacity	Utilization ratio
Steel plate	19.4 kN	121.1 kN	16.0 %
Nails	19.4 kN	19.7 kN	98.4 %
Block tearing	19.4 kN	34.7 kN	56.0 %

For the anchoring to the concrete slab, a HST expander bolt from Hilti was chosen according to the Anchoring Fastening Technology Manual. The expander bolt is shown in Figure 34. From the calculations it can be stated that two bolts are needed of the type M10. The diameter of each bolt is 10 mm and the anchoring length into the concrete should be at least 90 mm.

Figure 34 - The chosen expander bolt from Hilti should be with diameter 10 mm and anchoring length 90 mm

From the calculations of combined action from moment and shear in the anchoring, it was assumed that the bolts were yielding. The total stress from both actions was calculated to 93.2 MPa. Comparing this to the yield stress for steel S235, the utilization ratio was 39.7 %. Also, the needed thickness of the steel plate was calculated to 5mm.

9 Discussion

In some existing buildings there might be extra capacity in the structural system to utilize, which was the case for the reference building in this project. One difficulty with adding floors can be if the owner of a property does not want to extend even though the possibility exists. But, due to the amendment current three-dimension property, this obstacle may be overcome and might open up the possibility for more projects of floor additions.

From the evaluation of the different concepts, the chosen structural system for the added floors was a timber beam-post system built on site. According to the motivation in chapter 6.4.1.2, the concept with prefabricated modules could have been used as well. The beam-post system was chosen mainly since it was concluded to be more suitable for Strömshuset and due to the fact that modules are more difficult to adapt. It should also be mentioned that all concepts could have been used, but for the five stated criteria in this project, the chosen concept was more favorable due to the low self-weight and easy adaption.

Since two floors were removed there was a high degree of remaining capacity in the existing structural system. Due to this, it was assumed in the beginning of the project that the horizontal loads were going to limit the number of floors, instead of the vertical loads. According to the results in the calculations, this was confirmed.

From the calculations it turned out that due to the asymmetry of the building, the new floors could not resist the torsional moment from either five or three floors. To be able to add at least three floors, extra number of shear walls need to be added compared to the number of shear walls that were identified in the old drawings. In the reference building there are probably more shear walls than the identified ones in the stairwells and elevator shafts. It should also be mentioned that the calculations of the horizontal loads have been performed at Strömshuset as a single building instead of taking the nearby buildings into account. This resulted in higher loads than in the reality and calculations on the safe side.

One problem when designing the beams was the compression perpendicular to the grain, where the utilization ratios were too large for the three smallest cross-sectional dimensions. One way to lower the compression force could be to use a similar solution as the reinforcing principle presented in Figure 13. Over the support, a steel or FRP plate might be use to spread out the pressure and thereby lower the compression force between the timber column and beam. Thus, the largest dimensions might not be necessary to use if one to three floors are added.

There are some parameters that might cause uncertainties in the results of the calculations. The existing drawings of Strömshuset were hard to interpret and some necessary information was not always stated. Therefore, assumptions were made regarding steel and concrete quality, which might influence the results to some extent.

9.1 Connection in the transition

The chosen connection in the transition between the reference building and added floors consist of a nailed steel plate welded to a base plate, which in turn is bolted to the concrete slab with expander bolts.

The connection in the transition was calculated with fixed end, which mean that moments are transferred. Therefore, the columns can help to resist the torsional moment. Since the loads are transferred to the stiffest elements the contribution from the columns might be low and were not considered in the calculations of torsion.

The anchoring lengths of the bolts turned out to be longer than the thickness of the concrete slab. Casting concrete heels at the positions of the columns to fit the expander bolts can be one solution to solve this. One problem with this solution might be the contact area between the heel and the slab, since it is important to achieve a sufficient attachment. Another solution might be to cast a new thicker concrete slab on top of the existing slab. In this case the attachment problem might still remain. Therefore, a more suitable solution can be to replace the existing slab with a new one, to better ensure enough attachment, but this solution may be more expensive.

Another possible design of the connection in the transition could be to cast the base plate into the slab, by cutting out the parts of the slab where the columns are placed. Then the connection would be another variety of a nailed steel plate. In this case the expander bolts would not be needed, but the problem with the attachment to the existing concrete slab would still exist.

A connection of the type slotted-in steel plates might also be a suitable solution. One advantage by using this connection would have been that fire protection is achieved since the surrounding timber covers the steel. Also in this case, the steel plates would probably require an anchoring length longer than the thickness of the existing slab.

For the case of adding five floors, the magnitude of the loads in the connection are large. Therefore glued-in rods were not considered since it is a more appropriate solution for a structural system with small vertical loads.

Since the drawings of the reference building is old and not updated, it is difficult to fully understand how the floor structure is structured in reality. Therefore, the connection might have to be changeed in design to suit the floor structure in the best possible way. It is also important to mention that the suggested connections might not be the final design.

10 Conclusions

The aim of the study was to find the most suitable concept to perform an addition of floors in timber for residences, on an existing multi-activity building in the central part of Gothenburg. Further, a principle design of the identified concept was made.

The objectives were to identify and suggest suitable details in the transition between the existing building and added floors. Also, to determine what might limit the total number of added floors.

- The most suitable concept to add floors on the multi-activity building, Strömshuset, is a beam-post system in timber built on site. The chosen concept was favorable due to the low self-weight and the adaptability of the system.

- When adding floors it is appropriate to perform a principle design since these type of projects depend on the existing building. Both the structural system and remaining capacity in the existing building differ for each case. Therefore, the most suitable building method depends on the individual building.

- The existing load-bearing structure can resist the additional vertical loads and the structure is not in risk of tilting when five new timber floors were added. The added floors can withstand the tilting moment in the transition between the reference building and added floors as well.

- The beams need to be strengthened in the contact area between the columns and beams, for example with a steel or FRP plate to lower the compressional force perpendicular to the grain in the beam. Otherwise, larger dimensions of the columns have to be used even if one to three floors are added. The dimensions for four and five added floors managed the criterion.

- Due to the asymmetry of the building, the limiting action turned out to be torsional moment. Even an addition of three floors were too much to handle and it can be concluded that more shear walls need to be added. To achieve a more realistic results the columns need to be accounted for, but since the utilization ratios due to torsion were very high it was concluded that more shear walls might still be needed. Therefore, a deeper analysis is suggested.

- If the existing concrete slab is retained as the top floor, connections in the transition will be problematic. One problem is to achieve a sufficient attachment between the old existing slab and the new casted concrete. Another problem if expander bolts are used is the anchoring length which is too long.

10.1 Recommendations of further investigations

In the list below, recommendations of further studies and investigations concerning the possibility to add more floors on an existing building are listed.

- If timber is to be used as a structural system, a detailed design of the needed shear walls should be performed. The number of needed shear walls or bracing units as well as their position need to be determined. Also, the contribution from the columns can be considered in the design of shear walls.

- Since the existing concrete slab consist of old concrete with other characteristics than the concrete that is used today it is of interest to investigate the possible problem with attachment deeper. To investigate the impact of the different properties and how it might affect the strength in the connections.

- Since the evaluation is based on only five criteria it might be interesting to perform a preliminary design on the other concepts as well. Then, a deeper comparison between the concepts can be made according to the calculations.

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Table of content for the Appendix

Appendix 17 – Control of capacity in cassette floor

Appendix 18 – Control of capacity in shear walls

Appendix 1 - Input data for the existing building

1.1 Geometry of the existing building

The lengts of the facades in Strömshuset is illustrated in the picture.

$L_1 := 20.3m$

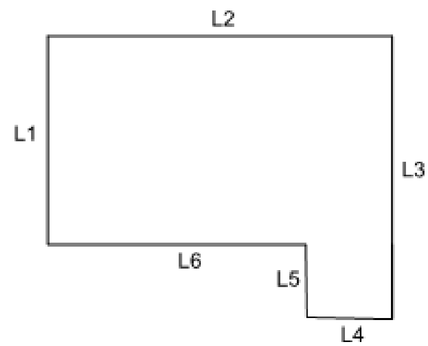
$L_2 := 33.5m$

$L_3 := 27.6m$

$L_4 := 8.3m$

$L_5 := 7.1m$

$L_6 := 25.1m$



The heights of the elements in Strömshuset are presented below. In the picture below, both the total height including the basement and roof are illustrated as well as the height above the ground.

$h_{floor} := 0.4m$ Height of floor structure

$h_{base.slab} := 0.5m$ Height of the bottom floor

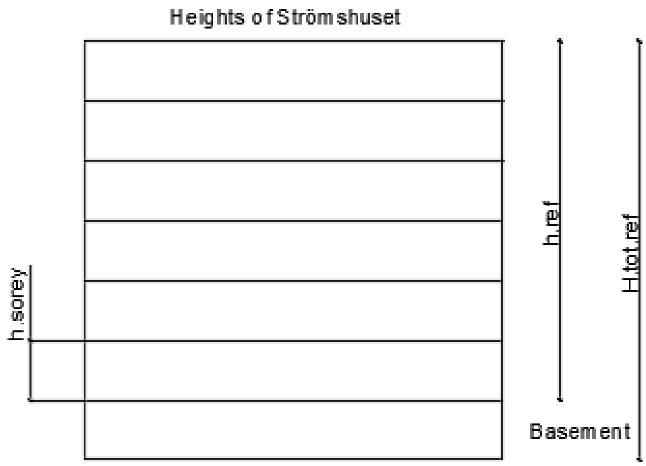
$h_{storey} := 2.8m$ Height of each floor

$h_{roof} := 0.4m$ Height of roof

$H_{tot.ref} := 7h_{floor} + h_{base.slab} + 6h_{storey} + h_{roof} = 20.5\,m$ Total height of the buidling

$h_{ref} := 6h_{floor} + 6.h_{storey} = 19.2\,m$ Height of the building above the ground floor

$L_{column} := h_{storey} = 2.8\,m$ Height of a column. Assumed to be the same as the height of one storey



1.2 Self -weight for the different elements

1.2.1 Floor structure

The floor structure in the basement consist of a concrete slab. In the storeys above the basement, the floor structure consist of steel beams casted in concrete. On top of these, a layer of concrete are casted and covered with a timber flooring.

$G_{k.c.floor} := 1.92\frac{kN}{m^2}$ Concrete cover of slab, 80mm

$G_{k.t.floor} := 0.3\frac{kN}{m^2}$ Timber floor covering

$G_{k.s.floor} := 3\frac{kN}{m^2}$ Steel beams casted in concrete

$G_{k.slab} := 12\frac{kN}{m^2}$ Base slab

$G_{k.haunch} := 6\frac{kN}{m}$ Base haunch

1.2.2 Steel colums

$m_{column} := 112.7\frac{kg}{m}$ The mass of one steel column

$G_{k.column} := g\cdot m_{column} = 1.105\cdot\frac{kN}{m}$ Self-weight of one column

1.2.3 Partition walls

$G_{k.div.wall} := 0.4 \frac{kN}{m^2}$ Partition walls

$G_{k.inst} := 0.3 \frac{kN}{m^2}$ Installations

1.2.4 External wall

$G_{k.ew} := 10.7 \frac{kN}{m}$ External walls (concrete and windows), per floor

$G_{k.con} := 22.6 \frac{kN}{m}$ Concrete wall in basement

1.2.5 Roof structure

$G_{k.roof} := 0.3 \frac{kN}{m^2}$ Roof, cellular plastic and installations

1.3 Material data

1.3.1 Steel

$E := 210GPa$ Modulus of elasticity for steel

$\gamma_{M1} := 1$ Partitial factor

$f_y := 235MPa$ Steel quality for S235

Appendix 2 - Capacity of the columns in the existing building

2.1 Capacity due buckling

2.1.1 Input data - geometry of the clumns

The input data below are taken from Table II:10 from Stålbyggnadsinstitutet, where the indata of the old DIP steel profiles are presented.

$A_{column.28} := 153.58cm^2$ Cross-sectional area for column DIP28

$A_{column.26} := 120.72cm^2$ Cross-sectional area for column DIP26

$A_{column.24} := 111.32cm^2$ Cross-sectional area for column DIP24

$A_{column.22} := 91.13cm^2$ Cross-sectional area for column DIP22

$h_{column} := 280mm$ Height of the column

$b_{column} := 280mm$ Width of the flange

$t_{flange} := 20mm$ Thickess of the flange

$i_z := 71.5mm$ Radius of gyration i z-direction

$i_y := 120.1mm$ Radius of gyration i y-direction

2.1.2 Buckling curve

$$\frac{h_{column}}{b_{column}} = 1$$

$$buckling_{curve} := \left| \begin{array}{ll} \text{"Curve c"} & \text{if } t_{flange} \le 100mm \\ \text{"Curve d"} & \text{otherwise} \end{array} \right. = \text{"Curve c"}$$

From table 6.1 EC
Since steel quality S235 and buckling around z-direction

$buckling_{curve} = \text{"Curve c"}$

$\alpha := 0.49$ Imperfection factor, EC 1993-1-1 Table 6.3

2.1.3 Buckling lenght

$\beta := 1$ Factor depending on boundary condition
Simply supported in both ends

$L_{column} = 2.8\,m$ Length of column

$L_c := \beta \cdot L_{column} = 2.8\,m$ Buckling length for the column, assume simply supported

2.1.4 Calculation of capacity

$$\boxed{N_{b.Rd} := \frac{\chi \cdot A_{column} \cdot f_y}{\gamma_{M1}}}$$

Formula for calculate the capacity of a column. EC 19931-1 Eq 6.47

$$\lambda := \frac{L_c}{i_z} = 39.161$$

Slenderness ratio, EC 1993-1-1 Eq 6.50

$$\lambda_c := \frac{\lambda}{\pi} \cdot \sqrt{\frac{f_y}{E}} = 0.417$$

Relative slenderness ratio. EC 1993-1-1 Eq 6.50

$$\Phi := 0.5 \cdot \left[1 + \alpha \cdot \left(\lambda_c - 0.2\right) + {\lambda_c}^2\right] = 0.64$$

$$\chi := \frac{1}{\Phi + \sqrt{\Phi^2 - {\lambda_c}^2}} = 0.888$$

Reduction factor due to buckling. EC 1993-1-1, Eq 6.49

$$N_{b.Rd.28} := \frac{\chi \cdot A_{column.28} \cdot f_y}{\gamma_{M1}} = 3.206 \cdot MN$$

Capacity of column DIP28

$$N_{b.Rd.26} := \frac{\chi \cdot A_{column.26} \cdot f_y}{\gamma_{M1}} = 2.52 \cdot MN$$

Capacity of column DIP26

$$N_{b.Rd.24} := \frac{\chi \cdot A_{column.24} \cdot f_y}{\gamma_{M1}} = 2.324 \cdot MN$$

Capacity of column DIP24

$$N_{b.Rd.22} := \frac{\chi \cdot A_{column.22} \cdot f_y}{\gamma_{M1}} = 1.902 \cdot MN$$

Capacity of column DIP22

The utilization ratio for each column was calculated in Appendix 14, according to the following equation. The x-notation depends on which column type that was calculated for.

$$\boxed{\frac{N_{Ed.}}{N_{b.Rd.xx}} \le 1}$$

Appendix 3 - Design of the timber floor structure

In this appendix, the floor structure for the new added floors are designed according to sternght and dynamic response. In this appendix, the equations from the results in Appendix 17 are presented.
The floor are chosen from the manufacturer Martinsons in Sweden and therefore the self-weight of the floor are know. The floor structure is a cassette floor where the top flange consist of cross-laminated timber (CLT) and the web and bottom flange of glulam (GL30c).

3.1 Geometry of the floor structure

The dimensions for the parts of the floor are listed below and illustrated in the Figure.

$b_{f.floor} := 7.5m$ Largest span of one floor element

$l_{f.floor} := 5.5m$ Largest width of the floor structural

$t_{tf.floor} := 82mm$ Thickness of top flange

$l_{top.floor} := 600mm$ Length of the top flange

$t_{web.floor} := 45mm$ Thickness of web

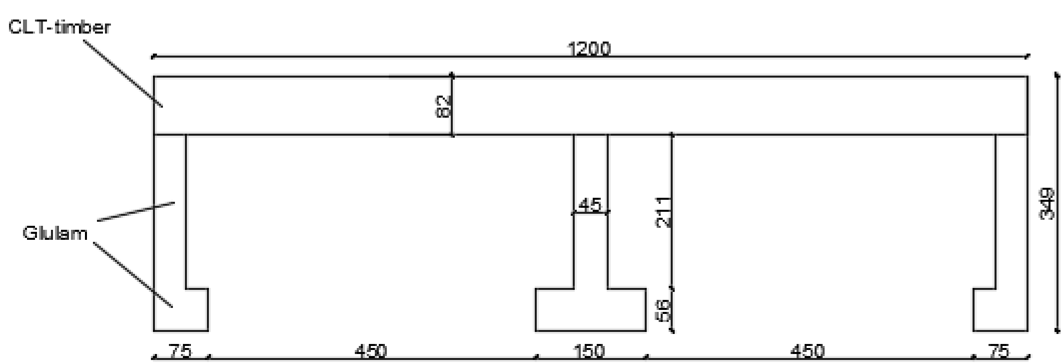
$h_{web.floor} := 211mm$ Heigth of the web

$t_{bf.floor} := 56mm$ Thickness of bottom flange

$h_{bf.floor} := 150mm$ Width of the bottom flange

$h_{floor.cassette} := t_{tf.floor} + h_{web.floor} + t_{bf.floor} = 0.349\,m$ Total height of the floor structure

$s_{floor} := 0.6m$ Space between the webs, which is equal the width of one floor element



$A_0 := 10 \cdot m^2$

$A_{trib} := 5.5m \cdot 7m = 38.5\,m^2$ The largest tributary area

3.2 Loads acting on the floor structure

The loads on the floor are both due to self-weights and due to imposed load for a residental building.

3.2.1 Load from self-weight

$g_{k.floor.self} := 63\frac{kg}{m^2}\cdot g = 0.618\cdot\frac{kN}{m^2}$ Self-weight of the floor structure given from the manufacturer Martinsons

$g_{k.in.wall} := 0.5\frac{kN}{m^2}$ Assumed weight of the inner walls and installations

$g_{k.floor} := g_{k.floor.self} + g_{k.in.wall} = 1.118\cdot\frac{kN}{m^2}$ Total self-weight of the floors structure

$G_{k.floor} := g_{k.floor}\cdot s_{floor} = 0.671\cdot\frac{kN}{m}$ Self-weight of the floor due to distance between the webs

3.2.2 Load from imposed load

$q_{k.imp} := 2.0\frac{kN}{m^2}$ Imposed load for residental building EC 1991-1-1, Table 6.2

$Q_{k.imp} := q_{k.imp}\cdot s_{floor} = 1.2\cdot\frac{kN}{m}$ Imposed load in the floor due to distance between the webs

$\psi_{0.i} := 0.7$ Combination coefficient for variable loads

$\alpha_A := \frac{5}{7}\cdot\psi_{0.i} + \frac{A_0}{A_{trib}} = 0.76$ Reduction factor due to variable load - imposed load

3.2.3 Load combination - imposed load as main load

The used load combinations in ULS are according to the National Standards in Sweden.

$\gamma_d := 0.91$ Partial coefficient for safety class 2

ULS

$Q_{floor.ULS.a} := \gamma_d\cdot 1.35\cdot G_{k.floor} + \gamma_d\cdot 1.5\cdot Q_{k.imp}\cdot\alpha_A = 2.068\cdot\frac{kN}{m}$ Eq 6.10a

$Q_{floor.ULS.b} := 0.89\gamma_d\cdot 1.35\cdot G_{k.floor} + \gamma_d\cdot 1.5\cdot Q_{k.imp}\cdot\alpha_A = 1.978\cdot\frac{kN}{m}$ Eq 6.10b

SLS

$Q_{floor.SLS.a} := 1.0\cdot G_{k.floor} + 1.0\cdot Q_{k.imp} = 1.871\cdot\frac{kN}{m}$

***3.2.4 Maximum* shear force and bending moment in the floor strucutre**

ULS

$$V_{floor.ULS} := Q_{floor.ULS.a} \cdot \frac{b_{f.floor}}{2} = 7.756 \cdot kN$$

Maximum shear force in the floor structure in ULS

$$M_{floor.ULS} := \frac{Q_{floor.ULS.a} \cdot b_{f.floor}^{2}}{8} = 14.543 \cdot kN \cdot m$$

Maximum moment in the floor structure in ULS

SLS

$$V_{floor.SLS} := Q_{floor.SLS.a} \cdot \frac{b_{f.floor}}{2} = 7.015 \cdot kN$$

Maximum shear force in the floor structure in SLS

$$M_{floor.SLS} := \frac{Q_{floor.SLS.a} \cdot b_{f.floor}^{2}}{8} = 13.153 \cdot kN \cdot m$$

Maximum moment in the floor structure in SLS

3.3 Design material parameters

The material parameters that are used in the calculations below are taken from the "Handbok i KL-trä" from Martinsons for the CLT.

3.3.1 Cross laminated timber

$$k_{mod.CLT} := 0.7$$

Climat class 2 and long term load. EC 1995-1-1 Table 3.1

$$\gamma_{M.CLT} := 1.2$$

Partial coefficient. EC 1995-1-1 Table 2.3

$$f_{c.0.k.CLT} := 21MPa$$

Characteristic value for compression parallel to the grain

$$f_{c.0.d.CLT} := \frac{f_{c.0.k.CLT} \cdot k_{mod.CLT}}{\gamma_{M.CLT}} = 12.25 \cdot MPa$$

Compression in the top flange parallel to the grain

$$f_{rv.k.CLT} := 4.0MPa$$

Characteristic value for panel shear

$$f_{rv.d.CLT} := \frac{f_{rv.k.CLT} \cdot k_{mod.CLT}}{\gamma_{M.CLT}} = 2.333 \cdot MPa$$

Panel shear between top flange and web due to stresses in diffrent directions

3.3.2 Glulam GL30c

$$k_{mod.GL} := 0.7$$

Climat class 2 and long term load. EC 1995-1-1 Table 3.1

$\gamma_{M.GL} := 1.25$

Partial coefficient. EC 1995-1-1 Table 2.3

$f_{t.0.k.GL} := 19.5MPa$

Characteristic value for tension parallell to the grain in bottom flange and in the web

$$f_{t.0.d.GL} := \frac{f_{t.0.k.GL} \cdot k_{mod.GL}}{\gamma_{M.GL}} = 10.92 \cdot MPa$$

Tension in the bottom flange and web parallel to the grain

$f_{pv.k.GL} := 3.5MPa$

Characteristic value for panel shear

$$f_{pv.d.GL} := \frac{f_{pv.k.GL} \cdot k_{mod.GL}}{\gamma_{M.GL}} = 1.96 \cdot MPa$$

Panel shear between bottom flange and web due to stresses in same directions. The value is the same for the rolling shear of the web

$f_{c.0.k.GL} := 24.5MPa$

Characteristic value for compression parallell to the grain in the web

$$f_{c.0.d.GL} := \frac{f_{c.0.k.GL} \cdot k_{mod.GL}}{\gamma_{M.GL}} = 13.72 \cdot MPa$$

Compression parallel to the grain in the web

3.4 Design of floor structure, initial

Top flange

$A_{top.flange} := t_{tf.floor} \cdot l_{top.floor} = 0.049\, m^2$

Area of the CLT flange

$E_{mean.CLT} := 7GPa$

Elastic modulus parallel to the grain for CLT

$G_{top.flange} := 440MPa$

Shear modulus for the top flange, CLT

$EA_{top.flange} := A_{top.flange} \cdot E_{mean.CLT} = 344.4 \cdot MN$

$$z_{top.flange} := \frac{t_{tf.floor}}{2} = 0.041\, m$$

Centre of gravity of the top flange

$EAz_{top.flange} := EA_{top.flange} \cdot z_{top.flange} = 14.12 \cdot MN \cdot m$

$GA_{top.flange} := G_{top.flange} \cdot A_{top.flange} = 21.648 \cdot MN$

Web

$A_{web.floor} := h_{web.floor} \cdot t_{web.floor} = 9.495 \times 10^{-3}\, m^2$

Area of the web

$E_{0.mean.glulam} := 13000MPa$ — Elastic modulus parallell to the grain for GL30c

$G_{web} := 638.4MPa$ — Shear modulus for the web, GL30c

$EA_{web} := A_{web.floor} \cdot E_{0.mean.glulam} = 123.435 \cdot MN$

$$z_{web} := t_{tf.floor} + \frac{h_{web.floor}}{2} = 0.188\,m$$

Centre of gravity of the web

$EAz_{web} := EA_{web} \cdot z_{web} = 23.144 \cdot MN \cdot m$

$GA_{web} := A_{web.floor} \cdot G_{web} = 6.062 \cdot MN$

Bottom flange

$A_{bottom.flange} := t_{bf.floor} \cdot h_{bf.floor} = 8.4 \times 10^{-3}\,m^2$ — Area of the bottom flange

$G_{bottom.flange} := 760MPa$ — Shear modulus for the bottom flange, GL30c

$EA_{bottom.flange} := A_{bottom.flange} \cdot E_{0.mean.glulam} = 109.2 \cdot MN$

$$z_{bottom.flange} := t_{tf.floor} + h_{web.floor} + \frac{t_{bf.floor}}{2} = 0.321\,m$$

Centre of gravity of the web

$EAz_{bottom.flange} := EA_{bottom.flange} \cdot z_{bottom.flange} = 35.053 \cdot MN \cdot m$

$GA_{bottom.flange} := A_{bottom.flange} \cdot G_{bottom.flange} = 6.384 \cdot MN$

Neutral axis of the floor structure

$$z_{na.floor} := \frac{EAz_{top.flange} + EAz_{web} + EAz_{bottom.flange}}{EA_{top.flange} + EA_{web} + EA_{bottom.flange}} = 0.125\,m$$

Stiffness for the different parts of the floor structure

$EI_{top.flange} := 2.64MN \cdot m^2$ — Stiffness of the top flange

$EI_{web} := 0.935MN \cdot m^2$ — Stiffness of the web

$EI_{bottom.flange} := 4.21MN \cdot m^2$ — Stiffness of the bottom flange

$EI_{tot.floor} := EI_{top.flange} + EI_{web} + EI_{bottom.flange} = 7.785 \times 10^6 \cdot N \cdot m^2$ — Total stiffness of the floors structure

3.4.1 Control of stresses in the floor structure

Bending stress in the top flange

$y_{i.\sigma.top.flange} := z_{na.floor} - \frac{t_{tf.floor}}{2} = 0.084\,m$

Location of the stress in the top flange

$\sigma_{c.top.flange} := \frac{M_{floor.ULS}}{EI_{tot.floor}} \cdot y_{i.\sigma.top.flange} \cdot E_{mean.CLT} = 1.103 \cdot MPa$

Bending stress in the top flange

Utilization ratio

$\frac{\sigma_{c.top.flange}}{f_{c.0.d.CLT}} = 9.002 \cdot \%$ $\quad 0.09 \leq 1 = 1$

OK !

Panel shear in the bottom of the top flange

$S_{1.pv.top.flange} := A_{top.flange} \cdot \left(z_{na.floor} - \frac{t_{tf.floor}}{2} \right) = 4.149 \times 10^{-3} \cdot m^3$

First moment of inertia for the top flange

$\tau_{pv.top.flange} := \frac{V_{floor.ULS} \cdot S_{1.pv.top.flange} \cdot E_{mean.CLT}}{\left(EI_{tot.floor} \cdot t_{web.floor} \right)} = 0.643 \cdot MPa$

Panel shear in the transition between the web and the top flange

Utilization ratio

$\frac{\tau_{pv.top.flange}}{f_{rv.d.CLT}} = 27.558 \cdot \%$ $\quad 0.276 \leq 1 = 1$

OK !

Stresses in top of the web

$y_{i.\sigma.top.web} := z_{na.floor} - t_{tf.floor} = 0.043\,m$

Location of the bending stress in the top web

$\sigma_{c.top.web} := \frac{M_{floor.ULS}}{EI_{tot.floor}} \cdot y_{i.\sigma.top.web} \cdot E_{0.mean.glulam} = 1.052 \cdot MPa$

Bending stress in the top web

Utilization ratio

$\frac{\sigma_{c.top.web}}{f_{c.0.d.GL}} = 7.669 \cdot \%$ $\quad 0.08 \leq 1 = 1$

OK !

$$S_{1.pv.top.web} := A_{top.flange} \cdot \left(z_{na.floor} - \frac{t_{tf.floor}}{2} \right) = 4.149 \times 10^{-3} \cdot m^3$$

First moment of inertia for the top web

$$\tau_{pv.top.web} := \frac{V_{floor.ULS} \cdot S_{1.pv.top.web} \cdot E_{0.mean.glulam}}{\left(EI_{tot.floor} \cdot t_{web.floor} \right)} = 1.194 \cdot MPa$$

Panel shear in the transition between the web and the top flange

Utilization ratio

$$\frac{\tau_{pv.top.web}}{f_{pv.d.GL}} = 60.927 \cdot \% \qquad 0.61 \le 1 = 1$$

OK !

Stresses in neutral axis, web

$$S_{1.pv.web} := A_{top.flange} \cdot \left(z_{na.floor} - \frac{t_{tf.floor}}{2} \right) = 4.149 \times 10^{-3} \cdot m^3$$

First moment of inertia for the top flange

$$S_{2.web} := t_{web.floor} \cdot \left(z_{na.floor} - t_{tf.floor} \right) \cdot \frac{\left(z_{na.floor} - t_{tf.floor} \right)}{2} = 4.224 \times 10^{-5} \cdot m^3$$

First moment of inertia for the web

$$\tau_{pv.web.na} := \frac{V_{floor.ULS} \cdot \left(S_{1.pv.web} \cdot E_{mean.CLT} + S_{2.web} \cdot E_{0.mean.glulam} \right)}{\left(EI_{tot.floor} \cdot t_{web.floor} \right)} = 0.655 \cdot MPa$$

Shear at the neutral axis, in the web

Utilization ratio

$$\frac{\tau_{pv.web.na}}{f_{pv.d.GL}} = 33.427 \cdot \% \qquad 0.33 \le 1 = 1$$

OK !

Stresses in bottom of the web

$$y_{i.\sigma.bottom.web} := h_{floor.cassette} - z_{na.floor} - t_{bf.floor} = 0.168\,m$$

Location of the bending stress in the top web

$$\sigma_{c.bottom.web} := \frac{M_{floor.ULS}}{EI_{tot.floor}} \cdot y_{i.\sigma.bottom.web} \cdot E_{0.mean.glulam} = 4.072 \cdot MPa$$

Bending stress in bottom of the web

$$\frac{\sigma_{c.bottom.web}}{f_{t.0.d.GL}} = 37.29 \cdot \% \qquad 0.373 \le 1 = 1$$

OK !

$$S_{1.pv.bottom.web} := A_{bottom.flange} \cdot \left(h_{floor.cassette} - z_{na.floor} - \frac{t_{bf.floor}}{2} \right) = 1.644 \times 10^{-3} \cdot m^3$$

First moment of inertia for the bottom web

$$\tau_{pv.bottom.web} := \frac{V_{floor.ULS} \cdot S_{1.pv.bottom.web} \cdot E_{0.mean.glulam}}{\left(EI_{tot.floor} \cdot t_{web.floor}\right)} = 0.473 \cdot MPa$$

Panel shear in the transition between the web and the top flange

Utilization ratio

$$\frac{\tau_{pv.bottom.web}}{f_{pv.d.GL}} = 24.138 \cdot \% \qquad 0.241 \le 1 = 1$$

OK !

Panel shear in the bottom of the top flange

$$S_{1.pv.bottom.flange} := S_{1.pv.bottom.web} = 1.644 \times 10^{-3} \cdot m^3$$

First moment of inertia for bottom flange

$$\tau_{pv.bottom.flange} := \frac{V_{floor.ULS} \cdot S_{1.pv.bottom.flange} \cdot E_{0.mean.glulam}}{\left(EI_{tot.floor} \cdot t_{web.floor}\right)} = 0.473 \cdot MPa$$

Panel shear in the transition between the web and the bottom flange

Utilization ratio

$$\frac{\tau_{pv.bottom.flange}}{f_{pv.d.GL}} = 24.138 \cdot \% \qquad 0.241 \le 1 = 1$$

OK !

Bending stress in the bottom flange

$$y_{i.\sigma.bottom.flange} := h_{floor.cassette} - z_{na.floor} - \frac{t_{bf.floor}}{2} = 0.196\ m$$

Location of the stress in the bottom flange

$$\sigma_{c.bottom.flange} := \frac{M_{floor.ULS}}{EI_{tot.floor}} \cdot y_{i.\sigma.bottom.flange} \cdot E_{0.mean.glulam} = 4.752 \cdot MPa$$

Bending stress in the bottom flange

Utilization ratio

$$\frac{\sigma_{c.bottom.flange}}{f_{t.0.d.GL}} = 43.517 \cdot \% \qquad 0.435 \le 1 = 1$$

OK !

3.5 Design of floor structure, ULS- final

$\psi_2 := 0.3$

$k_{def.fin} := 0.8$

Top flange

$$E_{d.fin.top} := \frac{(E_{mean.CLT})}{(1 + \psi_2 \cdot k_{def.fin})} = 5.645 \times 10^3 \cdot MPa$$

$EA_{top.flange.fin} := E_{d.fin.top} \cdot A_{top.flange} = 277.742 \cdot MN$

$EAz_{top.flange.fin} := z_{top.flange} \cdot EA_{top.flange.fin} = 11.387 \cdot MN \cdot m$

Web

$$E_{d.fin.web} := \frac{E_{0.mean.glulam}}{(1 + \psi_2 \cdot k_{def.fin})} = 1.048 \times 10^4 \cdot MPa$$

$EA_{web.fin} := E_{d.fin.web} \cdot A_{web.floor} = 99.544 \cdot MN$

$EAz_{web.fin} := z_{web} \cdot EA_{web.fin} = 18.665 \cdot MN \cdot m$

Bottom flange

$$E_{d.fin.bottom} := \frac{E_{0.mean.glulam}}{(1 + \psi_2 \cdot k_{def.fin})} = 1.048 \times 10^4 \cdot MPa$$

$EA_{bottom.fin} := E_{d.fin.bottom} \cdot A_{bottom.flange} = 88.065 \cdot MN$

$EAz_{bottom.fin} := z_{bottom.flange} \cdot EA_{bottom.fin} = 28.269 \cdot MN \cdot m$

Stiffness for the different parts of the floor structure

$EI_{top.flange.fin} := 2.131 MN \cdot m^2$ Stiffness of the top flange, final stage

$EI_{web.fin} := 0.754 MN \cdot m^2$ Stiffness of the web, final stage

$EI_{bottom.flange.fin} := 3.395 MN \cdot m^2$ Stiffness of the bottom flange, final stage

$$EI_{tot.floor.fin} := EI_{top.flange.fin} + EI_{web.fin} + EI_{bottom.flange.fin} = 6.28 \times 10^{6} \cdot N \cdot m^{2}$$

Total stiffness of the floors structure

3.5.1 Control of stresses in the floor structure, ULS- final

Bending stress in the top flange

$$y_{i.\sigma.top.flange.fin} := z_{na.floor} - \frac{t_{tf.floor}}{2} = 0.084\,m$$

Location of the stress in the top flange

$$\sigma_{c.top.flange.fin} := \frac{M_{floor.ULS}}{EI_{tot.floor.fin}} \cdot y_{i.\sigma.top.flange.fin} \cdot E_{d.fin.top} = 1.102 \cdot MPa$$

Bending stress in the top flange

Utilization ratio

$$\frac{\sigma_{c.top.flange.fin}}{f_{c.0.d.CLT}} = 8.999 \cdot \%$$ $$0.9 \le 1 = 1$$

OK !

Panel shear in the bottom of the top flange

$$S_{1.pv.top.flange.fin} := A_{top.flange} \cdot \left(z_{na.floor} - \frac{t_{tf.floor}}{2} \right) = 4.149 \times 10^{-3} \cdot m^{3}$$

First moment of inertia for the top flange

$$\tau_{pv.top.flange.fin} := \frac{V_{floor.ULS} \cdot S_{1.pv.top.flange.fin} \cdot E_{d.fin.top}}{\left(EI_{tot.floor.fin} \cdot t_{web.floor} \right)} = 0.643 \cdot MPa$$

Panel shear in the transition between the web and the top flange

Utilization ratio

$$\frac{\tau_{pv.top.flange.fin}}{f_{rv.d.CLT}} = 27.55 \cdot \%$$ $$0.28 \le 1 = 1$$

OK !

Stresses in top of the web

$$y_{i.\sigma.top.web.fin} := z_{na.floor} - t_{tf.floor} = 0.043\,m$$

Location of the bending stress in the top web

$$\sigma_{c.top.web.fin} := \frac{M_{floor.ULS}}{EI_{tot.floor.fin}} \cdot y_{i.\sigma.top.web.fin} \cdot E_{d.fin.web} = 1.052 \cdot MPa$$

Bending stress in the top web

Utilization ratio

$$\frac{\sigma_{c.top.web.fin}}{f_{c.0.d.GL}} = 7.667 \cdot \%$$ $0.08 \le 1 = 1$ **OK !**

$$S_{1.pv.top.web.fin} := A_{top.flange} \cdot \left(z_{na.floor} - \frac{t_{tf.floor}}{2} \right) = 4.149 \times 10^{-3} \cdot m^3$$

First moment of inertia for the top web

$$\tau_{pv.top.web.fin} := \frac{V_{floor.ULS} \cdot S_{1.pv.top.web.fin} \cdot E_{d.fin.web}}{\left(EI_{tot.floor.fin} \cdot t_{web.floor} \right)} = 1.194 \cdot MPa$$

Panel shear in the transition between the web and the top flange

Utilization ratio

$$\frac{\tau_{pv.top.web.fin}}{f_{pv.d.GL}} = 60.91 \cdot \%$$ $0.61 \le 1 = 1$ **OK !**

Stresses in neutral axis, web

$$S_{1.pv.web.fin} := A_{top.flange} \cdot \left(z_{na.floor} - \frac{t_{tf.floor}}{2} \right) = 4.149 \times 10^{-3} \cdot m^3$$

First moment of inertia for the top flange

$$S_{2.web.fin} := t_{web.floor} \cdot \left(z_{na.floor} - t_{tf.floor} \right) \cdot \frac{\left(z_{na.floor} - t_{tf.floor} \right)}{2} = 4.224 \times 10^{-5} \cdot m^3$$

First moment of inertia for the web

$$\tau_{pv.web.na.fin} := \frac{V_{floor.ULS} \cdot \left(S_{1.pv.web.fin} \cdot E_{d.fin.top} + S_{2.web.fin} \cdot E_{d.fin.web} \right)}{\left(EI_{tot.floor.fin} \cdot t_{web.floor} \right)} = 0.655 \cdot MPa$$

Shear at the neutral axis, in the web

Utilization ratio

$$\frac{\tau_{pv.web.na.fin}}{f_{pv.d.GL}} = 33.418 \cdot \%$$ $0.33 \le 1 = 1$ **OK !**

Stresses in bottom of the web

$$y_{i.\sigma.bottom.web.fin} := h_{floor.cassette} - z_{na.floor} - t_{bf.floor} = 0.168\,m$$

Location of the bending stress in the top web

$$\sigma_{c.bottom.web.fin} := \frac{M_{floor.ULS}}{EI_{tot.floor.fin}} \cdot y_{i.\sigma.bottom.web.fin} \cdot E_{d.fin.web} = 4.071 \cdot MPa$$

Bending stress in bottom of the web

$$\frac{\sigma_{c.bottom.web.fin}}{f_{t.0.d.GL}} = 37.28 \cdot \% \qquad 0.373 \le 1 = 1$$

OK !

$$S_{1.pv.bottom.web.fin} := A_{bottom.flange} \cdot \left(h_{floor.cassette} - z_{na.floor} - \frac{t_{bf.floor}}{2} \right) = 1.644 \times 10^{-3} \cdot m^3$$

First moment of inertia for the bottom web

$$\tau_{pv.bottom.web.fin} := \frac{V_{floor.ULS} \cdot S_{1.pv.bottom.web.fin} \cdot E_{d.fin.web}}{\left(EI_{tot.floor.fin} \cdot t_{web.floor} \right)} = 0.473 \cdot MPa$$

Panel shear in the transition between the web and the top flange

Utilization ratio

$$\frac{\tau_{pv.bottom.web.fin}}{f_{pv.d.GL}} = 24.131 \cdot \% \qquad 0.241 \le 1 = 1$$

OK !

Panel shear in the bottom of the top flange

$$S_{1.pv.bottom.flange.fin} := S_{1.pv.bottom.web.fin} = 1.644 \times 10^{-3} \cdot m^3$$

First moment of inertia for bottom flange

$$\tau_{pv.bottom.flange.fin} := \frac{V_{floor.ULS} \cdot S_{1.pv.bottom.flange.fin} \cdot E_{d.fin.web}}{\left(EI_{tot.floor.fin} \cdot t_{web.floor} \right)} = 0.473 \cdot MPa$$

Panel shear in the transition between the web and the bottom flange

Utilization ratio

$$\frac{\tau_{pv.bottom.flange.fin}}{f_{pv.d.GL}} = 24.131 \cdot \% \qquad 0.24 \le 1 = 1$$

OK !

Bending stress in the bottom flange

$$y_{i.\sigma.bottom.flange.fin} := h_{floor.cassette} - z_{na.floor} - \frac{t_{bf.floor}}{2} = 0.196\, m$$

ation of the stress in the om flange

$$\sigma_{c.bottom.flange.fin} := \frac{M_{floor.ULS}}{EI_{tot.floor.fin}} \cdot y_{i.\sigma.bottom.flange.fin} \cdot E_{d.fin.bottom} = 4.751 \cdot MPa$$

Utilization ratio

Bending stress in the bottom flange

$$\frac{\sigma_{c.bottom.flange.fin}}{f_{t.0.d.GL}} = 43.505 \cdot \% \qquad 0.435 \le 1 = 1$$

OK !

3.6 Deflecion of the floor structure

Initial deflection SLS

$$u_{init.M} := \frac{5 \cdot Q_{floor.SLS.a} \cdot b_{f.floor}^{4}}{384 \cdot EI_{tot.floor}} = 9.9 \times 10^{-3}\,m$$

Initial deflection of the floor strucutre due to bending moment

$$u_{init.V} := \frac{1.2 \cdot Q_{floor.SLS.a} \cdot b_{f.floor}^{2}}{8 \cdot GA_{web}} = 2.604 \times 10^{-3}\,m$$

Initial deflection of the floor strucutre due to shear force

$$u_{init.tot} := u_{init.M} + u_{init.V} = 0.013\,m$$

Total initial deflection of the floor structure

$$u_{limit.initial} := \min\left(20mm, \frac{b_{f.floor}}{200}\right) = 0.02\,m$$

Limit of initial deflection

Utilization ratio

$$\frac{u_{init.tot}}{u_{limit.initial}} = 62.519 \cdot \% \qquad 0.625 \le 1 = 1$$

OK !

Final deflection SLS

$$EI_{tot.final} := 6.28 \cdot 10^{6} N \cdot m^{2}$$

In the final stage has long term factors like ψ.2=0.3 and also k.def=0.8 been used, therefore has the value of EI.tot changed. See Appendix 17

$$u_{fin.M} := \frac{5 \cdot Q_{floor.SLS.a} \cdot b_{f.floor}^{4}}{384 \cdot EI_{tot.final}} = 0.012\,m$$

Final deflection of the floor strucutre due to bending moment

$$u_{fin.V} := \frac{1.2 \cdot Q_{floor.SLS.a} \cdot b_{f.floor}^{2}}{8 \cdot GA_{web}} = 2.604 \times 10^{-3}\,m$$

Final deflection of the floor strucutre due to shear force

$$u_{fin.tot} := u_{fin.M} + u_{fin.V} = 0.015\,m$$

Total final deflection of the floor structure

$u_{limit.final} := \min\left(20mm, \frac{b_{f.floor}}{300}\right) = 0.02\,m$ Limit of initial deflection

Utilization ratio

$\frac{u_{fin.tot}}{u_{limit.final}} = 74.381\cdot\%$ $0.744 \le 1 = 1$ **OK !**

3.7 Dynamic analysis of the floor structure

The dynamic analysis of the floor structure consists of controlling the static delection, velocity response and number of modes between 40 Hz.

Material data

$EI_{tot.ULS} := 5.94 \times 10^{6} N\cdot m^{2}$ From the excel sheet in Appendix 17

Cross laminated timber

$\rho_{mean.CLT} := 480 \frac{kg}{m^3}\cdot g = 4.707\cdot\frac{kN}{m^3}$ Density of CLT

$E_{0.mean.CLT} := 11000MPa$ Elastic modulus parallell to the grain

Glulam GL30c

$\rho_{mean.glulam} := 4.3 \frac{kN}{m^3}$ Density of the glulam members

Self-weight of the floor

$$m_{floor} := \frac{\begin{pmatrix} \rho_{mean.CLT}\cdot t_{tf.floor}\cdot s_{floor}\cdot b_{f.floor} \cdots \\ + \rho_{mean.glulam}\cdot t_{web.floor}\cdot h_{web.floor}\cdot b_{f.floor} \cdots \\ + \rho_{mean.glulam}\cdot t_{bf.floor}\cdot h_{bf.floor}\cdot b_{f.floor} \end{pmatrix}}{b_{f.floor}\cdot s_{floor}} = 0.514\cdot\frac{kN}{m^2}$$

$m_{floor.} := \frac{m_{floor}}{g} = 52.438 \frac{kg}{m^2}$ Total self-weight of the floor

3.7.1 Check - Static deflection

$$\boxed{\frac{w_{floor}}{P_{pointload}} < a}$$

Criterion for the static deflection

$$a := 1.5\frac{mm}{kN}$$

Static criterion, EC 1995-1-1, Figure 7.2

$$P_{pointload} := 1kN$$

Static point load from human response, applied at any point on the floor

$$b := 100$$

$$w_{floor} := \frac{P_{pointload} \cdot b_{f.floor}^{3}}{48 \cdot EI_{tot.ULS}} = 1.48 \cdot mm$$

Static deflection of the floor

$$\frac{\frac{w_{floor}}{P_{pointload}}}{a} = 98.643 \cdot \% \qquad 0.986 \le 1 = 1$$

OK!

3.7.2 Check - Natural frequency

$$\boxed{f_1 \ge 8 \cdot Hz}$$

Criterion for natural frequency in a timber structure for a residental building

$$EI_l := \frac{EI_{tot.ULS}}{s_{floor}} = 9.9 \times 10^{6} \cdot N \cdot m$$

Stiffness of the floor

$$f_1 := \left(\frac{\pi}{2 \cdot b_{f.floor}^{2}}\right) \cdot \sqrt{\left(\frac{EI_l}{m_{floor.}}\right)} = 12.134 \cdot Hz$$

First natural frequency
EC 1995-1-1, EC 7.5

$$f_1 \ge 8Hz = 1$$

OK!

3.7.3 Check - Number of modes below 40 Hz

$$\boxed{\nu_{velocity} \le b^{\left(\frac{f_1}{Hz} \cdot \xi - 1\right)}}$$

Criterion for number of modes below 40 Hz

$$EI_b := E_{0.mean.CLT} \cdot \frac{t_{tf.floor}^{3}}{12} + E_{0.mean.glulam} \cdot \frac{t_{bf.floor}^{3}}{12} = 6.957 \times 10^{5} \cdot N \cdot m$$

$$n_{40} := \left[\left[\left(\frac{40Hz}{f_1}\right)^2 - 1\right]\cdot\left(\frac{l_{f.floor}}{b_{f.floor}}\right)^4\cdot\left(\frac{EI_l}{EI_b}\right)\right]^{0.25} = 2.524$$

Number of modes below 40Hz
EC 1995-1-1, Eq 7.7

$$\nu_{velocity} := \frac{4\cdot(0.4 + 0.6\cdot n_{40})}{m_{floor.}\cdot b_{f.floor}\cdot l_{f.floor} + 200\cdot kg} = 3.241 \times 10^{-3}\cdot\frac{m}{N\cdot s^2}$$

Peak velocity due to impluse for rectangular floor system, simply supported. EC 1995-1-1, Eq 7.6

$$\xi := 0.01$$

Modal damping ratio, 1%. EC 1995-1-1, section 7.3.1

$$b^{\left(\frac{f_1}{Hz}\cdot\xi-1\right)}\cdot\frac{m}{N\cdot s^2} = 0.017\,\frac{1}{kg}$$

Criteria for the unit impluse velocity response. EC 1995-1-1, Eq 7.4

$$\frac{\nu_{velocity}}{b^{\left(\frac{f_1}{Hz}\cdot\xi-1\right)}\cdot\frac{m}{N\cdot s^2}} = 18.535\cdot\%$$

$$0.185 \le 1 = 1\cdot 1$$

OK!

Appendix 4 - Design of beams

4.1 Material data

The beams are in glulaminated timber, GL30c and are assumed to be continous over the lenght of the building

4.1.1 Strength values

$\rho_{glulam} := 4300 \frac{N}{m^3}$ Density of the beam

$f_{m.g.k.glulam} := 30.0MPa$ Bending parallel to the grain

$f_{v.g.k.glulam} := 3.5MPa$ Shear strength

$f_{c.90.k.glulam} := 2.5MPa$ Compression perpendicular to the grain

$E_{0.05.glulam} := 10800MPa$ Elastic modulus

$E_{0.mean.glulam} := 13000MPa$ Elastic modulus parallel to the grain

$\gamma_{M.glulam} := 1.25$ Partial factor. EC 1995-1-1, Table 2.3

$k_{mod.glulam} := 0.7$ Strength modification factor. Assumed long term loading and service class 2

$k_{def.glulam} := 0.8$ Modification factor due to deformation for service class 2

$k_{cr} := 0.67$ Recommended value. Takes the influence of cracks into account. EC 1995-1-1

$k_{c.90.glulam} := 1$ Factor taking the load configuration, possibility of splitting and degree of compressive deformation into account

4.1.2 Dimensions

$l_{trib} := 7m$ Longest tributary length of the beams

$h_{lamell} := 45mm$ Height of one glulam lamella

$h_{beam.glulam} := 13 \cdot h_{lamell} = 0.585\,m$ Assumed height of the beam. Iterative process

$w_{beam.glulam} := 0.225m$ Assumed width of the beam. Iterative process

$A_{beam.glulam} := h_{beam.glulam} \cdot w_{beam.glulam} = 0.132\,m^2$ Area of beam section

$l_{span.GL} := 5.5m$

Maximum span length of beam

4.2 Loads acting on the beams

$g_{k.floor} := g_{k.floor.self} + g_{k.in.wall} = 1.118 \cdot \frac{kN}{m^2}$

Self - weight of the floor structure including inner walls and installations

$q_{k.imp} := 2.0 \frac{kN}{m^2}$

Imposed load for residental building EC 1991-1-1, Table 6.2

4.2.1 Load combinations in ULS

The used load combinations in ULS are according to the Nation Standards in Sweden for equations 6.10a and 6.10b.
Assuming that the beams are simply supported and continous over the spans.

$\gamma_d := 0.91$

Partial coefficient for safety class 2

ULS - Equation 6.10a

$$Q_{beam.a} := \gamma_d \cdot 1.35 \cdot \left(g_{k.floor} \cdot l_{trib} + \rho_{glulam} \cdot A_{beam.glulam}\right) + 1.5 \cdot \psi_{0.i} \cdot q_{k.imp} \cdot l_{trib} = 25.008 \cdot \frac{kN}{m}$$

ULS - Equation 6.10b

$$Q_{beam.b} := \gamma_d \cdot 0.89 \cdot 1.35 \cdot \left(g_{k.floor} \cdot l_{trib} + \rho_{glulam} \cdot A_{beam.glulam}\right) + 1.5 \cdot q_{k.imp} \cdot l_{trib} = 30.174 \cdot \frac{kN}{m}$$

$Q_{beam} := \max\left(Q_{beam.a}, Q_{beam.b}\right) = 30.174 \cdot \frac{kN}{m}$

Maximum load on the beam

4.2.2 Maximum bending moment

$$M_{Ed.max.beam} := \frac{Q_{beam} \cdot l_{span.GL}^{2}}{8} = 114.096 \cdot kN \cdot m$$

Maximum bending moment occurs in the middle of the span

4.2.3 Maximum shear force

$$V_{Ed.max.beam} := \frac{Q_{beam} \cdot l_{span.GL}}{2} = 82.979 \cdot kN$$

Maximum shear force occurs in the ends of the beam

4.3 Check - Moment capacity of the beams

$\boxed{M_{Ed.max.beam} \le M_{Rd.glulam}}$

Criteria for the moment capacity of the beam

$$k_{h.beam.glulam} := \min\left[\left(\frac{600mm}{h_{beam.glulam}}\right)^{0.1}, 1.1\right] = 1.003$$

Since h is less than 600 mm. EC 1995-1-1, Eq 3.2

$$W_{beam.glulam} := \frac{w_{beam.glulam} \cdot h_{beam.glulam}^2}{6} = 0.013 \cdot m^3$$

Section modulus

$$I_{beam.glulam} := \frac{w_{beam.glulam} \cdot h_{beam.glulam}^3}{12} = 3.754 \times 10^{-3} \, m^4$$

Second moment of inertia

$$f_{m.g.d.glulam} := k_{mod.glulam} \cdot k_{h.beam.glulam} \cdot \frac{f_{m.g.k.glulam}}{\gamma_{M.glulam}} = 16.843 \cdot MPa$$

Design value for bending stress parallel to the grain. EC 1995-1-1, Eq 2.17

$$f_{v.g.d.glulam} := k_{mod.glulam} \cdot \frac{f_{v.g.k.glulam}}{\gamma_{M.glulam}} = 1.96 \cdot MPa$$

Design shear strenght. EC 1995-1-1, Eq 2.17

$$M_{Rd.glulam} := f_{m.g.d.glulam} \cdot W_{beam.glulam} = 216.148 \cdot kN \cdot m$$

Maximum moment in the glulam beam

$$\frac{M_{Ed.max.beam}}{M_{Rd.glulam}} = 52.786 \cdot \% \qquad 0.528 < 1 = 1$$

OK!

4.4 Check - Shear capacity of the beams

$$\boxed{\tau_{d.glulam} \le f_{v.g.d.glulam}}$$

Criteria for the shear capacity of the beam

$$b_{eff.glulam} := k_{cr} \cdot w_{beam.glulam} = 0.151 \, m$$

Effective width of the beam. EC 1995-1-1, Eq 6.13a

$$S_{beam.glulam} := w_{beam.glulam} \cdot \frac{1}{2} \cdot h_{beam.glulam} \cdot \frac{1}{4} \cdot h_{beam.glulam} = 9.625 \times 10^{-3} \cdot m^3$$

First moment of inertia

$$\tau_{d.glulam} := \frac{S_{beam.glulam} \cdot V_{Ed.max.beam}}{I_{beam.glulam} \cdot b_{eff.glulam}} = 1.411 \cdot MPa$$

Shear force in the glulam beam

$$\frac{\tau_{d.glulam}}{f_{v.g.d.glulam}} = 72.009 \cdot \% \qquad 0.720 < 1 = 1$$

OK!

4.5 Check - Deflection of the beams

According to EC 1995-1-1 Equations 2.2-2.4 the deflection of the beam due to permanent and varibel load will be calculated in the following section

$\boxed{w_{fin.beam.glulam} \le \frac{l}{200}}$ Criteria for the deflection of the beam

$$w_{fin.G.beam.glulam} := \frac{\begin{pmatrix} g_{k.floor} \cdot l_{trib} \cdots \\ + \rho_{glulam} \cdot A_{beam.glulam} \end{pmatrix} \cdot 5 \cdot l_{trib}^{4}}{384 \cdot E_{0.mean.glulam} \cdot I_{beam.glulam}} \cdot \left(1 + k_{def.glulam}\right) = 9.676 \times 10^{-3}\,m$$

$$w_{fin.Q.beam.glulam} := \frac{q_{k.imp} \cdot l_{trib} \cdot 5 \cdot l_{trib}^{4}}{384 \cdot \left(E_{0.mean.glulam} \cdot I_{beam.glulam}\right)} \cdot \left(1 + k_{def.glulam} \cdot \psi_2\right) = 0.011\,m$$

$$w_{fin.beam.glulam} := w_{fin.G.beam.glulam} + w_{fin.Q.beam.glulam} = 0.021\,m$$

$$w_{limit} := \frac{l_{span.GL}}{200} = 0.028\,m$$

Limit of the deflection, l/200

$\frac{w_{fin.beam.glulam}}{w_{limit}} = 75.627 \cdot \%$ $\quad 0.756 < 1 = 1$ **OK!**

4.6 Check - Compression perpendicular to the grain

The dimensions of the columns are calculated in Appendix 5

$\boxed{\sigma_{c.90.d.contact} \le k_{c.90} \cdot f_{c.90.d}}$ Criteria for compressive stress in the effective contact area

$i := 0 .. 4$

$$h_{glulam.col} := \begin{pmatrix} 180 \\ 225 \\ 270 \\ 315 \\ 360 \end{pmatrix} mm$$

Height of column

$$w_{glulam.col} := \begin{pmatrix} 165 \\ 165 \\ 190 \\ 190 \\ 215 \end{pmatrix} mm$$

Width of column

$$A_{ef.contact_i} := h_{glulam.col_i} \cdot w_{glulam.col_i}$$

Effective contact area between the beam and column

$$A_{ef.contact} = \begin{pmatrix} 0.03 \\ 0.037 \\ 0.051 \\ 0.06 \\ 0.077 \end{pmatrix} m^2$$

$$\sigma_{c.90.d.contact_i} := \frac{V_{Ed.max.beam}}{A_{ef.contact_i}}$$

Design compressive stress in the effective contact area. EC 1995-1-1, Eq 6.4

$$\sigma_{c.90.d.contact} = \begin{pmatrix} 2.794 \\ 2.235 \\ 1.618 \\ 1.386 \\ 1.072 \end{pmatrix} \cdot MPa$$

$$f_{c.90.d.beam} := \frac{k_{mod.glulam} \cdot f_{c.90.k.glulam}}{\gamma_{M.glulam}} = 1.4 \cdot MPa$$

Design compressive strength perpendicular to the grain. EC 1995-1-1, Eq 2.17

$$\frac{\sigma_{c.90.d.contact}}{f_{c.90.d.beam} \cdot k_{c.90.glulam}} = \begin{pmatrix} 199.564 \\ 159.651 \\ 115.537 \\ 99.032 \\ 76.577 \end{pmatrix} \cdot \%$$

NOT OK!

Appendix 5 - Vertical loads in the columns on the added floors

The columns on in the added floors are in glulaminated timber and strength class GL30c. In this Appendix, the vertical loads in the columns are calculated for five different cases, where one-five floors are added.

5.1 Vertical loads on the new timber columns

5.1.1 Variable loads

Snowload

$\mu_1 := 0.8$

Snow load shape coefficient, angle of roof less than 30 degrees.
EC1991-1-3, Table 5.2

$s_k := 1.5 \frac{kN}{m^2}$

Characteristic snow load in Gothenburg
EC 1991-1-3, Table NB:1

$C_e := 1$

Exposure coefficient

$C_t := 1$

Thermal coefficient

$S := \mu_1 \cdot C_e \cdot C_t \cdot s_k = 1.2 \cdot \frac{kN}{m^2}$

Snow load, EC 1991-1-3 eq. 5.1

$\psi_{0.s} := 0.6$

Since
1.0<s.k<2.0

Imposed load

$q_{k.imp} := 2.0 \frac{kN}{m^2}$

Imposed load for residental building
EC 1991-1-1, Table 6.2

$\psi_{0.i.} := 0.7$

5.1.2 Permanent loads

Floor structure

$g_{k.floor.self} := 63 \frac{kg}{m^2} \cdot g = 0.618 \cdot \frac{kN}{m^2}$

Self-weight of the floor structure

$g_{k.in.wall} := 0.5 \frac{kN}{m^2}$

Assumed weight of the inner walls and installations

$g_{k.floor} := g_{k.floor.self} + g_{k.in.wall} = 1.118 \cdot \frac{kN}{m^2}$

Total self-weight for the floors structure

Roof structure

$g_{k.roof.timber} := 0.3 \frac{kN}{m^2}$ Roof in timber and installations

Glulam beam

$\rho_{GL} := 4300 \frac{N}{m^3}$ Density of glulam

$A_{beam.glulam} = 0.132\, m^2$ Area of beam section

$g_{k.beam} := \rho_{GL} \cdot A_{beam.glulam} = 0.566 \cdot \frac{kN}{m}$ Glulam beam

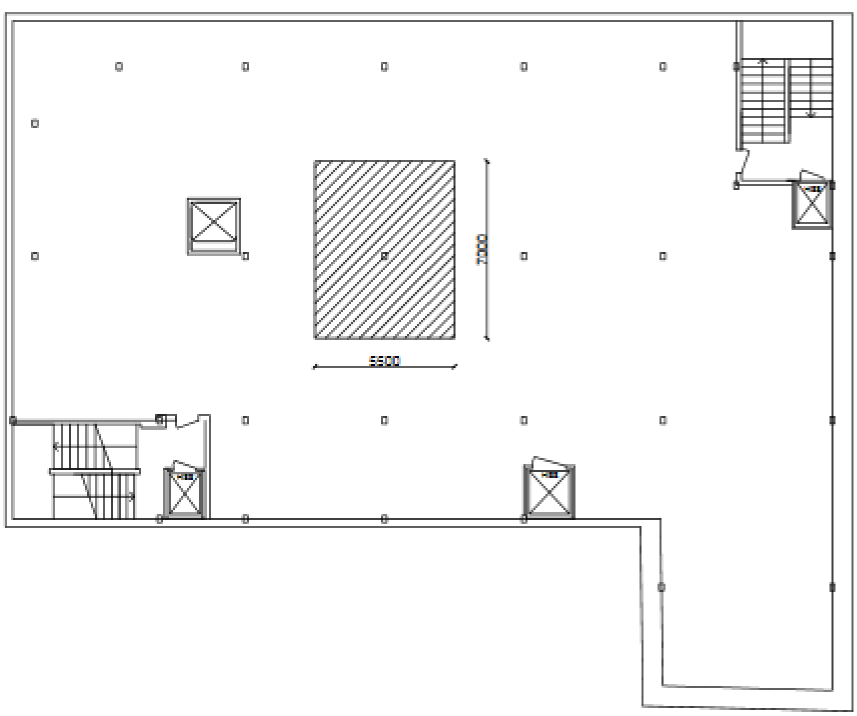
$l_{span.GL} = 5.5\, m$ Maximum span length of the beam

5.1.3 Geometry

$A_{trib} := 7.5m \cdot 5.5m = 41.25\, m^2$ The largest loaded area resisted by the columns according to the Figure below

$h_{column} := 2.8m$ The height of the column

The figure below shows the largest tributary area of the columns.



5.2 Load combinations in ULS

The used load combinations in ULS are according to the Nation Standards in Sweden for equations 6.10a and 6.10b. The calculations are performed in five different cases, depending on how many floors that are added to the existing building. In this thesis, the calculations are performed for up to five added floors.

$\gamma_d := 0.91$ Partial coefficient for safety class 2

5.2.1 Load when one floor is added

Imposed load as main load

$$Q_{column1.a.i} := \gamma_d \cdot 1.35 \cdot \left[\left(g_{k.floor} + g_{k.roof.timber} \right) \cdot A_{trib} + g_{k.beam} \cdot l_{span.GL} \right] \ldots + 1.5 \cdot \gamma_d \cdot \psi_{0.i.} \cdot q_{k.imp} \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot S \cdot \psi_{0.s} \cdot A_{trib} = 195.042 \cdot kN$$

$$Q_{column1.b.i} := \gamma_d \cdot 1.35 \cdot 0.89 \cdot \left[\left(g_{k.floor} + g_{k.roof.timber} \right) \cdot A_{trib} + g_{k.beam} \cdot l_{span.GL} \right] \ldots + 1.5 \cdot \gamma_d \cdot q_{k.imp} \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot \psi_{0.s} \cdot S \cdot A_{trib} = 220.502 \cdot kN$$

Snow load as main load

$$Q_{column1.a.s} := \gamma_d \cdot 1.35 \cdot \left[\left(g_{k.floor} + g_{k.roof.timber} \right) \cdot A_{trib} + g_{k.beam} \cdot l_{span.GL} \right] \ldots + 1.5 \cdot \gamma_d \cdot \psi_{0.s} \cdot S \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot q_{k.imp} \cdot \psi_{0.i.} \cdot A_{trib} = 195.042 \cdot kN$$

$$Q_{column1.b.s} := \gamma_d \cdot 1.35 \cdot 0.89 \cdot \left[\left(g_{k.floor} + g_{k.roof.timber} \right) \cdot A_{trib} + g_{k.beam} \cdot l_{span.GL} \right] \ldots + 1.5 \cdot \gamma_d \cdot S \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot \psi_{0.i.} \cdot q_{k.imp} \cdot A_{trib} = 213.745 \cdot kN$$

$$Q_{column1} := \max\left(Q_{column1.a.i}, Q_{column1.b.i}, Q_{column1.a.s}, Q_{column1.b.s} \right) = 220.502 \cdot kN$$

5.2.2 Load when two floors are added

Imposed load as main load

$$Q_{column2.a.i} := \gamma_d \cdot 1.35 \left[\left(2 g_{k.floor} + g_{k.roof.timber} \right) \cdot A_{trib} + 2 g_{k.beam} \cdot l_{span.GL} \right] \ldots + 1.5 \cdot \gamma_d \cdot \psi_{0.i.} \cdot 2 q_{k.imp} \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot S \cdot \psi_{0.s} \cdot A_{trib} = 334.342 \cdot kN$$

$$Q_{column2.b.i} := \gamma_d \cdot 1.35 \cdot 0.89 \cdot \left[\left(2 g_{k.floor} + g_{k.roof.timber} \right) \cdot A_{trib} + 2 g_{k.beam} \cdot l_{span.GL} \right] \ldots + 1.5 \cdot \gamma_d \cdot 2 q_{k.imp} \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot \psi_{0.s} \cdot S \cdot A_{trib} = 386.933 \cdot kN$$

Snow load as main load

$$Q_{column2.a.s} := \gamma_d \cdot 1.35 \cdot \left[\left(2 g_{k.floor} + g_{k.roof.timber} \right) \cdot A_{trib} + 2 g_{k.beam} \cdot l_{span.GL} \right] \ldots + 1.5 \cdot \gamma_d \cdot \psi_{0.s} \cdot S \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot 2 q_{k.imp} \cdot \psi_{0.i.} \cdot A_{trib} = 334.342 \cdot kN$$

$$Q_{column2.b.s} := \gamma_d \cdot 1.35 \cdot 0.89 \cdot \left[\left(2g_{k.floor} + g_{k.roof.timber}\right) \cdot A_{trib} + 2g_{k.beam} \cdot l_{span.GL}\right] \ldots = 346.393 \cdot kN$$
$$+ 1.5 \cdot \gamma_d \cdot S \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot \psi_{0.i.} \cdot 2q_{k.imp} \cdot A_{trib}$$

$$Q_{column2} := \max\left(Q_{column2.a.i}, Q_{column2.b.i}, Q_{column2.a.s}, Q_{column2.b.s}\right) = 386.933 \cdot kN$$

5.2.3 Load when three floors are added

Imposed load as main load

$$Q_{column3.a.i} := \gamma_d \cdot 1.35\left[\left(3g_{k.floor} + g_{k.roof.timber}\right) \cdot A_{trib} + 3g_{k.beam} \cdot l_{span.GL}\right] \ldots = 473.641 \cdot kN$$
$$+ 1.5 \cdot \gamma_d \cdot \psi_{0.i.} \cdot 3q_{k.imp} \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot S \cdot \psi_{0.s} \cdot A_{trib}$$

$$Q_{column3.b.i} := \gamma_d \cdot 1.35 \cdot 0.89 \cdot \left[\left(3g_{k.floor} + g_{k.roof.timber}\right) \cdot A_{trib} + 3g_{k.beam} \cdot l_{span.GL}\right] \ldots = 553.364 \cdot kN$$
$$+ 1.5 \cdot \gamma_d \cdot 3q_{k.imp} \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot \psi_{0.s} \cdot S \cdot A_{trib}$$

Snow load as main load

$$Q_{column3.a.s} := \gamma_d \cdot 1.35 \cdot \left[\left(3g_{k.floor} + g_{k.roof.timber}\right) \cdot A_{trib} + 3g_{k.beam} \cdot l_{span.GL}\right] \ldots = 473.641 \cdot kN$$
$$+ 1.5 \cdot \gamma_d \cdot \psi_{0.s} \cdot S \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot 3q_{k.imp} \cdot \psi_{0.i.} \cdot A_{trib}$$

$$Q_{column3.b.s} := \gamma_d \cdot 1.35 \cdot 0.89 \cdot \left[\left(3g_{k.floor} + g_{k.roof.timber}\right) \cdot A_{trib} + 3g_{k.beam} \cdot l_{span.GL}\right] \ldots = 479.04 \cdot kN$$
$$+ 1.5 \cdot \gamma_d \cdot S \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot \psi_{0.i.} \cdot 3q_{k.imp} \cdot A_{trib}$$

$$Q_{column3} := \max\left(Q_{column3.a.i}, Q_{column3.b.i}, Q_{column3.a.s}, Q_{column3.b.s}\right) = 553.364 \cdot kN$$

5.2.4 Load when four floors are added

Imposed load as main load

$$Q_{column4.a.i} := \gamma_d \cdot 1.35\left[\left(4g_{k.floor} + g_{k.roof.timber}\right) \cdot A_{trib} + 4g_{k.beam} \cdot l_{span.GL}\right] \ldots = 612.94 \cdot kN$$
$$+ 1.5 \cdot \gamma_d \cdot \psi_{0.i.} \cdot 4q_{k.imp} \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot S \cdot \psi_{0.s} \cdot A_{trib}$$

$$Q_{column4.b.i} := \gamma_d \cdot 1.35 \cdot 0.89 \cdot \left[\left(4g_{k.floor} + g_{k.roof.timber}\right) \cdot A_{trib} + 4g_{k.beam} \cdot l_{span.GL}\right] \ldots = 719.796 \cdot kN$$
$$+ 1.5 \cdot \gamma_d \cdot 4q_{k.imp} \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot \psi_{0.s} \cdot S \cdot A_{trib}$$

Snow load as main load

$$Q_{column4.a.s} := \gamma_d \cdot 1.35 \cdot \left[\left(4g_{k.floor} + g_{k.roof.timber}\right) \cdot A_{trib} + 4g_{k.beam} \cdot l_{span.GL}\right] \ldots = 612.94 \cdot kN$$
$$+ 1.5 \cdot \gamma_d \cdot \psi_{0.s} \cdot S \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot 4q_{k.imp} \cdot \psi_{0.i.} \cdot A_{trib}$$

$$Q_{column4.b.s} := \gamma_d \cdot 1.35 \cdot 0.89 \cdot \left[\left(4g_{k.floor} + g_{k.roof.timber}\right) \cdot A_{trib} + 4g_{k.beam} \cdot l_{span.GL}\right] \ldots = 611.688 \cdot kN$$
$$+ 1.5 \cdot \gamma_d \cdot S \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot \psi_{0.i.} \cdot 4q_{k.imp} \cdot A_{trib}$$

$$Q_{column4} := \max\left(Q_{column4.a.i}, Q_{column4.b.i}, Q_{column4.a.s}, Q_{column4.b.s}\right) = 719.796 \cdot kN$$

5.2.5 Load when five floors are added

Imposed load as main load

$$Q_{column5.a.i} := \gamma_d \cdot 1.35 \left[\left(5g_{k.floor} + g_{k.roof.timber}\right) \cdot A_{trib} + 5g_{k.beam} \cdot l_{span.GL}\right] \ldots = 752.239 \cdot kN$$
$$+ 1.5 \cdot \gamma_d \cdot \psi_{0.i.} \cdot 5q_{k.imp} \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot S \cdot \psi_{0.s} \cdot A_{trib}$$

$$Q_{column5.b.i} := \gamma_d \cdot 1.35 \cdot 0.89 \cdot \left[\left(5g_{k.floor} + g_{k.roof.timber}\right) \cdot A_{trib} + 5g_{k.beam} \cdot l_{span.GL}\right] \ldots = 886.227 \cdot kN$$
$$+ 1.5 \cdot \gamma_d \cdot 5q_{k.imp} \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot \psi_{0.s} \cdot S \cdot A_{trib}$$

Snow load as main load

$$Q_{column5.a.s} := \gamma_d \cdot 1.35 \cdot \left[\left(5g_{k.floor} + g_{k.roof.timber}\right) \cdot A_{trib} + 5g_{k.beam} \cdot l_{span.GL}\right] \ldots = 752.239 \cdot kN$$
$$+ 1.5 \cdot \gamma_d \cdot \psi_{0.s} \cdot S \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot 5q_{k.imp} \cdot \psi_{0.i.} \cdot A_{trib}$$

$$Q_{column5.b.s} := \gamma_d \cdot 1.35 \cdot 0.89 \cdot \left[\left(5g_{k.floor} + g_{k.roof.timber}\right) \cdot A_{trib} + 5g_{k.beam} \cdot l_{span.GL}\right] \ldots = 744.335 \cdot kN$$
$$+ 1.5 \cdot \gamma_d \cdot S \cdot A_{trib} + 1.5 \cdot \gamma_d \cdot \psi_{0.i.} \cdot 5q_{k.imp} \cdot A_{trib}$$

$$Q_{column5} := \max\left(Q_{column5.a.i}, Q_{column5.b.i}, Q_{column5.a.s}, Q_{column5.b.s}\right) = 886.227 \cdot kN$$

$$Q_{glulam.column} := \begin{pmatrix} Q_{column1} \\ Q_{column2} \\ Q_{column3} \\ Q_{column4} \\ Q_{column5} \end{pmatrix} = \begin{pmatrix} 220.502 \\ 386.933 \\ 553.364 \\ 719.796 \\ 886.227 \end{pmatrix} \cdot kN$$

Maximum vertical load in the columns for all cases upp to five added floors

5.3 Capacity due to buckling of the most loaded timber column

5.3.1 Geometry

$i := 0 .. 4$

Since calculations are for one to five added floors.

$$h_{glulam.col} := \begin{pmatrix} 180 \\ 225 \\ 270 \\ 315 \\ 360 \end{pmatrix} mm$$

Height of the column

$$w_{glulam.col} := \begin{pmatrix} 165 \\ 165 \\ 190 \\ 190 \\ 215 \end{pmatrix} mm$$

Width of the column

$$A_{glulam.col_i} := h_{glulam.col_i} \cdot w_{glulam.col_i}$$

Cross-sectional area of the column

$$A_{glulam.col} = \begin{pmatrix} 0.03 \\ 0.037 \\ 0.051 \\ 0.06 \\ 0.077 \end{pmatrix} m^2$$

5.3.2 Material data glulam GL30c

$f_{c.0.k.glulam} := 24.5MPa$

Compression parallel to grain

$E_{0.05.glulam} := 13000MPa$

Elastic modulus for glulam parallel to the grain

$\gamma_{M.glulam} := 1.25$

Partial factor

$k_{mod.glulam} := 0.7$

Strength modification factor. Assuming long term loading and service class 2.

$k_{def.glulam} := 0.8$

Deformation modification factor for service class 2

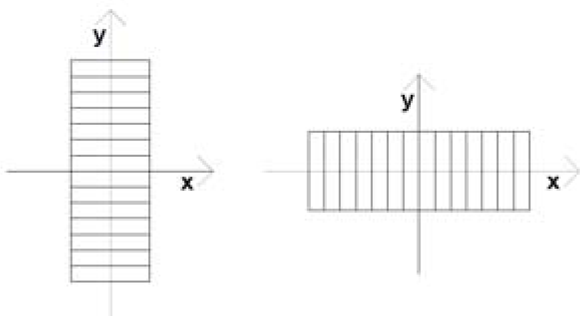
5.3.3 Size effect of member due to bending

$$k_{h.glulam.y_i} := \begin{cases} \min\left[\left(\dfrac{600mm}{h_{glulam.col_i}}\right)^{0.1}, 1.1\right] & \text{if } h_{glulam.col_i} \le 600mm \\ 1 & \text{otherwise} \end{cases}$$

Since h is less than 600 mm. EC 1995-1-1, Eq 3.2

$$k_{h.glulam.x_i} := \begin{cases} \min\left[\left(\dfrac{600mm}{w_{glulam.col_i}}\right)^{0.1}, 1.1\right] & \text{if } w_{glulam.col_i} \le 600mm \\ 1 & \text{otherwise} \end{cases}$$

Where y and x indicates strong respectively weak axis and is shown below



5.3.4 Design strength values

$$f_{c.0.d.glulam.y_i} := k_{mod.glulam} \cdot k_{h.glulam.y_i} \frac{f_{c.0.k.glulam}}{\gamma_{M.glulam}}$$

$$f_{c.0.d.glulam.x_i} := k_{mod.glulam} \cdot k_{h.glulam.x_i} \frac{f_{c.0.k.glulam}}{\gamma_{M.glulam}}$$

5.3.5 Moment of inertia

$$I_{glulam.col.y_i} := \frac{w_{glulam.col_i} \cdot \left(h_{glulam.col_i}\right)^3}{12}$$

$$I_{glulam.col.x_i} := \frac{h_{glulam.col_i} \cdot \left(w_{glulam.col_i}\right)^3}{12}$$

5.3.6 Radius of gyration

$$i_{glulam.col.y_i} := \sqrt{\frac{I_{glulam.col.y_i}}{A_{glulam.col_i}}}$$

$$i_{glulam.col.x_i} := \sqrt{\frac{I_{glulam.col.x_i}}{A_{glulam.col_i}}}$$

5.3.7 Slenderness ratio and relative slenderness

The slenderness depends on the boundary conditions and simply supported edges are assumed

$$\lambda_{glulam.col.y_i} := \frac{h_{column}}{i_{glulam.col.y_i}}$$

$$\lambda_{glulam.col.x_i} := \frac{h_{column}}{i_{glulam.col.x_i}}$$

Calculation of the relative slenderness

$$\lambda_{rel.glulam.col.y_i} := \frac{\lambda_{glulam.col.y_i}}{\pi} \cdot \sqrt{\frac{f_{c.0.k.glulam}}{E_{0.05.glulam}}}$$ EC 1995-1-1, Eq 6.27

$$\lambda_{rel.glulam.col.x_i} := \frac{\lambda_{glulam.col.x_i}}{\pi} \cdot \sqrt{\frac{f_{c.0.k.glulam}}{E_{0.05.glulam}}}$$ EC 1995-1-1, Eq 6.22

$$\lambda_{rel.glulam.col.y} = \begin{pmatrix} 0.745 \\ 0.596 \\ 0.496 \\ 0.426 \\ 0.372 \end{pmatrix}$$

The relative slenderness, λ.rel > 0.3. This indicates that the column needs to be checked against buckling.

$$\lambda_{rel.glulam.col.x} = \begin{pmatrix} 0.812 \\ 0.812 \\ 0.705 \\ 0.705 \\ 0.623 \end{pmatrix}$$

5.3.8 Strength reduction factor

$$\beta_{c.glulam} := 0.1$$

The factor is 0.1 for glulam elements. EC 1995-1-1, Eq 6.29

$$k_{glulam.col.y_i} := 0.5 \cdot \left[1 + \beta_{c.glulam} \cdot \left(\lambda_{rel.glulam.col.y_i} - 0.3\right) + \left(\lambda_{rel.glulam.col.y_i}\right)^2\right]$$

EC 1995-1-1, Eq 6.27

$$k_{glulam.col.x_i} := 0.5 \cdot \left[1 + \beta_{c.glulam} \cdot \left(\lambda_{rel.glulam.col.x_i} - 0.3\right) + \left(\lambda_{rel.glulam.col.x_i}\right)^2\right]$$

EC 1995-1-1, Eq 6.28

$$k_{c.glulam.col.y_i} := \frac{1}{k_{glulam.col.y_i} + \sqrt{\left(k_{glulam.col.y_i}\right)^2 - \left(\lambda_{rel.glulam.col.y_i}\right)^2}}$$

The instability factor. EC 1995-1-1, Eq 6.25

$$k_{c.glulam.col.x_i} := \frac{1}{k_{glulam.col.x_i} + \sqrt{\left(k_{glulam.col.x_i}\right)^2 - \left(\lambda_{rel.glulam.col.x_i}\right)^2}}$$

The instability factor. EC 1995-1-1, Eq 6.26

5.3.9 Critcal axial load

According to EC 1995-1-1, Eq 6.23 and 6.24, the dimensioning compression stress in both directions is calculated as:

$$\sigma_{c.0.d.glulam.col.y_i} := k_{c.glulam.col.y_i} \cdot f_{c.0.d.glulam.y_i}$$

$$\sigma_{c.0.d.glulam.col.x_i} := k_{c.glulam.col.x_i} \cdot f_{c.0.d.glulam.x_i}$$

The axial force in both directions is calculated according to EC 1995-1-1, Eq 6.36

$$N_{cr.glulam.col.y_i} := \sigma_{c.0.d.glulam.col.y_i} \cdot A_{glulam.col_i}$$

$$N_{cr.glulam.col.x_i} := \sigma_{c.0.d.glulam.col.x_i} \cdot A_{glulam.col_i}$$

Total maximum axial force allowed:

$$N_{cr.glulam.col_i} := \min\left(N_{cr.glulam.col.y_i}, N_{cr.glulam.col.x_i}\right)$$

5.4 Check - Capacity due to combined bending and shear

5.4.1 Bending moment

$F_{3.L2.5} := 10.601\,\frac{kN}{m}$ $\quad F_{3.L3.5} := 9.395\,\frac{kN}{m}$

These values are calculated in Appendix 9

$F_{4.L2.5} := 10.795\,\frac{kN}{m}$ $\quad F_{4.L3.5} := 9.567\,\frac{kN}{m}$

$$M_{Ed.column.L2} := \frac{\left(\frac{F_{3.L2.5}+F_{4.L2.5}}{2}\right)\cdot h_{column}^{\,2}}{12} = 6.989\cdot kN\cdot m$$

Maximum bending moment for a column with fixed end caused by the wind load on facade L2

$$M_{Ed.column.L3} := \frac{\left(\frac{F_{3.L3.5}+F_{4.L3.5}}{2}\right)\cdot h_{column}^{\,2}}{12} = 6.194\cdot kN\cdot m$$

Maximum bending moment for a column with fixed end caused by the wind load on facade L2

$$M_{Ed.column} := \max\left(M_{Ed.column.L2}, M_{Ed.column.L3}\right) = 6.989\cdot kN\cdot m$$

$$\sigma_{m.y.d.column_i} := \frac{6\cdot M_{Ed.column}}{w_{glulam.col_i}\cdot\left(h_{glulam.col_i}\right)^2}$$

$$\sigma_{m.x.d.column_i} := \frac{6\cdot M_{Ed.column}}{h_{glulam.col_i}\cdot\left(w_{glulam.col_i}\right)^2}$$

5.4.2 Design strength values due to bending in strong direction - ULS

$$f_{m.d.glulam.col.y_i} := k_{mod.glulam}\cdot k_{h.glulam.y_i}\,\frac{f_{m.g.k.glulam}}{\gamma_{M.glulam}}$$

Design value for bending stress parallell to the grain. EC 1995-1-1, Eq 2.17

$$f_{m.d.glulam.col.x_i} := k_{mod.glulam}\cdot k_{h.glulam.x_i}\,\frac{f_{m.g.k.glulam}}{\gamma_{M.glulam}}$$

5.4.3 Actual loads and resisting capcacity

$$Q_{glulam.column} = \begin{pmatrix} 220.502 \\ 386.933 \\ 553.364 \\ 719.796 \\ 886.227 \end{pmatrix}\cdot kN \qquad N_{cr.glulam.col.y} = \begin{pmatrix} 411.05 \\ 536.276 \\ 743.131 \\ 862.619 \\ 1.108\times 10^3 \end{pmatrix}\cdot kN$$

$$\sigma_{m.y.d.column} = \begin{pmatrix} 7.844 \\ 5.02 \\ 3.028 \\ 2.224 \\ 1.505 \end{pmatrix} \cdot MPa \qquad f_{m.d.glulam.col.y} = \begin{pmatrix} 18.48 \\ 18.48 \\ 18.197 \\ 17.918 \\ 17.68 \end{pmatrix} \cdot MPa$$

Utilization ratio

$$check_{y_i} := \frac{Q_{glulam.column_i}}{N_{cr.glulam.col.y_i}} + \frac{\sigma_{m.y.d.column_i}}{f_{m.d.glulam.col.y_i}}$$

Criterion for combined bending moment and compression. EC 1995-1-1, Eq 6.19

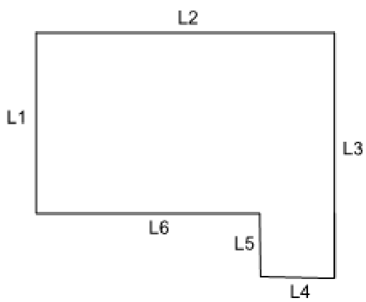
$$check_y = \begin{pmatrix} 96.092 \\ 99.319 \\ 91.103 \\ 95.857 \\ 88.476 \end{pmatrix} \cdot \%$$

OK !

Appendix 6 - Wind load on the reference building

6.1 Wind load

The wind load is dependent on geographical location as well as sourrounding environment



It is assumed that the wind load acting on facade L2 is equal to opposing side, L4+L6.

The same assumption is made for facade L3 wich is equal to L5+L1.

6.1.1 Peak velocity pressure

$v_{b.0} := 25\frac{m}{s}$ Wind velocity in Gothenburg

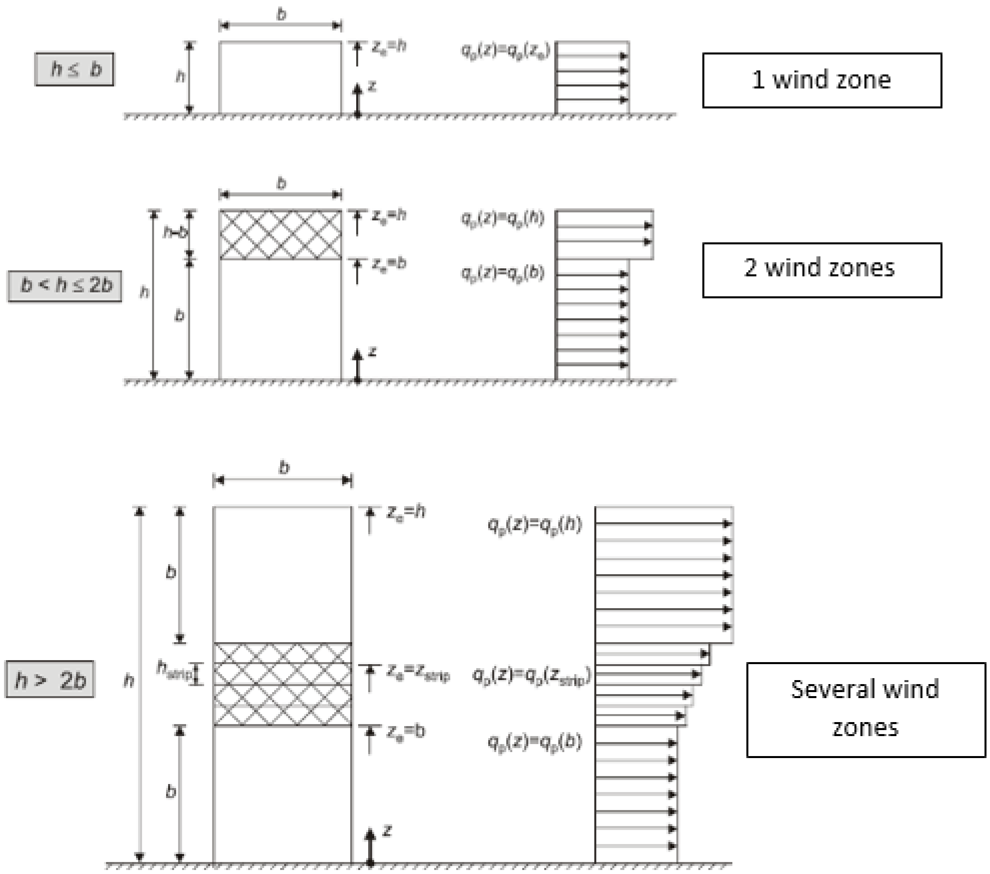
$c_{dir} := 1$ Direction factor, recomended value

$c_{season} := 1$ Seasonal factor, recomended value

$v_b := v_{b.0} \cdot c_{dir} \cdot c_{season} = 25\frac{m}{s}$ Basic wind velocity. EC 1991-1-4, Eq 4.1

6.1.2 Height of the wind zones

The acting wind load on the building should be divided into different zones depending on the ratio between the height and width of the building. According to EC 1991-1-4 Section 7.2.2, three different cases exist wich is illustrated in the picture below.



$$h_{ref.L2} := \begin{cases} \text{"1 zone"} & \text{if } h_{ref} \le L_2 \\ \text{"2 zones"} & \text{if } L_2 < h_{ref} \le 2 \cdot L_2 \\ \text{"Several zones"} & \text{otherwise} \end{cases} = \text{"1 zone"}$$

When the wind is acting on facade L2, one zone is applied, which means that one wind zone is acting on the building

$$h_{ref.L3} := \begin{cases} \text{"1 zone"} & \text{if } h_{ref} \le L_3 \\ \text{"2 zones"} & \text{if } L_3 < h_{ref} \le 2 \cdot L_3 \\ \text{"Several zones"} & \text{otherwise} \end{cases} = \text{"1 zone"}$$

When the wind is acting on facade L3, one zone is applied

$$z_{ref.L2} := h_{ref} = 19.2\,m$$

Height of wind zone

$$z_{ref.L3} := h_{ref} = 19.2\,m$$

Height of wind zone

6.1.3 Mean wind velocity

$z_0 := 1.0\text{m}$

Terrain roughness factor for category IV. EC 1991-1-4, Table 4.1

$z_{0.II} := 0.05\text{m}$

$k_r := 0.19 \cdot \left(\frac{z_0}{z_{0.II}}\right)^{0.07} = 0.234$

Terrain factor. EC 1991-1-4, Eq 4.5

$c_0 := 1$

Topography factor, recomended value

Wind on facade L2 and L3

The mean wind velocity is the same for both facades.

$c_{r.ref} := k_r \cdot \ln\left(\frac{z_{ref.L2}}{z_0}\right) = 0.692$

Roughness factor for the reference building. The factor is the same for both facades. EC 1991-1-4, Eq 4.4

$v_{m.ref} := c_{r.ref} \cdot c_0 \cdot v_b = 17.311 \frac{\text{m}}{\text{s}}$

Mean wind velocity for the reference buidling. EC 1991-1-4, Eq 4.3

6.1.4 Wind turbulence

$k_l := 1.0$

Wind turbulence factor, recommended value. EC 1991-1-4, Section 4.4

$\sigma_v := k_r \cdot v_b \cdot k_l = 5.858 \frac{\text{m}}{\text{s}}$

Standard deviation of the turbulence EC 1991-1-4, Eq 4.6

Wind on facade L2 and L3

$l_{v.ref} := \frac{\sigma_v}{v_{m.ref}} = 0.338$

Wind turbulence acting on the reference buildning. The factor is the same for the both facades. EC 1991-1-4, Eq 4.7

6.1.5 Characteristic veolcity pressure

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

Density of air

Wind on facade L2 and L3

$q_{p.ref} := \left(1 + 7 + l_{v.ref}\right) \cdot \frac{1}{2} \cdot \rho_{air} \cdot {v_{m.ref}}^2 = 1.562 \cdot \text{kPa}$

Characteristic velocity pressure acting on the reference building. The pressure is the same for both facades. EC 1991-1-4, Eq 4.8

6.1.6 Peak veolcity pressure

Wind on facade L2

$e_{L2} := \min\left(L_2, 2 \cdot h_{ref}\right) = 33.5\,m$

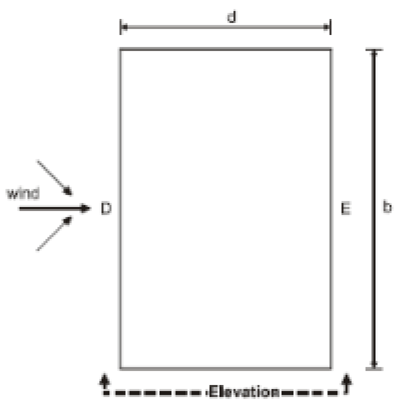
$d_{L2} := L_3 = 27.6\,m$

Wind on facade L3

$e_{L3} := \min\left(L_3, 2 \cdot h_{ref}\right) = 27.6\,m$

$d_{L3} := L_2 = 33.5\,m$

Due to criteria form Eurocode 1991-1-4, Section 7.2 the pressure coefficients should be determined according to the picture below.



According to EC 1991-1-4 Table 7.1, the shape factors for the wind load on external walls are determined. For intermediate values of the ratio (h/d), linear interpolation should be used.

$$ratio_{L2.ref} := \frac{h_{ref}}{d_{L2}} = 0.696$$

$$ratio_{L3.ref} := \frac{h_{ref}}{d_{L3}} = 0.573$$

$$C_{pe.10.D.L2.ref} := 0.7 + (0.8 - 0.7) \cdot \frac{\left(ratio_{L2.ref} - 0.25\right)}{1 - 0.25} = 0.759$$

$$C_{pe.10.D.L3.ref} := 0.7 + (0.8 - 0.7) \cdot \frac{\left(ratio_{L3.ref} - 0.25\right)}{1 - 0.25} = 0.743$$

Shape factors for the windward side

$$C_{pe.10.E.L2.ref} := -0.3 + (-0.5 + 0.3) \cdot \frac{\left(ratio_{L2.ref} - 0.25\right)}{1 - 0.25} = -0.419$$

$$C_{pe.10.E.L3.ref} := -0.3 + (-0.5 + 0.3) \cdot \frac{\left(ratio_{L3.ref} - 0.25\right)}{1 - 0.25} = -0.386$$

Shape factors for the leeward side

6.1.7 Wind pressure on the facades

Wind pressure on facade L2

$w_{L2.ref.D} := q_{p.ref} \cdot C_{pe.10.D.L2.ref} = 1.186 \cdot kPa$

Wind pressure, EC 1991-1-4 Eq. 5.1

$w_{L2.ref.E} := q_{p.ref} \cdot C_{pe.10.E.L2.ref} = -0.654 \cdot kPa$

Wind pressure on facade L3

$w_{L3.ref.D} := q_{p.ref} \cdot C_{pe.10.D.L3.ref} = 1.16 \cdot kPa$

$w_{L3.ref.E} := q_{p.ref} \cdot C_{pe.10.E.L3.ref} = -0.603 \cdot kPa$

6.1.8 Total wind pressure on the facades

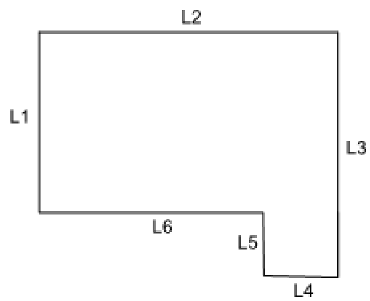
$w_{L2.ref} := w_{L2.ref.D} - w_{L2.ref.E} = 1.84 \cdot kPa$

$w_{L3.ref} := w_{L3.ref.D} - w_{L3.ref.E} = 1.763 \cdot kPa$

Appendix 7 - Wind loads on added floors

The calculations for the wind loads in this Appendix is made for cases were one to five floors are added on the existing building.

Case 1: One floor added
Case 2: Two floors added
Case 3: Three floors added
Case 4: Four floors added
Case 5: Five floors added



The geometry of the building

$L_2 = 33.5\,m$

$L_3 = 27.6\,m$

7.1 Case 1 - One floor is added

7.1.1 Geometry

$h_{floor.tim} := t_{tf.floor} + h_{web.floor} + t_{bf.floor} = 0.349\,m$ Height of the casette floor

$h_{beam.glulam} = 0.585\,m$ Height of glulam beam

$h_{column} = 2.8\,m$ Height of the columns

$h_{tot.floor.tim} := h_{floor.tim} + h_{beam.glulam} + h_{column} = 3.734\,m$

Total height of one floor for the new construction

$h_{roof.timber} := h_{beam.glulam} + h_{roof} = 0.985\,m$ Height of the timber roof

$H_{case1} := h_{ref} + h_{tot.floor.tim} + h_{roof} = 23.334\,m$ Total height of the building if one more floor is added

7.1.2 Height of the wind zones

$$h_{case1.L2} := \begin{cases} \text{"1 zone"} & \text{if } H_{case1} \le L_2 \\ \text{"2 zones"} & \text{if } L_2 < H_{case1} \le 2 \cdot L_2 \\ \text{"Several zones"} & \text{otherwise} \end{cases} = \text{"1 zone"}$$

When the wind is acting on facade L2 one zone is applied

$$h_{case1.L3} := \begin{cases} \text{"1 zone"} & \text{if } H_{case1} \le L_3 \\ \text{"2 zones"} & \text{if } L_3 < H_{case1} \le 2 \cdot L_3 \\ \text{"Several zones"} & \text{otherwise} \end{cases} = \text{"1 zone"}$$

When the wind is acting on facade L3 one zone is applied.

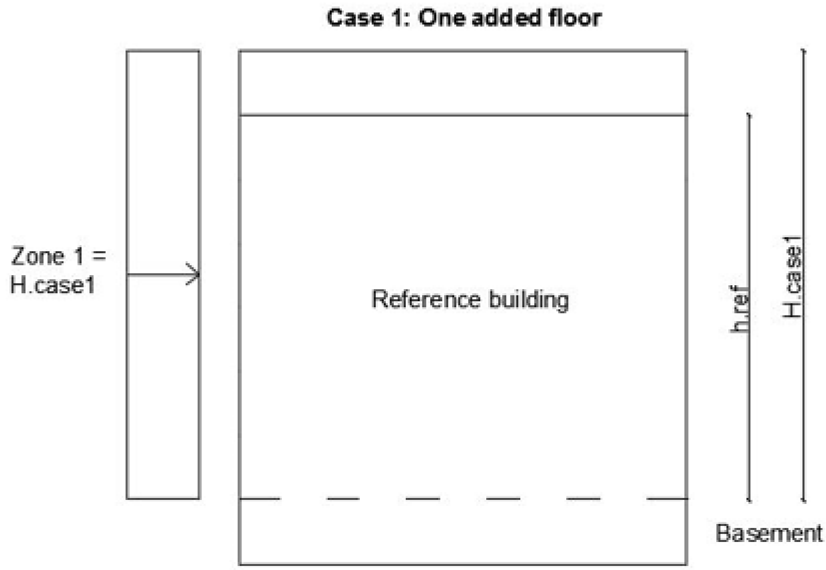
$$z_{case1.L2} := H_{case1} = 23.334\,m$$

Height of wind zone for facade L2

$$z_{case1.L3} := H_{case1} = 23.334\,m$$

Height of wind zone for facade L3

The wind load are the same for both facade L2 and L3.



7.1.3 Mean wind velocity

Wind on facade L2 and L3

$$c_{r.case1} := k_r \cdot \ln\left(\frac{z_{case1.L2}}{z_0}\right) = 0.738$$

Roughness factor for Case 1. The factor is the same for both facades. EC 1991-1-4, Eq 4.4

$$v_{m.case1} := c_{r.case1} \cdot c_0 \cdot v_b = 18.453\,\frac{m}{s}$$

Mean wind velocity for Case 1.
EC 1991-1-4, Eq 4.3

7.1.4 Wind turbulence

Wind on facade L2 and L3

$$l_{v.case1} := \frac{\sigma_v}{v_{m.case1}} = 0.317$$

Wind turbulence for Case 1. The factor is the same for both facades.
EC 1991-1-4, Eq 4.7

7.1.5 Characteristic veolcity pressure

Wind on facade L2 and L3

$$q_{p.case1} := \left(1 + 7 + l_{v.case1}\right)\cdot\frac{1}{2}\cdot\rho_{air}\cdot v_{m.case1}{}^2 = 1.77\cdot kPa$$

Characteristic velocity pressure for Case 1. The pressure is the same for both facades. EC 1991-1-4. Eq 4.8

7.1.6 Peak veolcity pressure

Wind on facade L2

$$d_{case1.L2} := L_3 = 27.6\,m$$

Wind on facade L3

$$d_{case1.L3} := L_2 = 33.5\,m$$

According to EC 1991-1-4 Table 7.1, the shape factors for the wind load on external walls are determined. For intermediate values of the ratio (h/d), linear interpolation should be used

$$ratio_{L2.case1} := \frac{H_{case1}}{d_{L2}} = 0.845$$

$$ratio_{L3.case1} := \frac{H_{case1}}{d_{L3}} = 0.697$$

$$C_{pe.10.D.L2.case1} := 0.7 + (0.8 - 0.7)\cdot\frac{\left(ratio_{L2.case1} - 0.25\right)}{1 - 0.25} = 0.779$$

$$C_{pe.10.D.L3.case1} := 0.7 + (0.8 - 0.7)\cdot\frac{\left(ratio_{L3.case1} - 0.25\right)}{1 - 0.25} = 0.76$$

Shape factor for the windward side

$$C_{pe.10.E.L2.case1} := -0.3 + (-0.5 + 0.3)\cdot\frac{\left(ratio_{L2.case1} - 0.25\right)}{1 - 0.25} = -0.459$$

$$C_{pe.10.E.L3.case1} := -0.3 + (-0.5 + 0.3)\cdot\frac{\left(ratio_{L3.case1} - 0.25\right)}{1 - 0.25} = -0.419$$

Shape factor for the leeward side

7.1.7 Wind pressure on the facades

Wind pressure on facade L2

$$w_{L2.case1.D} := q_{p.case1}\cdot C_{pe.10.D.L2.case1} = 1.38\cdot kPa$$

Wind pressure, EC 1991-1-4 Eq. 5.1

$$w_{L2.case1.E} := q_{p.case1}\cdot C_{pe.10.E.L2.case1} = -0.812\cdot kPa$$

Wind pressure on facade L3

$$w_{L3.case1.D} := q_{p.case1}\cdot C_{pe.10.D.L3.case1} = 1.344\cdot kPa$$

$w_{L3.case1.E} := q_{p.case1} \cdot C_{pe.10.E.L3.case1} = -0.742 \cdot kPa$

7.1.8 Total wind pressure on the facades

$w_{L2.case1} := w_{L2.case1.D} - w_{L2.case1.E} = 2.192 \cdot kPa$

$w_{L3.case1} := w_{L3.case1.D} - w_{L3.case1.E} = 2.086 \cdot kPa$

7.2 Case 2 - Two floors are added

$H_{case2} := h_{ref} + 2h_{tot.floor.tim} + h_{roof} = 27.068\,m$ Total height if two more floors are added

7.2.1 Height of the wind zones

$$h_{case2.L2} := \begin{vmatrix} \text{"1 zone"} & \text{if } H_{case2} \le L_2 \\ \text{"2 zones"} & \text{if } L_2 < H_{case2} \le 2 \cdot L_2 \\ \text{"Several zones"} & \text{otherwise} \end{vmatrix} = \text{"1 zone"}$$

When the wind is acting on facade L2 one zone is applied

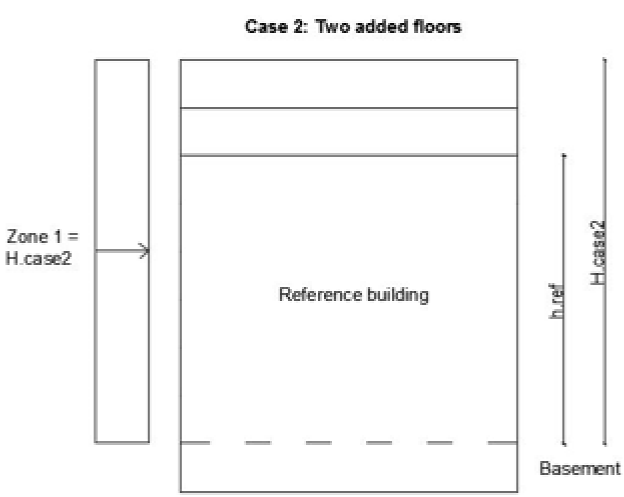
$$h_{case2.L3} := \begin{vmatrix} \text{"1 zone"} & \text{if } H_{case2} \le L_3 \\ \text{"2 zones"} & \text{if } L_3 < H_{case2} \le 2 \cdot L_3 \\ \text{"Several zones"} & \text{otherwise} \end{vmatrix} = \text{"1 zone"}$$

When the wind is acting on facade L3 one zone is applied.

$z_{case2.L2} := H_{case2} = 27.068\,m$ Height of wind zone

$z_{case2.L3} := H_{case2} = 27.068\,m$ Height of wind zone

The wind load are the same for both facade L2 and L3.



7.2.2 Mean wind velocity

Wind on facade L2 and L3

$$c_{r.case2} := k_r \cdot \ln\left(\frac{z_{case2.L2}}{z_0}\right) = 0.773$$

Roughness factor for Case 2. The factor is the same for both facades. EC 1991-1-4, Eq 4.4

$$v_{m.case2} := c_{r.case2} \cdot c_0 \cdot v_b = 19.322\,\frac{m}{s}$$

Mean wind velocity for Case 2. EC 1991-1-4, Eq 4.3

7.2.3 Wind turbulence

Wind on facade L2 and L3

$$l_{v.case2} := \frac{\sigma_v}{v_{m.case2}} = 0.303$$

Wind turbulence for Case 2. The factor is the same for both facades. EC 1991-1-4, Eq 4.7

7.2.4 Characteristic veolcity pressure

Wind on facade L2 and L3

$$q_{p.case2} := \left(1 + 7 + l_{v.case2}\right) \cdot \frac{1}{2} \cdot \rho_{air} \cdot v_{m.case2}^{\;2} = 1.938 \cdot kPa$$

Characteristic velocity pressure for Case 2. The pressure is the same for both facades. EC 1991-1-4. Eq 4.8

7.2.5 Peak veolcity pressure

Wind on facade L2

$$d_{case2.L2} := L_3 = 27.6\,m$$

Wind on facade L3

$$d_{case2.L3} := L_2 = 33.5\,m$$

According to EC 1991-1-4 Table 7.1, the shape factors for the wind load on external walls are determined. For intermediate values of the ratio (h/d), linear interpolation should be used.

$$ratio_{L2.case2} := \frac{H_{case2}}{d_{L2}} = 0.981$$

$$ratio_{L3.case2} := \frac{H_{case2}}{d_{L3}} = 0.808$$

$$C_{pe.10.D.L2.case2} := 0.7 + (0.8 - 0.7) \cdot \frac{\left(ratio_{L2.case2} - 0.25\right)}{1 - 0.25} = 0.797$$

$$C_{pe.10.D.L3.case2} := 0.7 + (0.8 - 0.7) \cdot \frac{\left(ratio_{L3.case2} - 0.25\right)}{1 - 0.25} = 0.774$$

Shape factor for the windward side

$$C_{pe.10.E.L2.case2} := -0.3 + (-0.5 + 0.3) \cdot \frac{(ratio_{L2.case2} - 0.25)}{1 - 0.25} = -0.495$$

$$C_{pe.10.E.L3.case2} := -0.3 + (-0.5 + 0.3) \cdot \frac{(ratio_{L3.case2} - 0.25)}{1 - 0.25} = -0.449$$

Shape factor for the leeward side

7.2.6 Wind pressure on the facades

Wind pressure on facade L2

$$w_{L2.case2.D} := q_{p.case2} \cdot C_{pe.10.D.L2.case2} = 1.545 \cdot kPa$$

Wind pressure, EC 1991-1-4 Eq. 5.1

$$w_{L2.case2.E} := q_{p.case2} \cdot C_{pe.10.E.L2.case2} = -0.959 \cdot kPa$$

Wind pressure on facade L3

$$w_{L3.case2.D} := q_{p.case2} \cdot C_{pe.10.D.L3.case2} = 1.5 \cdot kPa$$

$$w_{L3.case2.E} := q_{p.case2} \cdot C_{pe.10.E.L3.case2} = -0.87 \cdot kPa$$

7.2.7 Total wind pressure on the facades

$$w_{L2.case2} := w_{L2.case2.D} - w_{L2.case2.E} = 2.504 \cdot kPa$$

$$w_{L3.case2} := w_{L3.case2.D} - w_{L3.case2.E} = 2.37 \cdot kPa$$

7.3 Case 3 - Three floors are added

$$H_{case3} := h_{ref} + 3h_{tot.floor.tim} + h_{roof} = 30.802\,m$$

Total height if three more floors are added

7.3.1 Height of the wind zones

$$h_{case3.L2} := \begin{array}{|l} \text{"1 zone"} \quad \text{if } H_{case3} \le L_2 \\ \text{"2 zones"} \quad \text{if } L_2 < H_{case3} \le 2 \cdot L_2 \\ \text{"Several zones"} \quad \text{otherwise} \end{array} = \text{"1 zone"}$$

When the wind is acting on facade L2 one zone is applied

$$h_{case3.L3} := \begin{array}{|l} \text{"1 zone"} \quad \text{if } H_{case3} \le L_3 \\ \text{"2 zones"} \quad \text{if } L_3 < H_{case3} \le 2 \cdot L_3 \\ \text{"Several zones"} \quad \text{otherwise} \end{array} = \text{"2 zones"}$$

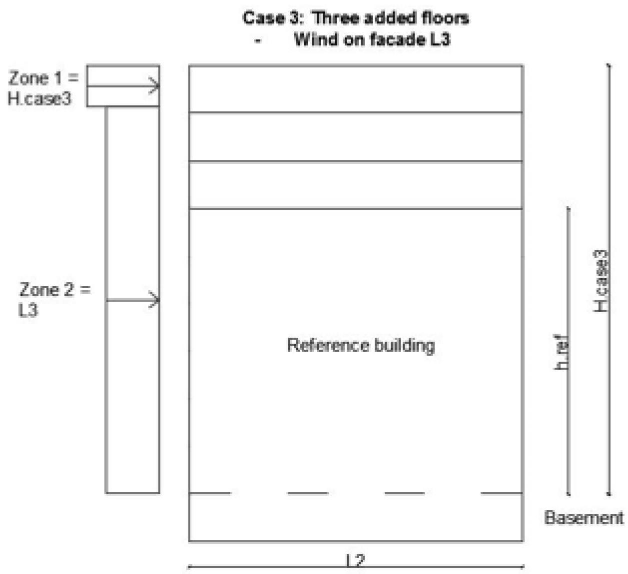
When the wind is acting on facade L3 two zones are applied

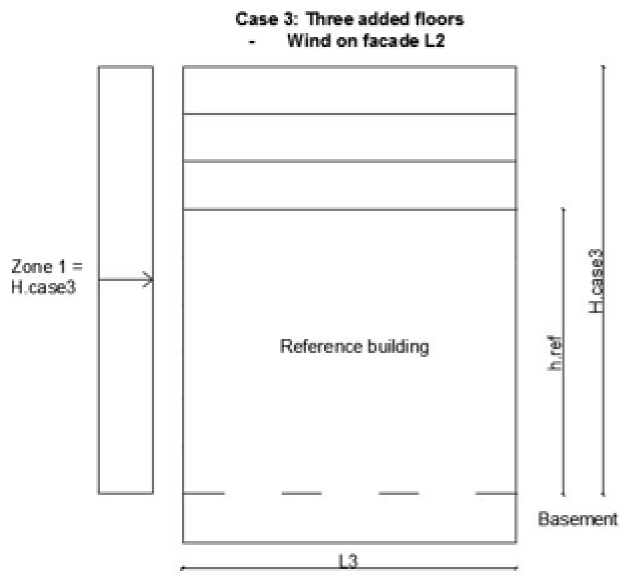
$z_{case3.L2} := H_{case3} = 30.802\,m$ Height of wind zone for facade L2

$z_{case3.L3.zone1} := H_{case3} = 30.802\,m$ Height of wind zone 1 for facade L3

$z_{case3.L3.zone2} := L_3 = 27.6\,m$ Height of wind zone 2 for facade L3

The wind pressure is no longer the same for both facades. For case 3, two wind zones are applied on facade L3 and one zone for facade L2. This is shown in the figures below.





7.3.2 Mean wind velocity

Wind on facade L2

$$c_{r.case3.L2} := k_r \cdot \ln\left(\frac{z_{case3.L2}}{z_0}\right) = 0.803$$

Roughness factor for Case 3. EC 1991-1-4, Eq 4.4

$$v_{m.case3.L2} := c_{r.case3.L2} \cdot c_0 \cdot v_b = 20.08\,\frac{m}{s}$$

Mean wind velocity for Case 3. EC 1991-1-4, Eq 4.3

Wind on facade L3

$$c_{r.case3.L3.zone1} := k_r \cdot \ln\left(\frac{z_{case3.L3.zone1}}{z_0}\right) = 0.803$$

Roughness factor for Case 3. EC 1991-1-4, Eq 4.4

$$c_{r.case3.L3.zone2} := k_r \cdot \ln\left(\frac{z_{case3.L3.zone2}}{z_0}\right) = 0.777$$

$$v_{m.case3.L3.zone1} := c_{r.case3.L3.zone1} \cdot c_0 \cdot v_b = 20.08\,\frac{m}{s}$$

Mean wind velocity for Case 3. EC 1991-1-4, Eq 4.3

$$v_{m.case3.L3.zone2} := c_{r.case3.L3.zone2} \cdot c_0 \cdot v_b = 19.436\,\frac{m}{s}$$

7.3.3 Wind turbulence

Wind on facade L2

$$l_{v.case3.L2} := \frac{\sigma_v}{v_{m.case3.L2}} = 0.292$$

Wind turbulence for Case 3. EC 1991-1-4, Eq 4.7

Wind on facade L3

$$l_{v.case3.L3.zone1} := \frac{\sigma_v}{v_{m.case3.L3.zone1}} = 0.292$$

Wind turbulence for Case 3. EC 1991-1-4, Eq 4.7

$$l_{v.case3.L3.zone2} := \frac{\sigma_v}{v_{m.case3.L3.zone2}} = 0.301$$

7.3.4 Characteristic veolcity pressure

Wind on facade L2

$$q_{p.case3.L2} := \left(1 + 7 + l_{v.case3.L2}\right) \cdot \frac{1}{2} \cdot \rho_{air} \cdot v_{m.case3.L2}^{\;2} = 2.089 \cdot kPa$$

Characteristic velocity pressure for Case 3. EC 1991-1-4, Eq 4.8

Wind on facade L3

$$q_{p.case3.L3.zone1} := \left(1 + 7 + l_{v.case3.L3.zone1}\right)\cdot\frac{1}{2}\cdot\rho_{air}\cdot v_{m.case3.L3.zone1}{}^{2} = 2.089\cdot kPa$$

$$q_{p.case3.L3.zone2} := \left(1 + 7 + l_{v.case3.L3.zone2}\right)\cdot\frac{1}{2}\cdot\rho_{air}\cdot v_{m.case3.L3.zone2}{}^{2} = 1.96\cdot kPa$$

7.3.5 Peak veolcity pressure

Wind on facade L2

$$d_{case3.L2} := L_3 = 27.6\,m$$

Wind on facade L3

$$d_{case3.L3} := L_2 = 33.5\,m$$

According to EC 1991-1-4 Table 7.1, the shape factors for the wind load on external walls are determined. For intermediate values of the ratio (h/d), linear interpolation should be used

$$ratio_{L2.case3} := \frac{H_{case3}}{d_{L2}} = 1.116$$

$$ratio_{L3.case3} := \frac{H_{case3}}{d_{L3}} = 0.919$$

$$C_{pe.10.D.L2.case3} := 0.8$$

$$C_{pe.10.D.L3.case3} := 0.7 + (0.8 - 0.7)\cdot\frac{\left(ratio_{L3.case3} - 0.25\right)}{1 - 0.25} = 0.789$$

Shape factor for the windward side

$$C_{pe.10.E.L2.case3} := -0.5 + (-0.7 + 0.5)\cdot\frac{\left(ratio_{L2.case3} - 1\right)}{5 - 1} = -0.506$$

$$C_{pe.10.E.L3.case3} := -0.3 + (-0.5 + 0.3)\cdot\frac{\left(ratio_{L3.case3} - 0.25\right)}{1 - 0.25} = -0.479$$

Shape factor for the leeward side

7.3.6 Wind pressure on the facades

Wind pressure on facade L2

$$w_{L2.case3.D} := q_{p.case3.L2}\cdot C_{pe.10.D.L2.case3} = 1.672\cdot kPa$$ Wind pressure, EC 1991-1-4 Eq, 5.1

$$w_{L2.case3.E} := q_{p.case3.L2}\cdot C_{pe.10.E.L2.case3} = -1.057\cdot kPa$$

Wind pressure on facade L3

$$w_{L3.case3.D.zone1} := q_{p.case3.L3.zone1}\cdot C_{pe.10.D.L3.case3} = 1.649\cdot kPa$$ Zone 1

$w_{L3.case3.D.zone2} := q_{p.case3.L3.zone2} \cdot C_{pe.10.D.L3.case3} = 1.547 \cdot kPa$ Zone 2

$w_{L3.case3.E.zone1} := q_{p.case3.L3.zone1} \cdot C_{pe.10.E.L3.case3} = -1 \cdot kPa$

$w_{L3.case3.E.zone2} := q_{p.case3.L3.zone2} \cdot C_{pe.10.E.L3.case3} = -0.938 \cdot kPa$

7.3.7 Total wind pressure on the facades

$w_{L2.case3} := w_{L2.case3.D} - w_{L2.case3.E} = 2.728 \cdot kPa$ Facade L2

$w_{L3.case3.zone1} := w_{L3.case3.D.zone1} - w_{L3.case3.E.zone1} = 2.649 \cdot kPa$ Facade L3 - zone 1

$w_{L3.case3.zone2} := w_{L3.case3.D.zone2} - w_{L3.case3.E.zone2} = 2.485 \cdot kPa$ Facade L3 - zone 2

7.4 Case 4 - Four floors are added

$H_{case4} := h_{ref} + 4h_{tot.floor.tim} + h_{roof} = 34.536\,m$ Total height if three more floors are added

7.4.1 Height of wind zones

$$h_{case4.L2} := \begin{cases} \text{"1 zone"} & \text{if } H_{case4} \le L_2 \\ \text{"2 zones"} & \text{if } L_2 < H_{case4} \le 2 \cdot L_2 \\ \text{"Several zones"} & \text{otherwise} \end{cases} = \text{"2 zones"}$$

When the wind is acting on facade L2 two zones are applied

$$h_{case4.L3} := \begin{cases} \text{"1 zone"} & \text{if } H_{case4} \le L_3 \\ \text{"2 zones"} & \text{if } L_3 < H_{case4} \le 2 \cdot L_3 \\ \text{"Several zones"} & \text{otherwise} \end{cases} = \text{"2 zones"}$$

When the wind is acting on facade L3 two zones are applied.

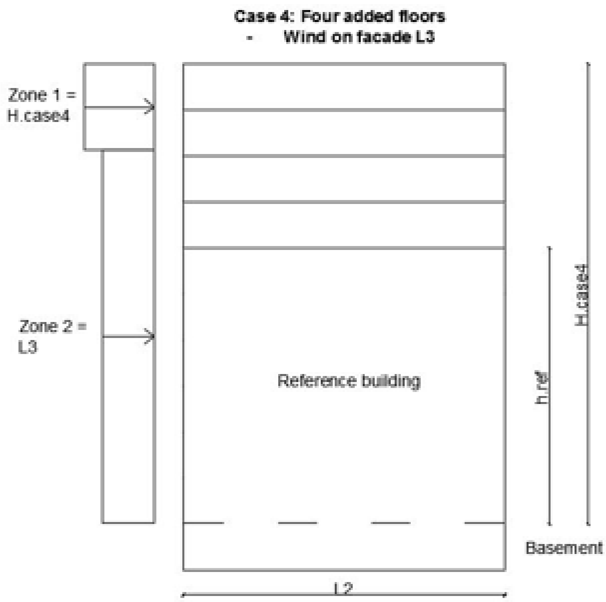
$z_{case4.L2.zone1} := H_{case4} = 34.536\,m$ Height of wind zone 1 for facade L2

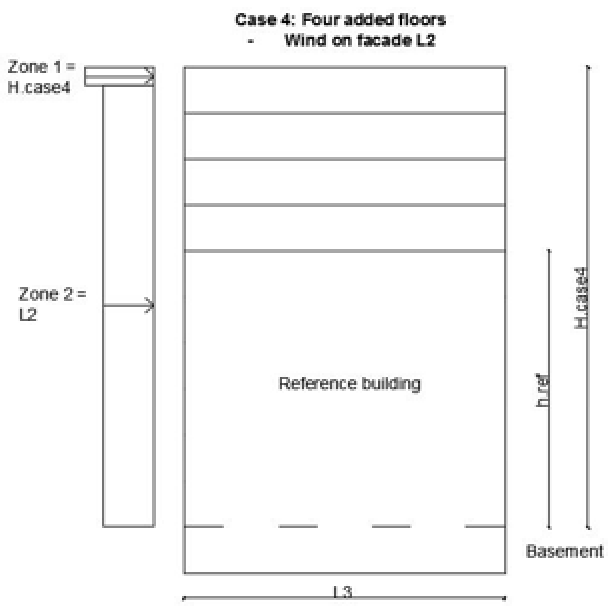
$z_{case4.L2.zone2} := L_2 = 33.5\,m$ Height of wind zone 2 for facade L2

$z_{case4.L3.zone1} := H_{case4} = 34.536\,m$ Height of wind zone 1 for facade L3

$z_{case4.L3.zone2} := L_3 = 27.6\,m$ Height of wind zone 2 for facade L3

For case 4, two wind zones are applied on both facade L2 and L3. This is shown in the figures below.





7.4.2 Mean wind velocity

Wind on facade L2

$$c_{r.case4.L2.zone1} := k_r \cdot \ln\left(\frac{z_{case4.L2.zone1}}{z_0}\right) = 0.83$$

Roughness factor for Case 4.
EC 1991-1-4, Eq 4.4

$$c_{r.case4.L2.zone2} := k_r \cdot \ln\left(\frac{z_{case4.L2.zone2}}{z_0}\right) = 0.823$$

$$v_{m.case4.L2.zone1} := c_{r.case4.L2.zone1} \cdot c_0 \cdot v_b = 20.75\,\frac{m}{s}$$

Mean wind velocity for Case 4.
EC 1991-1-4, Eq 4.3

$$v_{m.case4.L2.zone2} := c_{r.case4.L2.zone2} \cdot c_0 \cdot v_b = 20.571\,\frac{m}{s}$$

Wind on facade L3

$$c_{r.case4.L3.zone1} := k_r \cdot \ln\left(\frac{z_{case4.L3.zone1}}{z_0}\right) = 0.83$$

Roughness factor for Case 4.
EC 1991-1-4, Eq 4.4

$$c_{r.case4.L3.zone2} := k_r \cdot \ln\left(\frac{z_{case4.L3.zone2}}{z_0}\right) = 0.777$$

$$v_{m.case4.L3.zone1} := c_{r.case4.L3.zone1} \cdot c_0 \cdot v_b = 20.75\,\frac{m}{s}$$

Mean wind velocity for Case 4.
EC 1991-1-4, Eq 4.3

$$v_{m.case4.L3.zone2} := c_{r.case4.L3.zone2} \cdot c_0 \cdot v_b = 19.436\,\frac{m}{s}$$

7.4.3 Wind turbulence

Wind on facade L2

$$I_{v.case4.L2.zone1} := \frac{\sigma_v}{v_{m.case4.L2.zone1}} = 0.282$$

Wind turbulence for Case 4.
EC 1991-1-4, Eq 4.7

$$I_{v.case4.L2.zone2} := \frac{\sigma_v}{v_{m.case4.L2.zone2}} = 0.285$$

Wind on facade L3

$$I_{v.case4.L3.zone1} := \frac{\sigma_v}{v_{m.case4.L3.zone1}} = 0.282$$

Wind turbulence for Case 4.
EC 1991-1-4, Eq 4.7

$$I_{v.case4.L3.zone2} := \frac{\sigma_v}{v_{m.case4.L3.zone2}} = 0.301$$

7.4.4 Characteristic veolcity pressure

The characteristic velocity pressure is calculated according to EC 1991-1-4, Eq 4.8

Wind on facade L2

$$q_{p.case4.L2.zone1} := \left(1 + 7 + I_{v.case4.L2.zone1}\right) \cdot \frac{1}{2} \cdot \rho_{air} \cdot v_{m.case4.L2.zone1}^{\;2} = 2.229 \cdot kPa$$

$$q_{p.case4.L2.zone2} := \left(1 + 7 + I_{v.case4.L2.zone2}\right) \cdot \frac{1}{2} \cdot \rho_{air} \cdot v_{m.case4.L2.zone2}{}^2 = 2.191 \cdot kPa$$

Wind on facade L3

$$q_{p.case4.L3.zone1} := \left(1 + 7 + I_{v.case4.L3.zone1}\right) \cdot \frac{1}{2} \cdot \rho_{air} \cdot v_{m.case4.L3.zone1}{}^2 = 2.229 \cdot kPa$$

$$q_{p.case4.L3.zone2} := \left(1 + 7 + I_{v.case4.L3.zone2}\right) \cdot \frac{1}{2} \cdot \rho_{air} \cdot v_{m.case4.L3.zone2}{}^2 = 1.96 \cdot kPa$$

7.4.5 Peak veolcity pressure

Wind on facade L2

$$d_{case4.L2} := L_3 = 27.6\,m$$

Wind on facade L3

$$d_{case4.L3} := L_2 = 33.5\,m$$

According to EC 1991-1-4 Table 7.1, the shape factors for the wind load on external walls are determined. For intermediate values of the ratio (h/d), linear interpolation should be used.

$$ratio_{L2.case4} := \frac{H_{case4}}{d_{L2}} = 1.251$$

$$ratio_{L3.case4} := \frac{H_{case4}}{d_{L3}} = 1.031$$

$$C_{pe.10.D.L2.case4} := 0.8$$

$$C_{pe.10.D.L3.case4} := 0.8$$

Shape factors for the windward side

$$C_{pe.10.E.L2.case4} := -0.5 + (-0.7 + 0.5) \cdot \frac{\left(ratio_{L2.case4} - 1\right)}{5 - 1} = -0.513$$

$$C_{pe.10.E.L3.case4} := -0.5 + (-0.7 + 0.5) \cdot \frac{\left(ratio_{L3.case4} - 1\right)}{5 - 1} = -0.502$$

Shape factors for the leeward side

7.4.6 Wind pressure on the facades

Wind pressure on facade L2

Wind pressure, EC 1991-1-4 Eq. 5.1

$$w_{L2.case4.D.zone1} := q_{p.case4.L2.zone1} \cdot C_{pe.10.D.L2.case4} = 1.783 \cdot kPa$$ Zone 1

$$w_{L2.case4.D.zone2} := q_{p.case4.L2.zone2} \cdot C_{pe.10.D.L2.case4} = 1.753 \cdot kPa$$ Zone 2

$$w_{L2.case4.E.zone1} := q_{p.case4.L2.zone1} \cdot C_{pe.10.E.L2.case4} = -1.142 \cdot kPa$$

$w_{L2.case4.E.zone2} := q_{p.case4.L2.zone2} \cdot C_{pe.10.E.L2.case4} = -1.123 \cdot kPa$

Wind pressure on facade L3

$w_{L3.case4.D.zone1} := q_{p.case4.L3.zone1} \cdot C_{pe.10.D.L3.case4} = 1.783 \cdot kPa$ Zone 1

$w_{L3.case4.D.zone2} := q_{p.case4.L3.zone2} \cdot C_{pe.10.D.L3.case4} = 1.568 \cdot kPa$ Zone 2

$w_{L3.case4.E.zone1} := q_{p.case4.L3.zone1} \cdot C_{pe.10.E.L3.case4} = -1.118 \cdot kPa$

$w_{L3.case4.E.zone2} := q_{p.case4.L3.zone2} \cdot C_{pe.10.E.L3.case4} = -0.983 \cdot kPa$

7.4.7 Total wind pressure on the facades

$w_{L2.case4.zone1} := w_{L2.case4.D.zone1} - w_{L2.case4.E.zone1} = 2.925 \cdot kPa$ Facade L2 - zone 1

$w_{L2.case4.zone2} := w_{L2.case4.D.zone2} - w_{L2.case4.E.zone2} = 2.876 \cdot kPa$ Facade L2 - zone 2

$w_{L3.case4.zone1} := w_{L3.case4.D.zone1} - w_{L3.case4.E.zone1} = 2.901 \cdot kPa$ Facade L3 - zone 1

$w_{L3.case4.zone2} := w_{L3.case4.D.zone2} - w_{L3.case4.E.zone2} = 2.551 \cdot kPa$ Facade L3 - zone 2

7.5 Case 5 - Five floors are added

$H_{case5} := h_{ref} + 5h_{tot.floor.tim} + h_{roof} = 38.27\,m$ Total height if three more floors are added

7.5.1 Height of the wind zones

$$h_{case5.L2} := \begin{cases} \text{"1 zone"} & \text{if } H_{case5} \le L_2 \\ \text{"2 zones"} & \text{if } L_2 < H_{case5} \le 2 \cdot L_2 \\ \text{"Several zones"} & \text{otherwise} \end{cases} = \text{"2 zones"}$$

When the wind is acting on facade L2 two zones are applied

$$h_{case5.L3} := \begin{cases} \text{"1 zone"} & \text{if } H_{case5} \le L_3 \\ \text{"2 zones"} & \text{if } L_3 < H_{case5} \le 2 \cdot L_3 \\ \text{"Several zones"} & \text{otherwise} \end{cases} = \text{"2 zones"}$$

When the wind is acting on facade L3 two zones are applied.

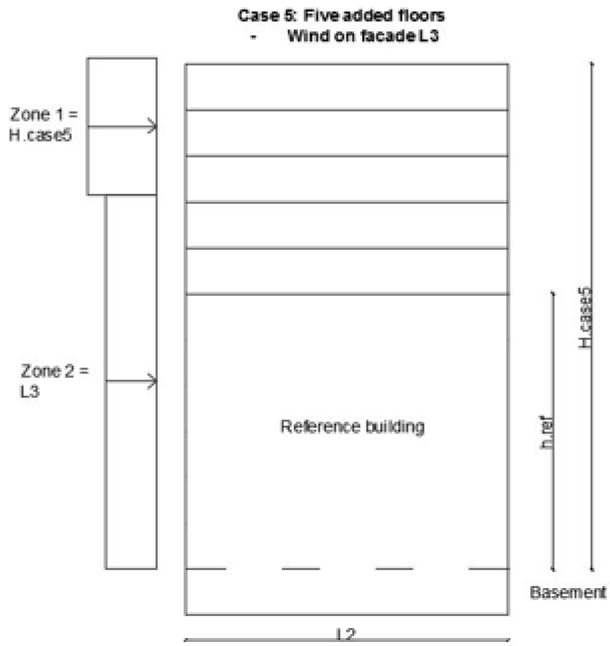
$z_{case5.L2.zone1} := H_{case5} = 38.27\,m$ Height of wind zone 1 for facade L2

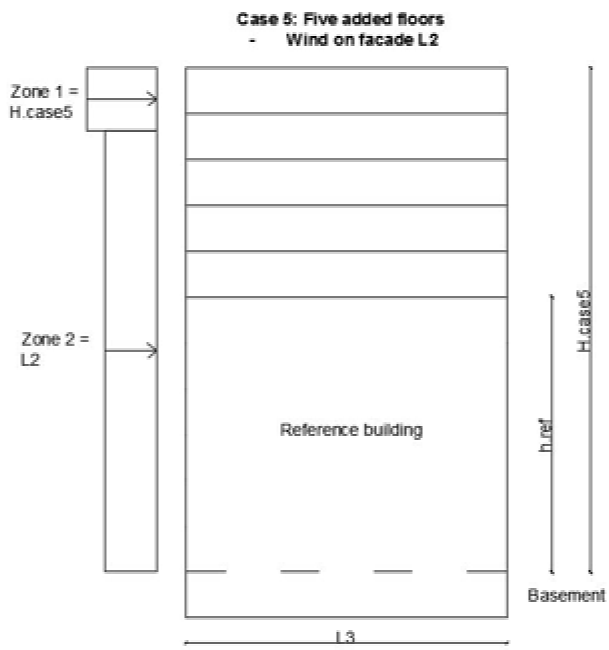
$z_{case5.L2.zone2} := L_2 = 33.5\,m$ Height of wind zone 2 for facade L2

$z_{case5.L3.zone1} := H_{case5} = 38.27\,m$ Height of wind zone 1 for facade L3

$z_{case5.L3.zone2} := L_3 = 27.6\,\mathrm{m}$ Height of wind zone 2 for facade L3

For case 5, two wind zones are applied on both facade L2 and L3. This is shown in the figures below.





7.5.2 Mean wind velocity

Wind on facade L2

$$c_{r.case5.L2.zone1} := k_r \cdot \ln\left(\frac{z_{case5.L2.zone1}}{z_0}\right) = 0.854$$

Roughness factor for Case 5.
EC 1991-1-4, Eq 4.4

$$c_{r.case5.L2.zone2} := k_r \cdot \ln\left(\frac{z_{case5.L2.zone2}}{z_0}\right) = 0.823$$

$$v_{m.case5.L2.zone1} := c_{r.case5.L2.zone1} \cdot c_0 \cdot v_b = 21.351\,\frac{m}{s}$$

lean wind velocity for Case 5.
C 1991-1-4, Eq 4.3

$$v_{m.case5.L2.zone2} := c_{r.case5.L2.zone2} \cdot c_0 \cdot v_b = 20.571\,\frac{m}{s}$$

Wind on facade L3

$$c_{r.case5.L3.zone1} := k_r \cdot \ln\left(\frac{z_{case5.L3.zone1}}{z_0}\right) = 0.854$$

Roughness factor for Case 5.
EC 1991-1-4, Eq 4.4

$$c_{r.case5.L3.zone2} := k_r \cdot \ln\left(\frac{z_{case5.L3.zone2}}{z_0}\right) = 0.777$$

$$v_{m.case5.L3.zone1} := c_{r.case5.L3.zone1} \cdot c_0 \cdot v_b = 21.351\,\frac{m}{s}$$

lean wind velocity for Case 5.
C 1991-1-4, Eq 4.3

$$v_{m.case5.L3.zone2} := c_{r.case5.L3.zone2} \cdot c_0 \cdot v_b = 19.436\,\frac{m}{s}$$

7.5.3 Wind turbulence

Wind on facade L2

$$l_{v.case5.L2.zone1} := \frac{\sigma_v}{v_{m.case5.L2.zone1}} = 0.274$$

Wind turbulence for Case 5.
EC 1991-1-4, Eq 4.7

$$l_{v.case5.L2.zone2} := \frac{\sigma_v}{v_{m.case5.L2.zone2}} = 0.285$$

Wind on facade L3

$$l_{v.case5.L3.zone1} := \frac{\sigma_v}{v_{m.case5.L3.zone1}} = 0.274$$

Wind turbulence for Case 5.
EC 1991-1-4, Eq 4.7

$$l_{v.case5.L3.zone2} := \frac{\sigma_v}{v_{m.case5.L3.zone2}} = 0.301$$

7.5.4 Characteristic veolcity pressure

Characteristic velocity pressure for Case 5, EC 1991-1-4. Eq 4.8

Wind on facade L2

$$q_{p.case5.L2.zone1} := \left(1 + 7 + l_{v.case5.L2.zone1}\right)\cdot\frac{1}{2}\cdot\rho_{air}\cdot v_{m.case5.L2.zone1}{}^{2} = 2.358\cdot kPa$$

$$q_{p.case5.L2.zone2} := \left(1 + 7 + l_{v.case5.L2.zone2}\right)\cdot\frac{1}{2}\cdot\rho_{air}\cdot v_{m.case5.L2.zone2}{}^{2} = 2.191\cdot kPa$$

Wind on facade L3

$$q_{p.case5.L3.zone1} := \left(1 + 7 + l_{v.case5.L3.zone1}\right)\cdot\frac{1}{2}\cdot\rho_{air}\cdot v_{m.case5.L3.zone1}{}^{2} = 2.358\cdot kPa$$

$$q_{p.case5.L3.zone2} := \left(1 + 7 + l_{v.case5.L3.zone2}\right)\cdot\frac{1}{2}\cdot\rho_{air}\cdot v_{m.case5.L3.zone2}{}^{2} = 1.96\cdot kPa$$

7.5.5 Peak veolcity pressure

Wind on facade L2

$$d_{case5.L2} := L_3 = 27.6\,m$$

Wind on facade L3

$$d_{case5.L3} := L_2 = 33.5\,m$$

According to EC 1991-1-4 Table 7.1, the shape factors for the wind load on external walls are determined. For intermediate values of the ratio (h/d), linear interpolation should be used

$$ratio_{L2.case5} := \frac{H_{case5}}{d_{L2}} = 1.387$$

$$ratio_{L3.case5} := \frac{H_{case5}}{d_{L3}} = 1.142$$

$$C_{pe.10.D.L2.case5} := 0.8$$

$$C_{pe.10.D.L3.case5} := 0.8$$

$$C_{pe.10.E.L2.case5} := -0.5 + (-0.7 + 0.5)\cdot\frac{\left(ratio_{L2.case5} - 1\right)}{5 - 1} = -0.519$$

$$C_{pe.10.E.L3.case5} := -0.5 + (-0.7 + 0.5)\cdot\frac{\left(ratio_{L3.case5} - 1\right)}{5 - 1} = -0.507$$

7.5.6 Wind pressure on the facades

Wind pressure on facade L2

Wind pressure, EC 1991-1-4 Eq. 5.1

$$w_{L2.case5.D.zone1} := q_{p.case5.L2.zone1}\cdot C_{pe.10.D.L2.case5} = 1.886\cdot kPa$$ Zone 1

$$w_{L2.case5.D.zone2} := q_{p.case5.L2.zone2}\cdot C_{pe.10.D.L2.case5} = 1.753\cdot kPa$$ Zone 2

$w_{L2.case5.E.zone1} := q_{p.case5.L2.zone1} \cdot C_{pe.10.E.L2.case5} = -1.224 \cdot kPa$

$w_{L2.case5.E.zone2} := q_{p.case5.L2.zone2} \cdot C_{pe.10.E.L2.case5} = -1.138 \cdot kPa$

Wind pressure on facade L3

$w_{L3.case5.D.zone1} := q_{p.case5.L3.zone1} \cdot C_{pe.10.D.L3.case5} = 1.886 \cdot kPa$ Zone 1

$w_{L3.case5.D.zone2} := q_{p.case5.L3.zone2} \cdot C_{pe.10.D.L3.case5} = 1.568 \cdot kPa$ Zone 2

$w_{L3.case5.E.zone1} := q_{p.case5.L3.zone1} \cdot C_{pe.10.E.L3.case5} = -1.196 \cdot kPa$

$w_{L3.case5.E.zone2} := q_{p.case5.L3.zone2} \cdot C_{pe.10.E.L3.case5} = -0.994 \cdot kPa$

7.5.7 Total wind pressure on the facades

$w_{L2.case5.zone1} := w_{L2.case5.D.zone1} - w_{L2.case5.E.zone1} = 3.11 \cdot kPa$ Facade L2 - zone 1

$w_{L2.case5.zone2} := w_{L2.case5.D.zone2} - w_{L2.case5.E.zone2} = 2.891 \cdot kPa$ Facade L2 - zone 2

$w_{L3.case5.zone1} := w_{L3.case5.D.zone1} - w_{L3.case5.E.zone1} = 3.082 \cdot kPa$ Facade L3 - zone 1

$w_{L3.case5.zone2} := w_{L3.case5.D.zone2} - w_{L3.case5.E.zone2} = 2.562 \cdot kPa$ Facade L3 - zone 2

Appendix 8 - Horizontal stability for the added floors

8.1 Unintended inclination

8.1.1 Indata

$n := 38$ — Number of columns and shear walls on one floor

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

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

$\alpha_{md} := \alpha_0 + \frac{\alpha_d}{\sqrt{n}} = 4.947 \times 10^{-3}$ — Unintended inclination angle

$g_{ex.wall} := 0.5 \frac{kN}{m^2}$ — Self-weigth external walls

$G_{k.floor.ref} := G_{k.c.floor} + G_{k.t.floor} + G_{k.s.floor} = 5.22 \cdot \frac{kN}{m^2}$

Self - weigth of the existing floor structure

8.1.2 Geometry

$C_{ex.wall} := L_1 + L_2 + L_3 + L_4 + L_5 + L_6 = 121.9\,m$ — Circumference for the building

$A_{floor} := L_1 \cdot L_2 + L_4 \cdot L_5 = 738.98\,m^2$ — Approximate total area of the floor

$l_{span.beam} := L_2$ — Lenght of the continous beam

$A_{tot.col} := A_{glulam.col_4} \cdot 30 = 2.322\,m^2$ — Total area of all the columns on one floor which is totally 30 columns

$n_{beam} := 4$ — Number of beams on one floor

$h_{ex.wall} := h_{tot.floor.tim} = 3.734\,m$ — Height for the external wall for one floor

8.2 Unintended inclination - Load combinations in ULS

The used load combinations in ULS are according to the National Standards in Sweden for equations 6.10a and 6.10b. The calculations are performed for two cases, where thw self-weight are unfavourable and favorable.

$\gamma_d := 0.91$ — Partial coefficient for safety class 2

Self-weight unfavourable on the top floor

$$V_{d.top.unf} := \gamma_d \cdot 1.1 \cdot \left(g_{k.roof.timber} \cdot A_{floor} + g_{k.beam} \cdot l_{span.beam} \cdot n_{beam}\right) \ldots + 1.5 \cdot \gamma_d \cdot \psi_{0.s} \cdot S \cdot A_{floor} = 1.024 \times 10^3 \cdot kN$$

Self-weight favourable on the top floor

$$V_{d.top.fav} := 0.9 \cdot \left[\left(g_{k.roof.timber}\right) \cdot A_{floor} + g_{k.beam} \cdot l_{span.beam} \cdot n_{beam}\right] \ldots + 0 \cdot \gamma_d \cdot \psi_{0.s} \cdot S \cdot A_{floor} = 267.783 \cdot kN$$

$$V_{d.top} := \max\left(V_{d.top.unf}, V_{d.top.fav}\right) = 1.024 \times 10^3 \cdot kN$$

Self-weight unfavourable on the fifth floor

$$V_{d.5.unf} := \gamma_d \cdot 1.1 \cdot \left[g_{k.floor} \cdot A_{floor} + g_{k.beam} \cdot l_{span.beam} \cdot n_{beam} \cdots + \left(\rho_{glulam} \cdot A_{tot.col} \cdot h_{column}\right) + g_{ex.wall} \cdot h_{ex.wall} \cdot C_{ex.wall}\right] \ldots + 1.5 \cdot \gamma_d \cdot \psi_{0.i} \cdot q_{k.imp} \cdot A_{floor} = 2.571 \times 10^3 \cdot kN$$

Self-weight favourable on the fifth floor

$$V_{d.5.fav} := 0.9 \cdot \left[g_{k.floor} \cdot A_{floor} + g_{k.beam} \cdot l_{span.beam} \cdot n_{beam} \cdots + \left(\rho_{glulam} \cdot A_{tot.col} \cdot h_{column}\right) + g_{ex.wall} \cdot h_{ex.wall} \cdot C_{ex.wall}\right] \ldots + 0 \cdot \gamma_d \cdot \psi_{0.i} \cdot q_{k.imp} \cdot A_{floor} = 1.042 \times 10^3 \cdot kN$$

Self-weight unfavourable on the first floor

$$V_{d.1.unf} := \gamma_d \cdot 1.1 \cdot \left[G_{k.floor.ref} \cdot A_{floor} + \left(\rho_{glulam} \cdot A_{tot.col} \cdot h_{column}\right) \cdots + g_{ex.wall} \cdot h_{ex.wall} \cdot C_{ex.wall}\right] \ldots + 1.5 \cdot \gamma_d \cdot \psi_{0.i} \cdot q_{k.imp} \cdot A_{floor} = 5.529 \times 10^3 \cdot kN$$

Self-weight favourable on the first floor

$$V_{d.1.fav} := 0.9 \cdot \left[G_{k.floor.ref} \cdot A_{floor} + \left(\rho_{glulam} \cdot A_{tot.col} \cdot h_{column}\right) \cdots + g_{ex.wall} \cdot h_{ex.wall} \cdot C_{ex.wall}\right] \ldots + 0 \cdot \gamma_d \cdot \psi_{0.i} \cdot q_{k.imp} \cdot A_{floor} = 3.702 \times 10^3 \cdot kN$$

$V_{d.4.unf} := V_{d.5.unf}$

$V_{d.4.fav} := V_{d.5.fav}$

$V_{d.3.unf} := V_{d.5.unf}$

$V_{d.3.fav} := V_{d.5.fav}$

$V_{d.2.unf} := V_{d.5.unf}$

$V_{d.2.fav} := V_{d.5.fav}$

The load is the same for the rest of the floors as for the fith floor

$$V_{d.unf} := \begin{pmatrix} V_{d.top.unf} \\ V_{d.5.unf} \\ V_{d.4.unf} \\ V_{d.3.unf} \\ V_{d.2.unf} \\ V_{d.1.unf} \end{pmatrix} = \begin{pmatrix} 1.024 \times 10^3 \\ 2.571 \times 10^3 \\ 2.571 \times 10^3 \\ 2.571 \times 10^3 \\ 2.571 \times 10^3 \\ 5.529 \times 10^3 \end{pmatrix} \cdot kN$$

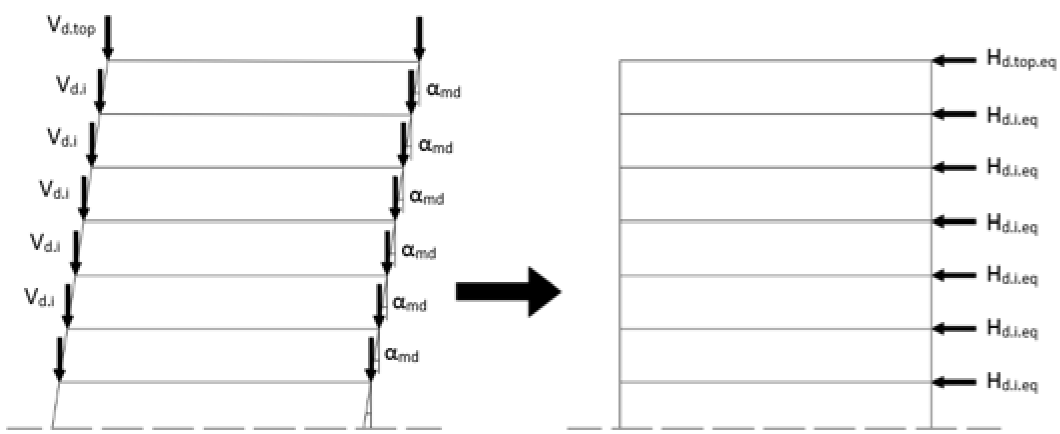
Resulting vertical load for each floor when self-weight is unfavourable

$$V_{d.fav} := \begin{pmatrix} V_{d.top.fav} \\ V_{d.5.fav} \\ V_{d.4.fav} \\ V_{d.3.fav} \\ V_{d.2.fav} \\ V_{d.1.fav} \end{pmatrix} = \begin{pmatrix} 267.783 \\ 1.042 \times 10^3 \\ 1.042 \times 10^3 \\ 1.042 \times 10^3 \\ 1.042 \times 10^3 \\ 3.702 \times 10^3 \end{pmatrix} \cdot kN$$

Resulting vertical load for each floor when self-weight is favourable

8.2.1 Horizontal loads due to unintended inclination - self-weight unfavourable

The Figure below illustrates the principle for determining the horizontal loads from the calculated vertical loads.



$j := 0..5$

$H_{d.unf.ui_j} := V_{d.unf_j} \cdot \alpha_{md} = ...$

Calculation of the horizontal loads due to unintended inclination with self-weight unfavourable

$$H_{d.unf.ui} = \begin{pmatrix} 5.066 \\ 12.717 \\ 12.717 \\ 12.717 \\ 12.717 \\ 27.352 \end{pmatrix} \cdot kN$$

8.2.2 Horizontal loads due to unintended inclination - self-weight favourable

$$H_{d.fav.ui_j} := V_{d.fav_j} \cdot \alpha_{md} = \cdots$$

Calculation of the horizontal loads due to unintended inclination with self-weight favourable

$$H_{d.fav.ui} = \begin{pmatrix} 1.325 \\ 5.153 \\ 5.153 \\ 5.153 \\ 5.153 \\ 18.311 \end{pmatrix} \cdot kN$$

8.3 Check - Tilting when five floors are added

8.3.1 Moment due to unintended inclination

Self-weight unfavourable

$$M_{Ed.unf.ui.5} := H_{d.unf.ui_0} \cdot 5 \cdot h_{tot.floor.tim} + H_{d.unf.ui_1} \cdot 4 \cdot h_{tot.floor.tim} \cdots + H_{d.unf.ui_2} \cdot 3 \cdot h_{tot.floor.tim} + H_{d.unf.ui_3} \cdot 2 \cdot h_{tot.floor.tim} \cdots + H_{d.unf.ui_4} \cdot h_{tot.floor.tim} = 569.424 \cdot kN \cdot m$$

Self-weight favourable

$$M_{Ed.fav.ui.5} := H_{d.fav.ui_0} \cdot 5 \cdot h_{tot.floor.tim} + H_{d.fav.ui_1} \cdot 4 \cdot h_{tot.floor.tim} \cdots + H_{d.fav.ui_2} \cdot 3 \cdot h_{tot.floor.tim} + H_{d.fav.ui_3} \cdot 2 \cdot h_{tot.floor.tim} \cdots + H_{d.fav.ui_4} \cdot h_{tot.floor.tim} = 217.139 \cdot kN \cdot m$$

8.3.2 Moment due to wind load

The moment due to the wind loads are calculated in the transition between the existing building and the added floors, which is shown in the figure below.

Wind on facade L2

$$h_{wind.L2.zone1.5} := H_{case5} - L_2 = 4.77\,m$$

$h_{wind.L2.zone2.5} := L_2 - h_{ref} = 14.3\,m$

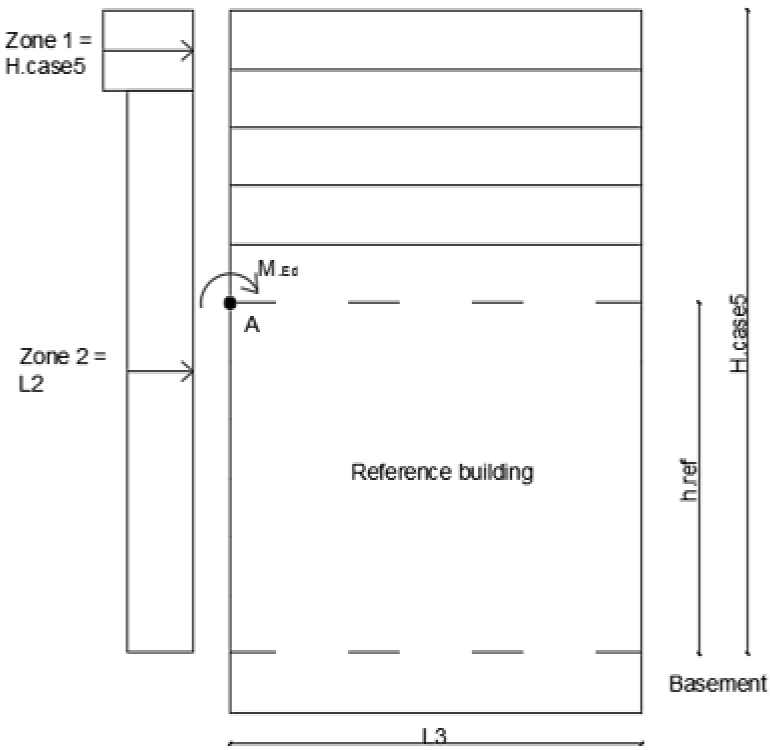
$l_{L2.1.5} := \left(\frac{h_{wind.L2.zone1.5}}{2} + h_{wind.L2.zone2.5}\right)$ Lever arm for moment due to zone 1

$M_{Ed.wind.L2.zone1.5} := \left(1.5w_{L2.case5.zone1}\right)\cdot L_2\cdot\left(H_{case5} - L_2\right)\cdot l_{L2.1.5} = 1.244 \times 10^4\cdot kN\cdot m$

$l_{L2.2.5} := \left[5.h_{tot.floor.tim} - \left(H_{case5} - L_2\right)\right]\cdot\frac{h_{wind.L2.zone2.5}}{2}$

Lever arm for moment due to zone 2

$M_{Ed.wind.L2.zone2.5} := \left(1.5w_{L2.case5.zone2}\right)\cdot L_2\cdot l_{L2.2.5} = 1.444 \times 10^4\cdot kN\cdot m$



The calculations of the moment due to the wind loads on facade L3 follows the same principle as for facade L2.

Wind on facade L3

$h_{wind.L3.zone1.5} := H_{case5} - L_3 = 10.67\,m$

$h_{wind.L3.zone2.5} := L_3 - h_{ref} = 8.4\,m$

$l_{L3.1.5} := \left(\frac{h_{wind.L3.zone1.5}}{2} + h_{wind.L3.zone2.5}\right)$ Lever arm for moment due to zone 1

$M_{Ed.wind.L3.zone1.5} := \left(1.5 w_{L3.case5.zone1}\right) \cdot L_3 \cdot \left(H_{case5} - L_3\right) \cdot l_{L3.1.5} = 1.87 \times 10^4 \cdot kN \cdot m$

$l_{L3.2.5} := \left[5.h_{tot.floor.tim} - \left(H_{case5} - L_3\right)\right] \cdot \dfrac{h_{wind.L2.zone2.5}}{2}$

Lever arm for moment due to zone 2

$M_{Ed.wind.L3.zone2.5} := \left(1.5 w_{L3.case5.zone2}\right) \cdot L_3 \cdot l_{L3.2.5} = 6.067 \times 10^3 \cdot kN \cdot m$

8.3.3 Total moment

Wind on facade L2, Self-weight unfavourable

$M_{Ed.unf.L2.5} := M_{Ed.unf.ui.5} + M_{Ed.wind.L2.zone1.5} + M_{Ed.wind.L2.zone2.5} = 2.745 \times 10^4 \cdot kN \cdot m$

Wind on facade L2, Self-weight favourable

$M_{Ed.fav.L2.5} := M_{Ed.fav.ui.5} + M_{Ed.wind.L2.zone1.5} + M_{Ed.wind.L2.zone2.5} = 2.709 \times 10^4 \cdot kN \cdot m$

Wind on facade L3, Self-weight unfavourable

$M_{Ed.unf.L3.5} := M_{Ed.unf.ui.5} + M_{Ed.wind.L3.zone1.5} + M_{Ed.wind.L3.zone2.5} = 2.533 \times 10^4 \cdot kN \cdot m$

Wind on facade L3, Self-weight favourable

$M_{Ed.fav.L3.5} := M_{Ed.fav.ui.5} + M_{Ed.wind.L3.zone1.5} + M_{Ed.wind.L3.zone2.5} = 2.498 \times 10^4 \cdot kN \cdot m$

8.3.4 Resisting moment

$e_{RC.L2} := \dfrac{L_3}{6} = 4.6\,m$

Distance to the rotation centre. Maximum eccentricity when wind act on facade L2

$e_{RC.L3} := \dfrac{L_2}{6} = 5.583\,m$

Distance to the rotation centre. Maximum eccentricity when wind act on facade L3

Wind on facade L2, Self-weight unfavourable

$M_{Rd.unf.L2.5} := e_{RC.L2} \cdot \left(V_{d.top.unf} + 4 \cdot V_{d.5.unf} + V_{d.1.unf}\right) = 7.745 \times 10^4 \cdot kN \cdot m$

Wind on facade L2, Self-weight favourable

$M_{Rd.fav.L2.5} := e_{RC.L2} \cdot \left(V_{d.top.fav} + 4 \cdot V_{d.5.fav} + V_{d.1.fav}\right) = 3.743 \times 10^4 \cdot kN \cdot m$

Wind on facade L3, Self-weight unfavourable

$$M_{Rd.unf.L3.5} := e_{RC.L3} \cdot \left(V_{d.top.unf} + 4 \cdot V_{d.5.unf} + V_{d.1.unf}\right) = 9.4 \times 10^{4} \cdot kN \cdot m$$

Wind on facade L3, Self-weight favourable

$$M_{Rd.fav.L3.5} := e_{RC.L3} \cdot \left(V_{d.top.fav} + 4 \cdot V_{d.5.fav} + V_{d.1.fav}\right) = 4.543 \times 10^{4} \cdot kN \cdot m$$

Utilization ratio

$$\boxed{M_{Rd} \geq M_{Ed}}$$

Facade L2

$$\frac{M_{Ed.unf.L2.5}}{M_{Rd.unf.L2.5}} = 35.439 \cdot \%$$

$$\frac{M_{Ed.fav.L2.5}}{M_{Rd.fav.L2.5}} = 72.392 \cdot \%$$

Facade L3

$$\frac{M_{Ed.unf.L3.5}}{M_{Rd.unf.L3.5}} = 26.949 \cdot \%$$

$$\frac{M_{Ed.fav.L3.5}}{M_{Rd.fav.L3.5}} = 54.991 \cdot \%$$

Appendix 9 - Horizontal stability for the whole building with five added floors

9.1 Unintended inclination for the existing building

$q_{k.imp.ref} := 3 \frac{kN}{m^2}$ Imposed load for office ares in the existing building

9.1.1 Self-weight

$g_{ew} := 10.7 \frac{kN}{m}$ Self-weigth external walls

$G_{k.column} = 1.105 \cdot \frac{kN}{m}$ Self-weight of the steel columns

$G_{k.floor.ref} := G_{k.c.floor} + G_{k.t.floor} + G_{k.s.floor} = 5.22 \cdot \frac{kN}{m^2}$ Self - weigth of the existing floor structure

9.1.2 Geometry

$C_{ex.wall} := L_1 + L_2 + L_3 + L_4 + L_5 + L_6 = 121.9\,m$ Circumference for the building

$A_{floor} := L_1 \cdot L_2 + L_4 \cdot L_5 = 738.98\,m^2$ Total area of the floor

$h_{ex.wall.ref} := h_{column} + h_{floor} = 3.2\,m$ Height for the external wall for one floor in the reference building

9.1.3 Load combinations in ULS

The used load combinations in ULS are according to the Nation Standards in Sweden for equations 6.10a and 6.10b.

$\gamma_d := 0.91$ Partial coefficient for safety class 2

Self-weight unfavourable on the floors in the existing building

$$V_{d.ref.unf} := \gamma_d \cdot 1.1 \cdot \left(G_{k.floor.ref} \cdot A_{floor} + G_{k.column} \cdot h_{column} + g_{ew} \cdot C_{ex.wall}\right) \ldots = 7.288 \times 10^3 \cdot kN$$
$$+ 1.5 \cdot \gamma_d \cdot \psi_{0.i} \cdot q_{k.imp.ref} \cdot A_{floor}$$

Self-weight favourable on the floors in the existing building

$$V_{d.ref.fav} := 0.9 \cdot \left(G_{k.floor.ref} \cdot A_{floor} + G_{k.column} \cdot h_{column} + g_{ew} \cdot C_{ex.wall}\right) \ldots = 4.648 \times 10^3 \cdot kN$$
$$+ 0 \cdot \gamma_d \cdot \psi_{0.i} \cdot q_{k.imp.ref} \cdot A_{floor}$$

The load is the same for the rest of the floors in the exsisting building.

Horizontal loads with self-weight unfavourable

$H_{d.unf.ref.ui} := V_{d.ref.unf} \cdot \alpha_{md} = 36.053 \cdot kN$

Calculation of the horizontal loads due to unintended inclination with self-weight unfavourable

Horizontal loads with self-weight favourable

$H_{d.fav.ref.ui} := V_{d.ref.fav} \cdot \alpha_{md} = 22.994 \cdot kN$

Calculation of the horizontal loads due to unintended inclination with self-weight favourable

9.2 Control of tilting for the whole building with five added floors

9.2.1 Moment due to unintended inclinations

Self-weight unfavourable

$$M_{Ed.unf.ui.tot} := H_{d.unf.ui_0} \cdot H_{case5} + H_{d.unf.ui_1} \cdot H_{case4} + H_{d.unf.ui_2} \cdot H_{case3} \cdots + H_{d.unf.ui_3} \cdot H_{case2} + H_{d.unf.ui_4} \cdot H_{case1} + H_{d.unf.ui_5} \cdot h_{ref} \cdots + H_{d.unf.ref.ui} \cdot \left(h_{storey} + h_{floor}\right)(5 + 4 + 3 + 2 + 1) = 3.921 \times 10^{3} \cdot kN \cdot m$$

Self-weight favourable

$$M_{Ed.fav.ui.tot} := H_{d.fav.ui_0} \cdot H_{case5} + H_{d.fav.ui_1} \cdot H_{case4} + H_{d.fav.ui_2} \cdot H_{case3} \cdots + H_{d.fav.ui_3} \cdot H_{case2} + H_{d.fav.ui_4} \cdot H_{case1} + H_{d.unf.ui_5} \cdot h_{ref} \cdots + H_{d.unf.ref.ui} \cdot \left(h_{storey} + h_{floor}\right)(5 + 4 + 3 + 2 + 1) = 2.903 \times 10^{3} \cdot kN \cdot m$$

9.2.2 Moment due to wind load

Wind on facade L2

$$l_{L2.1.tot} := \frac{h_{wind.L2.zone1.5}}{2} + L_2$$

$$M_{Ed.wind.L2.zone1.tot} := 1.5\left[w_{L2.case5.zone1} \cdot L_2 \cdot \left(H_{case5} - L_2\right) \cdot \left(l_{L2.1.tot}\right)\right] = 2.675 \times 10^{4} \cdot kN \cdot m$$

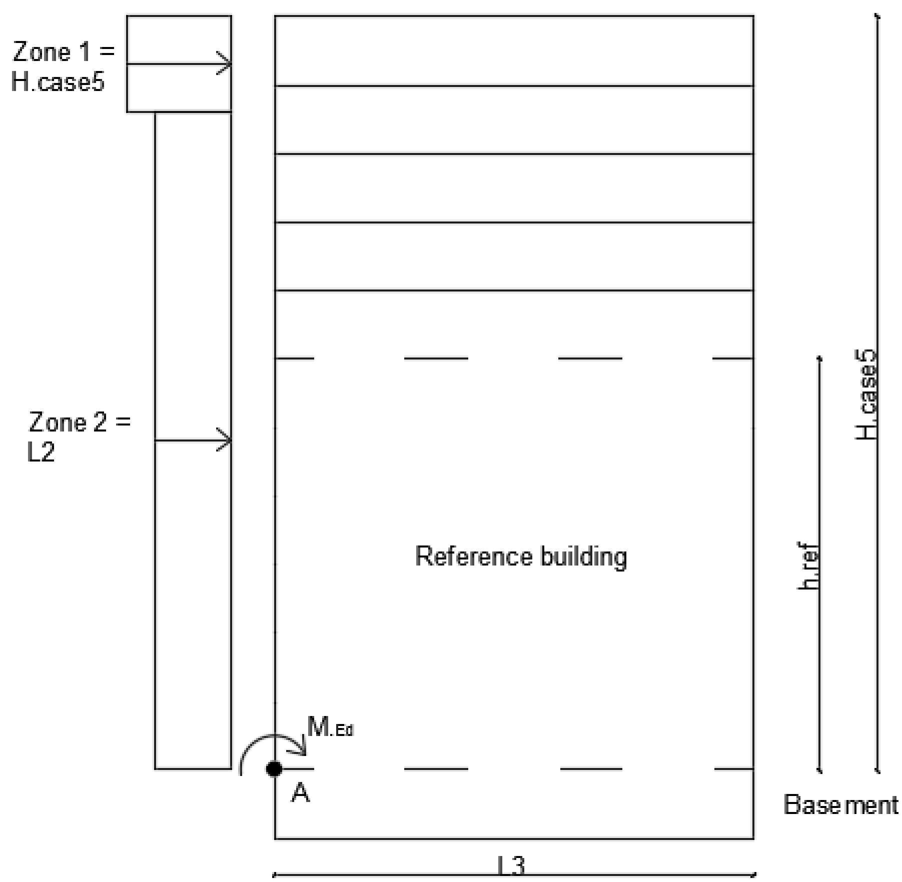
$$M_{Ed.wind.L2.zone2.tot} := 1.5\left(w_{L2.case5.zone2} \cdot L_2 \cdot L_2 \cdot \frac{L_2}{2}\right) = 8.152 \times 10^{4} \cdot kN \cdot m$$

Wind on facade L3

$$l_{L3.1.tot} := \frac{h_{wind.L3.zone1.5}}{2} + L_3$$

$$M_{Ed.wind.L3.zone1.tot} := 1.5\left[w_{L3.case5.zone1} \cdot L_3 \cdot \left(H_{case5} - L_3\right) \cdot \left(l_{L3.1.tot}\right)\right] = 4.483 \times 10^4 \cdot kN \cdot m$$

$$M_{Ed.wind.L3.zone2.tot} := 1.5\left(w_{L3.case5.zone2} \cdot L_3 \cdot L_3 \cdot \frac{L_3}{2}\right) = 4.04 \times 10^4 \cdot kN \cdot m$$



The calculations of the moment due to the wind loads on facade L3. The same principle for facade L2.

9.2.3 Total moment

Wind on facade L2, Self-weight unfavourable

$$M_{Ed.unf.L2.tot} := M_{Ed.unf.ui.tot} + M_{Ed.wind.L2.zone1.tot} + M_{Ed.wind.L2.zone2.tot} = 1.122 \times 10^{5} \cdot kN \cdot m$$

Wind on facade L2, Self-weight favourable

$$M_{Ed.fav.L2.tot} := M_{Ed.fav.ui.tot} + M_{Ed.wind.L2.zone1.tot} + M_{Ed.wind.L2.zone2.tot} = 1.112 \times 10^{5} \cdot kN \cdot m$$

Wind on facade L3, Self-weight unfavourable

$$M_{Ed.unf.L3.tot} := M_{Ed.unf.ui.tot} + M_{Ed.wind.L3.zone1.tot} + M_{Ed.wind.L3.zone2.tot} = 8.915 \times 10^{4} \cdot kN \cdot m$$

Wind on facade L3, Self-weight favourable

$$M_{Ed.fav.L3.tot} := M_{Ed.fav.ui.tot} + M_{Ed.wind.L3.zone1.tot} + M_{Ed.wind.L3.zone2.tot} = 8.814 \times 10^{4} \cdot kN \cdot m$$

9.2.4 Resisting moment

$$e_{RC.L2} := \frac{L_3}{6} = 4.6\,m$$

Distance to the rotation centre. Maximum eccentricity when wind act on facade L2

$$e_{RC.L3} := \frac{L_2}{6} = 5.583\,m$$

Distance to the rotation centre. Maximum eccentricity when wind act on facade L3

Wind on facade L2, Self-weight unfavourable

$$M_{Rd.unf.L2.tot} := e_{RC.L2} \cdot \left(V_{d.top.unf} + 4 \cdot V_{d.5.unf} + V_{d.1.unf} + 6 \cdot V_{d.ref.unf}\right) = 2.786 \times 10^{5} \cdot kN \cdot m$$

Wind on facade L2, Self-weight favourable

$$M_{Rd.fav.L2.tot} := e_{RC.L2} \cdot \left(V_{d.top.fav} + 4 \cdot V_{d.5.fav} + V_{d.1.fav} + 6 \cdot V_{d.ref.fav}\right) = 1.657 \times 10^{5} \cdot kN \cdot m$$

Wind on facade L3, Self-weight unfavourable

$$M_{Rd.unf.L3.tot} := e_{RC.L3} \cdot \left(V_{d.top.unf} + 4 \cdot V_{d.5.unf} + V_{d.1.unf} + 6 \cdot V_{d.ref.unf}\right) = 3.382 \times 10^{5} \cdot kN \cdot m$$

Wind on facade L3, Self-weight favourable

$$M_{Rd.fav.L3.tot} := e_{RC.L3} \cdot \left(V_{d.top.fav} + 4 \cdot V_{d.5.fav} + V_{d.1.fav} + 6 \cdot V_{d.ref.fav}\right) = 2.011 \times 10^{5} \cdot kN \cdot m$$

9.2.5 Utilization ratio

Criterion for tilting of the whole building. The resisting moment should be larger that the acting moment due to wind and unintended inclination

$$\boxed{M_{Rd.tot} \geq M_{Ed.tot}}$$

Facade L2

$$\frac{M_{Ed.unf.L2.tot}}{M_{Rd.unf.L2.tot}} = 40.268 \cdot \%$$

$$\frac{M_{Ed.fav.L2.tot}}{M_{Rd.fav.L2.tot}} = 67.083 \cdot \%$$

Facade L3

$$\frac{M_{Ed.unf.L3.tot}}{M_{Rd.unf.L3.tot}} = 26.364 \cdot \%$$

$$\frac{M_{Ed.fav.L3.tot}}{M_{Rd.fav.L3.tot}} = 43.816 \cdot \%$$

9.3 Distributed wind load when five floors are added

9.3.1 Influencing height of the wind load for each storey

$$h_{w.1} := \frac{h_{storey} + h_{floor}}{2} = 1.6\,m$$

$$h_{w.2} := h_{storey} + h_{floor} = 3.2\,m$$

$$h_{w.3} := \frac{h_{storey} + h_{floor}}{2} + \frac{h_{floor} + h_{tot.floor.tim}}{2} = 3.667\,m$$

$$h_{w.4} := h_{tot.floor.tim} = 3.734\,m$$

$$h_{w.5} := h_{w.4} = 3.734\,m$$

$$h_{w.6} := \frac{h_{tot.floor.tim} + h_{roof}}{2} = 2.067\,m$$

Wind pressure on facade L2

$$F_{1.L2.5} := h_{w.1} \cdot w_{L2.case5.zone2} = 4.626 \cdot \frac{kN}{m}$$

$$F_{2.L2.5} := h_{w.2} \cdot w_{L2.case5.zone2} = 9.251 \cdot \frac{kN}{m}$$

$$F_{3.L2.5} := h_{w.3} \cdot w_{L2.case5.zone2} = 10.601 \cdot \frac{kN}{m}$$

Wind pressure on facade L3

$$F_{1.L3.5} := h_{w.1} \cdot w_{L3.case5.zone2} = 4.099 \cdot \frac{kN}{m}$$

$$F_{2.L3.5} := h_{w.2} \cdot w_{L3.case5.zone2} = 8.198 \cdot \frac{kN}{m}$$

$$F_{3.L3.5} := h_{w.3} \cdot w_{L3.case5.zone2} = 9.395 \cdot \frac{kN}{m}$$

$F_{4.L2.5} := h_{w.4} \cdot w_{L2.case5.zone2} = 10.795 \cdot \frac{kN}{m}$

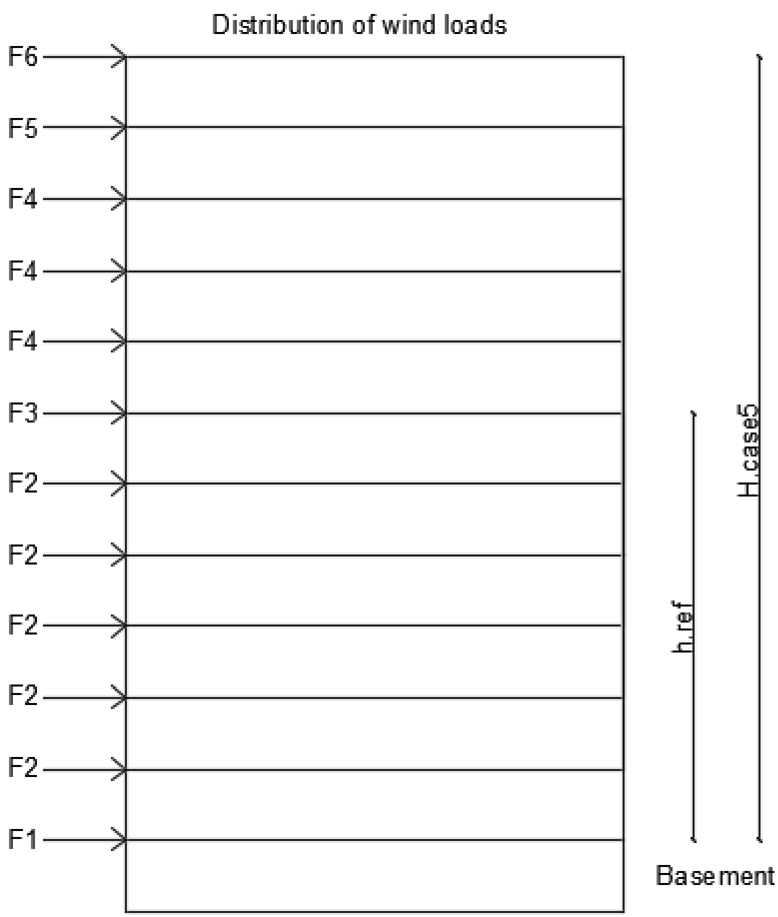
$F_{5.L2.5} := h_{w.5} \cdot w_{L2.case5.zone1} = 11.614 \cdot \frac{kN}{m}$

$F_{6.L2.5} := h_{w.6} \cdot w_{L2.case5.zone1} = 6.429 \cdot \frac{kN}{m}$

$F_{4.L3.5} := h_{w.4} \cdot w_{L3.case5.zone2} = 9.567 \cdot \frac{kN}{m}$

$F_{5.L3.5} := h_{w.5} \cdot w_{L3.case5.zone1} = 11.507 \cdot \frac{kN}{m}$

$F_{6.L3.5} := h_{w.6} \cdot w_{L3.case5.zone1} = 6.37 \cdot \frac{kN}{m}$



9.4 Distributed wind load when three floors are added

Wind pressure on facade L2

$F_{1.L2.3} := h_{w.1} \cdot w_{L2.case3} = 4.365 \cdot \frac{kN}{m}$

Wind pressure on facade L3

$F_{1.L3.3} := h_{w.1} \cdot w_{L3.case3.zone2} = 3.976 \cdot \frac{kN}{m}$

$F_{2.L2.3} := h_{w.2} \cdot w_{L2.case3} = 8.731 \cdot \frac{kN}{m}$

$F_{3.L2.3} := h_{w.3} \cdot w_{L2.case3} = 10.005 \cdot \frac{kN}{m}$

$F_{4.L2.3} := h_{w.4} \cdot w_{L2.case3} = 10.188 \cdot \frac{kN}{m}$

$F_{5.L2.3} := h_{w.5} \cdot w_{L2.case3} = 10.188 \cdot \frac{kN}{m}$

$F_{6.L2.3} := h_{w.6} \cdot w_{L2.case3} = 5.64 \cdot \frac{kN}{m}$

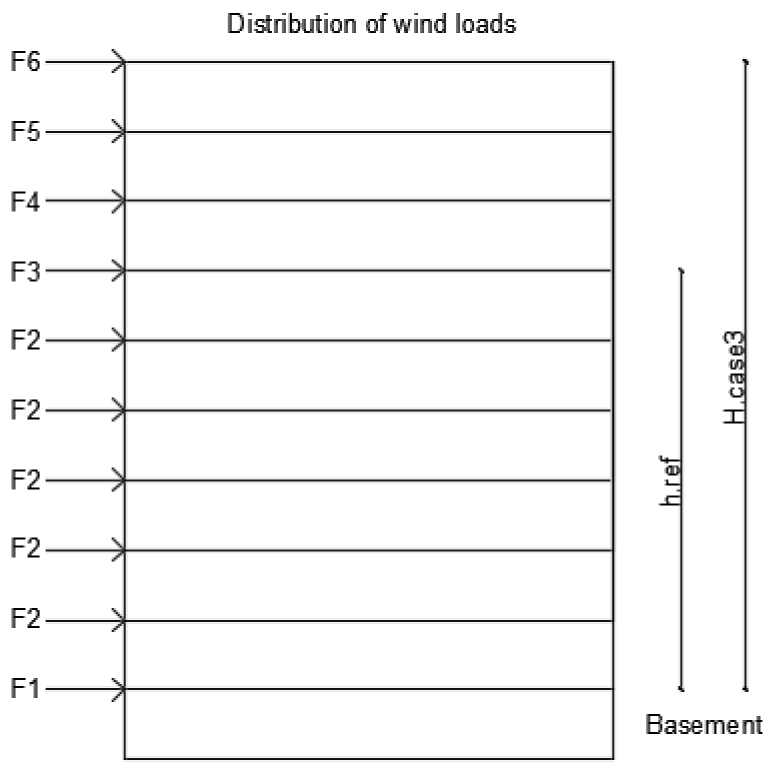
$F_{2.L3.3} := h_{w.2} \cdot w_{L3.case3.zone2} = 7.952 \cdot \frac{kN}{m}$

$F_{3.L3.3} := h_{w.3} \cdot w_{L3.case3.zone2} = 9.112 \cdot \frac{kN}{m}$

$F_{4.L3.3} := h_{w.4} \cdot w_{L3.case3.zone2} = 9.279 \cdot \frac{kN}{m}$

$F_{5.L3.3} := h_{w.5} \cdot w_{L3.case3.zone1} = 9.891 \cdot \frac{kN}{m}$

$F_{6.L3.3} := h_{w.6} \cdot w_{L3.case3.zone1} = 5.475 \cdot \frac{kN}{m}$



Appendix 10 - Control against fire

10.1 Control of timber columns

The design due to fire and the equations below follows the principles of Eurocode 1995-1-2.

10.1.1 Loads

$\eta_{fire} := 0.6$

Recommended reduction value for the relation between permanet load and imposed load

$Q_{column.fire_i} := \eta_{fire} \cdot Q_{glulam.column_i}$

EC 1995-1-2, Eq 2.8

$$Q_{column.fire} = \begin{pmatrix} 132.301 \\ 232.16 \\ 332.019 \\ 431.877 \\ 531.736 \end{pmatrix} \cdot kN$$

Verticals load according to fire for the five different cases

10.1.2 Geometry

$t_{fire} := 90min$

Fire safety for 90 minutes

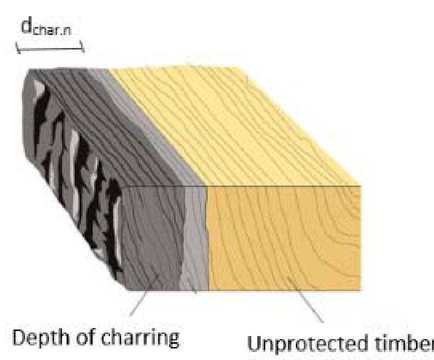
$\beta_{n.glulam} := 0.70 \cdot \frac{mm}{min}$

Design value due to charring for glulam material. EC 1995-1-2, Table 3.1

$d_{char.n} := \beta_{n.glulam} \cdot t_{fire} = 0.063\,m$

Design charring depth after the time of 90 minutes

$$h_{col.fire} := \begin{pmatrix} 225 \\ 225 \\ 270 \\ 315 \\ 360 \end{pmatrix} mm$$



$$w_{col.fire} := \begin{pmatrix} 165 \\ 165 \\ 190 \\ 190 \\ 215 \end{pmatrix} mm$$

$w_{column.fire_i} := w_{col.fire_i} - 2 \cdot d_{char.n}$

Width of column taking charring depth into account

$$w_{column.fire} = \begin{pmatrix} 0.039 \\ 0.039 \\ 0.064 \\ 0.064 \\ 0.089 \end{pmatrix} m$$

The cross-section width of the column in the case of fire for 90 minutes.

$h_{column.fire_i} := h_{col.fire_i} - 2.d_{char.n}$

Height of column taking charring depth into account

$$h_{column.fire} = \begin{pmatrix} 0.099 \\ 0.099 \\ 0.144 \\ 0.189 \\ 0.234 \end{pmatrix} m$$

The cross-section height of the column in the case of fire for 90 minutes.

$p_{fire_i} := 2 \cdot h_{column.fire_i} + 2 \cdot w_{column.fire_i}$

Cirkumference of the column after exposed to fire

$A_{column.fire_i} := w_{column.fire_i} \cdot h_{column.fire_i}$

Cross-section area of the column after exposed to fire

10.1.3 Design strength values

$\gamma_{M.fi} := 1.0$

Partial factor for timber exposed to fire

$k_{fi.glulam} := 1.15$

Modification factor for fire safety design

$$k_{mod.fire_i} := 1 - \frac{1}{125} \cdot \frac{p_{fire_i} \cdot m}{A_{column.fire_i}}$$

Modification factor due to fire for compression stress

$$k_{mod.fire} = \begin{pmatrix} 0.428 \\ 0.428 \\ 0.639 \\ 0.665 \\ 0.752 \end{pmatrix}$$

$$f_{d.fi_i} := k_{mod.fire_i} \cdot k_{fi.glulam} \cdot \frac{f_{c.0.k.glulam}}{\gamma_{M.fi}} = ... \cdot MPa$$

Design value for fire resistance

$$k_{mod.fire.E_i} := 1 - \frac{1}{330} \cdot \frac{p_{fire_i} \cdot m}{A_{column.fire_i}}$$

Modification factor due to fire for Youngs´s modulus

$$E_{fi_i} := k_{mod.fire.E_i} \cdot k_{fi.glulam} \cdot \frac{E_{0.05.glulam}}{\gamma_{M.fi}}$$

Youngs´s modulus due to fire reduced value

10.1.4 Relative slenderness ratio

In strong and weak direction

$$\lambda_{rel.glulam.col.fire.y_i} := \frac{\lambda_{glulam.col.y_i}}{\pi} \cdot \sqrt{\frac{f_{c.0.k.glulam}}{E_{fi_i}}}$$

EC 1995-1-1, Eq 6.27

$$\lambda_{rel.glulam.col.fire.x_i} := \frac{\lambda_{glulam.col.x_i}}{\pi} \cdot \sqrt{\frac{f_{c.0.k.glulam}}{E_{fi_i}}}$$

EC 1995-1-1, Eq 6.22

$$\lambda_{rel.glulam.col.fire.y} = \begin{pmatrix} 0.785 \\ 0.628 \\ 0.498 \\ 0.425 \\ 0.365 \end{pmatrix}$$

The relative slenderness is > 0.3. This indicates that the column needs to be checked against buckling.

$$\lambda_{rel.glulam.col.fire.x} = \begin{pmatrix} 0.856 \\ 0.856 \\ 0.708 \\ 0.704 \\ 0.611 \end{pmatrix}$$

10.1.5 The strenght reduction factor due to instability

In strong and weak direction

$$\beta_{c.glulam} := 0.1$$

The factor is 0.1 for glulam elements. EC 1995-1-1, Eq 6.29

$$k_{glulam.col.fire.y_i} := 0.5 \cdot \left[1 + \beta_{c.glulam} \cdot \left(\lambda_{rel.glulam.col.fire.y_i} - 0.3\right) + \left(\lambda_{rel.glulam.col.fire.y_i}\right)^2\right]$$

EC 1995-1-1, Eq 6.27

$$k_{glulam.col.fire.x_i} := 0.5 \cdot \left[1 + \beta_{c.glulam} \cdot \left(\lambda_{rel.glulam.col.fire.x_i} - 0.3\right) + \left(\lambda_{rel.glulam.col.fire.x_i}\right)^2\right]$$

EC 1995-1-1, Eq 6.28

$$k_{c.glulam.col.fire.y_i} := \frac{1}{k_{glulam.col.fire.y_i} + \sqrt{\left(k_{glulam.col.fire.y_i}\right)^2 - \left(\lambda_{rel.glulam.col.fire.y_i}\right)^2}}$$

The instability factor. EC 1995-1-1, Eq 6.25

$$k_{c.glulam.col.fire.x_i} := \frac{1}{k_{glulam.col.fire.x_i} + \sqrt{\left(k_{glulam.col.fire.x_i}\right)^2 - \left(\lambda_{rel.glulam.col.fire.x_i}\right)^2}}$$

The instability factor. EC 1995-1-1, Eq 6.26

10.1.6 Critcal axial load

According to EC 1995-1-1, Eq 6.23 and 6.24, the dimensioning compression stress in both directions is calculated as:

$$\sigma_{c.0.d.glulam.col.fire.y_i} := k_{c.glulam.col.fire.y_i} \cdot f_{d.fi_i}$$

$$\sigma_{c.0.d.glulam.col.fire.x_i} := k_{c.glulam.col.fire.x_i} \cdot f_{d.fi_i}$$

The axial force in both directions is calculated according to EC 1995-1-1, Eq 6.36

$$N_{cr.glulam.col.fire.y_i} := \sigma_{c.0.d.glulam.col.fire.y_i} \cdot A_{glulam.col_i}$$

$$N_{cr.glulam.col.fire.x_i} := \sigma_{c.0.d.glulam.col.fire.x_i} \cdot A_{glulam.col_i}$$

<u>Total maximum axial force allowed:</u>

$$N_{cr.glulam.col.fire_i} := \min\left(N_{cr.glulam.col.fire.y_i}, N_{cr.glulam.col.fire.x_i}\right)$$

10.1.7 Bending stress

The bending stress in both strong and weak direction is calculated as:

$$\sigma_{m.y.d.column.fire_i} := \frac{6 \cdot M_{Ed.column}}{w_{glulam.col_i} \cdot \left(h_{glulam.col_i}\right)^2}$$

$$\sigma_{m.x.d.column.fire_i} := \frac{6 \cdot M_{Ed.column}}{h_{glulam.col_i} \cdot \left(w_{glulam.col_i}\right)^2}$$

10.1.8 Design strength values due to bending

$$f_{m.d.glulam.col.fire.y_i} := k_{mod.fire_i} \cdot k_{fi.glulam} \frac{f_{m.g.k.glulam}}{\gamma_{M.fi}}$$

$$f_{m.d.glulam.col.fire.x_i} := k_{mod.fire_i} \cdot k_{fi.glulam} \frac{f_{m.g.k.glulam}}{\gamma_{M.fi}}$$

Design value for bending stress paralell to the grain. EC 1995-1-1, Eq 2.17

10.1.9 Check criterion due to combined actions

The column is checked due to combined actions af bending moment and compression

$$Q_{column.fire} = \begin{pmatrix} 132.301 \\ 232.16 \\ 332.019 \\ 431.877 \\ 531.736 \end{pmatrix} \cdot kN \qquad N_{cr.glulam.col.fire} = \begin{pmatrix} 310.849 \\ 388.562 \\ 857.916 \\ 1.044 \times 10^3 \\ 1.564 \times 10^3 \end{pmatrix} \cdot kN$$

$$\sigma_{m.y.d.column.fire} = \begin{pmatrix} 7.844 \\ 5.02 \\ 3.028 \\ 2.224 \\ 1.505 \end{pmatrix} \cdot MPa \qquad f_{m.d.glulam.col.fire.y} = \begin{pmatrix} 14.77 \\ 14.77 \\ 22.042 \\ 22.954 \\ 25.939 \end{pmatrix} \cdot MPa$$

Criterion for combined bending moment and compression, EC 1995-1-1, Eq 6.19

$$\boxed{check_{fire.y_i} := \frac{Q_{column.fire_i}}{N_{cr.glulam.col.fire.y_i}} + \frac{\sigma_{m.y.d.column.fire_i}}{f_{m.d.glulam.col.fire.y_i}}}$$

$$check_{fire.y} = \begin{pmatrix} 94.059 \\ 88.546 \\ 50.631 \\ 48.767 \\ 38.475 \end{pmatrix} \cdot \%$$

Utilization ratio due to combined action due to the risk for fire

10.2 Control of timber beams

10.2.1 Loads

$$Q_{beam} = 30.174 \cdot \frac{kN}{m}$$

Load acting on the beams

$$Q_{beam.fire} := \eta_{fire} \cdot Q_{beam} = 18.104 \cdot \frac{kN}{m}$$

Load acting on the beam according to fire

Maximum bending moment

$M_{Ed.max.beam.fire} := \dfrac{Q_{beam.fire} \cdot l_{span.GL}^{2}}{8} = 68.458 \cdot kN \cdot m$

Maximum bending moment occurs in the middle of the span according to fire

Maximum shear force

$V_{Ed.max.beam.fire} := \dfrac{Q_{beam.fire} \cdot l_{span.GL}}{2} = 49.787 \cdot kN$

Maximum shear force occurs in the ends of the beam according to fire

10.2.2 Geometry

$h_{beam.fire} := h_{beam.glulam} - d_{char.n} = 0.522\ m$

Height of the beam after fire

$w_{beam.fire} := w_{beam.glulam} - 2 \cdot d_{char.n} = 0.099\ m$

Width of the beam after fire

$p_{fire.beam} := 2 \cdot h_{beam.fire} + 2 \cdot w_{beam.fire}$

Cirkumference of the column after exposed to fire

$A_{fire.beam} := w_{beam.fire} \cdot h_{beam.fire}$

Cross-section area of the column after exposed to fire

10.2.3 Design strength values

$k_{mod.fire.beam} := 1 - \dfrac{1}{125} \cdot \dfrac{p_{fire.beam} \cdot m}{A_{fire.beam}}$

Modification factor due to fire for compression stress

$f_{v.g.d.glulam.fire} := k_{mod.fire.beam} \cdot k_{fi.glulam} \cdot \dfrac{f_{v.g.k.glulam}}{\gamma_{M.fi}} = 3.251 \cdot MPa$

Design value for shear due to fire

$f_{m.g.d.glulam.fire} := k_{mod.fire.beam} \cdot k_{fi.glulam} \cdot \dfrac{f_{m.g.k.glulam}}{\gamma_{M.fi}} = 27.867 \cdot MPa$

Design value for bending due to fire

10.2.4 Check - Moment capacity of the beams

$\boxed{M_{Ed.max.beam.fire} \leq M_{Rd.glulam.fire}}$

Criterion for the moment capacity of the beam

$M_{Rd.glulam.fire} := f_{m.g.d.glulam.fire} \cdot W_{beam.glulam} = 357.626 \cdot kN \cdot m$

Maximum allowed moment in the glulam beam due to fire

Utilization ratio

$\dfrac{M_{Ed.max.beam.fire}}{M_{Rd.glulam.fire}} = 19.142 \cdot \%$ $\quad 0.191 < 1 = 1$

OK!

10.2.5 Check - Shear capacity of the beams

$\boxed{\tau_{d.glulam.fire} \le f_{v.g.d.glulam.fire}}$

Criteria for the shear capacity of the beam

$$\tau_{d.glulam.fire} := \frac{S_{beam.glulam} \cdot V_{Ed.max.beam.fire}}{I_{beam.glulam} \cdot b_{eff.glulam}} = 0.847 \cdot MPa$$

Design shear force in the glulam beam

Utilization ratio of the beam for the shear capacity due to fire

$$\frac{\tau_{d.glulam.fire}}{f_{v.g.d.glulam.fire}} = 26.047 \cdot \% \qquad 0.260 < 1 = 1$$

OK!

Appendix 11 - Control of torsion

The calculations due to torsion were performed in the excel sheet in Appendix 18.
In this Appendix, the used equations are presented.

$$\rho_{con} := 2300 \frac{kg}{m^3}$$

Density of concrete C 20/25 in the reference building

$$t_{w1} := 300mm$$

Thickness of the shear walls 1-5

$$t_{w2} := 500mm$$

Thickness of shear wall 6

$$H_{shaft.5} := H_{case5} + h_{storey} = 41.07\,m$$

Total height of the shafts. One storey is added on top of the roof for the installations

Normal force in the shafts for case 5

$$N_{w1.5} := \rho_{con} \cdot g \cdot t_{w1} \cdot H_{shaft.5} = 277.904 \cdot \frac{kN}{m}$$

Normal force in the walls with a thickness of 300mm

$$N_{w6.5} := \rho_{con} \cdot g \cdot t_{w2} \cdot H_{shaft.5} = 463.173 \cdot \frac{kN}{m}$$

Normal force in the wall with a thickness of 500mm

Normal force in the shafts for case 3

$$H_{shaft.3} := H_{case3} + h_{storey} = 33.602\,m$$

Total height of the shafts. One storey is added on top of the roof for the installations

$$N_{w1.3} := \rho_{con} \cdot g \cdot t_{w1} \cdot H_{shaft.3} = 227.371 \cdot \frac{kN}{m}$$

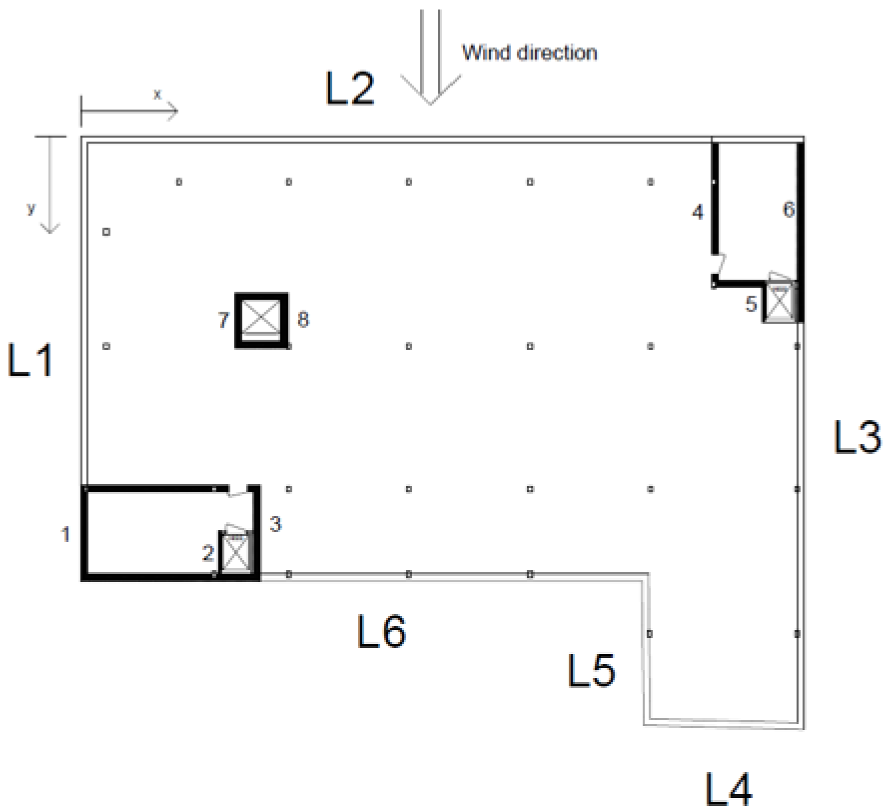
Normal force in the walls with a thickness of 300mm

$$N_{w6.3} := \rho_{con} \cdot g \cdot t_{w2} \cdot H_{shaft.3} = 378.952 \cdot \frac{kN}{m}$$

Normal force in the wall with a thickness of 500mm

11.1 Control of capacity in shear wall 1 for five added floors

The following calculations due to controll of stiffness are made for wall 1 when the wind acting on facade L2 and five new floor were added to the existing building.It will be the same principle for wall 2-8 and also when the wind is acting on facade L3. The results from all the calculations due to torsion moment can be find in Appendix 18.



Data from Appendix 18

$N_{Ed.1} := N_{w1.5} = 277.904 \frac{1}{m^2} \cdot kN \cdot m$ — Normal force per meter in wall 1

$V_{Ed.tot} := 3922.5kN$ — Total vertical load on the bottom floor of the building due to wind load and unintended inclination

$V_{Ed.} := 117.09 \frac{kN}{m}$ — Total vertical load of the bottom floor per meter

$M_{Ed.tot} := 88517.2kN \cdot m$ — Total moment on the bottom floor of the building, due to wind load and unintended inclination

$M_{Ed.} := 2642.31kN$ — Total moment on the bottom floor per meter

$h_F := \dfrac{M_{Ed.}}{V_{Ed.}} = 22.566\,m$ — Equivalent height of load resultant

$y_F := \dfrac{L_2}{2} = 16.75\,m$ — Position of load resultant in y-direction

$L_{wall.1} := 4.1m$ — Depth of wall 1

$y_1 := 0m$ — Location of wall 1

$EI_{tot} := 22.97m^4$ — Sum of the stiffness of wall 1-8, according Appendix 18

$V_{tot} := 610.52m^5$ — Sum of the loaction of wall*relative stiffness for wall 1-8, according to Appendix 18

11.1.1 Calculation of the shear wall capacity

$EI_1 := \dfrac{t_{w1} \cdot L_{wall.1}^{\ 3}}{12} = 1.723\,m^4$ — Relative stiffness of wall 1

$x_{wall} := \dfrac{V_{tot}}{EI_{tot}} = 26.579\,m$ — Rotationcenter in x-direction

$r_{wall1} := x_{wall} = 26.579\,m$ — Distance to rotationcenter for wall 1

$EI_{wall1} := \dfrac{EI_1}{EI_{tot}} = 0.075$ — Amount of shear stiffness of wall 1

$I_{wall1} := EI_1 \cdot r_{wall1}^{\ 2} = 1.217 \times 10^3 \cdot m^6$ — Steiners theory of wall 1

$I_{wall.tot} := 2.71579 \times 10^3 \cdot m^6$ — Steiners theroy of wall 1-8

$I_{rel.1} := \dfrac{I_{wall1}}{I_{wall.tot}} = 0.448$ — Rotation stiffness part in wall 1

11.1.2 Shear force, floor 1

$H_{w.1} := V_{Ed.tot} \cdot EI_{wall1} = 294.234 \cdot kN$ — Horizontal load, wall 1

$M_{H.w1} := h_F \cdot H_{w.1} = 6.64 \times 10^3 \cdot kN \cdot m$ — Moment of horizontal load

11.1.3 Load from rotation floor 1

$M_{tot.rot} := \left|y_F - x_{wall}\right| \cdot V_{Ed.tot} = 3.855 \times 10^{4} \cdot kN \cdot m$ Total rotation moment

$$H_{rot.1} := \frac{M_{tot.rot} \cdot I_{rel.1}}{r_{wall1}} = 650.141 \cdot kN$$ Horizontal load from rotation

$M_{rot.1} := H_{rot.1} \cdot h_F = 1.467 \times 10^{4} \cdot kN \cdot m$ Moment of horizontal rotation

11.1.4 Total load, floor 1

$H_{tot} := H_{w.1} + H_{rot.1} = 944.375 \cdot kN$ Total horizontal load

$M_{rot.tot} := M_{H.w1} + M_{rot.1} = 2.131 \times 10^{4} \cdot kN \cdot m$ Total rotation moment

$$F_{c.M.1} := \frac{M_{rot.tot}}{L_{wall.1}} = 5.198 \times 10^{3} \cdot kN$$ Compression at bottom of wall due to total moment of horizontal loads and rotation

11.1.5 Calculation of the capacity, floor 1

$$M_{Rd.1} := \left(N_{Ed.1} \cdot L_{wall.1}\right) \cdot \frac{L_{wall.1} - t_{w1}}{2} = 2.165 \times 10^{3} \, m \cdot kN$$ Resisting moment

$M_{Rd.extra} := \max\left(\left|M_{rot.tot}\right| - M_{Rd.1}, 0\right) = 1.915 \times 10^{4} \, m \cdot kN$ Extra needed moment

Utilization ratio

$$\frac{M_{rot.tot}}{M_{Rd.1}} = 984.411 \cdot \%$$

NOT OK !

The utilization factor on 984.411% corresponds to the value of the utilization factor on 966.2% , shown in Appendix X. The difference depends propably on the different number of decimals in the calculations due to the different calculations programs.

Appendix 12 -Design of column based connection

In the design of the connection between the columns in the added floors and the existing floor structure, the columns were assumed to be fixed. A nailed connection with steel plates has been chosen

12.1 Control of the load bearing capacity of the nails

12.1.1 Geometry

$t_p := 5mm$ Thickness of the steel plate

Assuming that the nails are quadratic and grooved

$d_{nail} := 4mm$ Diameter of the nail

$d_{head} := 2 \cdot d_{nail} = 8 \cdot mm$ Diameter of the head of the nail

$l_{nail} := 60mm$ Length of the nail

$t_{pen} := l_{nail} - t_p = 55 \cdot mm$ Penetration depth of the nail

$d_{hole} := d_{nail} + 1mm = 5 \cdot mm$ Diameter of the predrilled hole in the steel plate

$n_h := 3$ Number of nails perpendicular to the grain

$n_v := 5$ Number of nails parallell to the grain

12.1.2 Material data

$f_{u.nail} := 600MPa$ Tensile strenght of the nail

$f_{yk.nail} := 235MPa$ Yield strenght of steel S235

$f_{uk} := 340MPa$ Ultimate strenght in the steel plate

$f_{t.0.k} := 19.5MPa$ Tension parallell to the grain in bottom flange and in the web.

$f_{v.g.k} := 3.5MPa$ Characteristic value of the panel shear

$\gamma_{M.connection} := 1.3$ Partial factor due to connection in glulam

$\gamma_{M.steel} := 1.2$ Partial factor, Steel S235. EC1993-1-1, Section 6.1

12.1.3 Loads acting on the connection

$M_{Ed.column} := \max(M_{Ed.column.L2}, M_{Ed.column.L3}) = 6.989 \cdot kN \cdot m$ Design moment acting at the bottom of the column

$H_{column} := F_{3.L2.5} \cdot 5.5m = 58.306 \cdot kN$ Horizontal force acting on the most loaded column in facade L2

$F_{x.MEd} := \dfrac{M_{Ed.column}}{h_{glulam.col_4}} = 19.415 \cdot kN$ Resulting force caused by the moment

$F_y := H_{column} = 58.306 \cdot kN$ Force between the steel plate and the column

12.1.4 Distance between nails

$c_{nail} := 14 \cdot d_{nail} = 0.056\,m$ Smallest distance parallell to the grain direction between the nails, with no reduction EC 1995-1-1, Table 8.1

$\alpha_{nail} := 0$ Angle between direction of the force and direction of the grain

$a_1 := (7 + 8\,|\cos(\alpha_{nail})|) \cdot d_{nail} = 60 \cdot mm$ Smallest distance between nails parallell to the grain

$a_2 := 7 \cdot d_{nail} = 28 \cdot mm$ Smallest distance between nails perpendicular to the grain

$a_3 := (15 + 5 \cdot \cos(\alpha_{nail})) \cdot d_{nail} = 80 \cdot mm$ Smallest distance to the loaded end

Chosen value of the distance between nails in the different directions and needed size for the steel plate

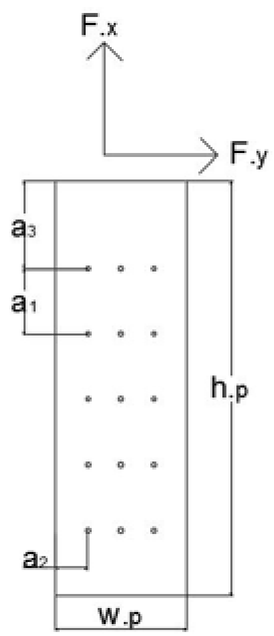
$a_{1.nail} := 60mm$

$a_{2.nail} := 30mm$

$a_{3.nail} := 80mm$

$w_p := 120mm$ Width of the steel plate

$h_p := 380mm$ Height of the steel plate



The figure shows the different distances and direction of loads. Also the number of nails both parallel and perpendicular to the grain

12.2. Control of the load bearing capacity in the steel plate

$\boxed{F_{x.MEd} \le N_{Rd.steel}}$

$A_{net} := t_p \cdot \left(w_p - n_v \cdot d_{hole}\right) = 475 \cdot mm^2$

$N_{Rd.steel} := 0.9 \cdot \dfrac{f_{uk} \cdot A_{net}}{\gamma_{M.steel}} = 121.125 \cdot kN$ Capacity of the steel plate

$\dfrac{F_{x.MEd}}{N_{Rd.steel}} = 0.16$ $\qquad 0.16 \le 1 = 1$ **OK !**

12.2.1 Characteristic embedment strenght

$m_{unit1} := \frac{1}{mm^{-0.3}} \cdot \frac{m^3}{kg} \cdot \frac{1}{g} \cdot MPa$

$f_{h.k} := 0.082 \cdot \rho_{glulam} \cdot d_{nail}^{-0.3} \cdot m_{unit1} = 23.722 \cdot MPa$ No predrillled hole, EC 1995-1-1 Eq 8.15

12.2.2 Yield moment, nail

$m_{unit2} := mm^{0.4}$

$M_{y.Rk} := 0.45 \cdot f_{u.nail} \cdot d_{nail}^{2.6} \cdot m_{unit2} = 9.925 \times 10^3 \cdot N \cdot mm$ Grooved and quadratic nail, EC 1995-1-1 Eq 8.14

12.2.3 Characteristic withdrawal capacity

$m_{unit3} := \frac{m}{N} \cdot kg \cdot \frac{1}{g} \cdot \frac{1}{s^2}$

$\rho := \rho_{glulam} \cdot m_{unit3} = 438.478 \frac{kg}{m^3}$

$m_{unit4} := \frac{10^6 \cdot m^5}{kg \cdot s^2}$

$f_{ax.k} := 20 \cdot 10^{-6} \cdot \rho^2 \cdot m_{unit4} = 3.845 \cdot MPa$ EC 1995-1-1 Eq 8.25

$f_{head.k} := 70 \cdot 10^{-6} \cdot \rho^2 \cdot m_{unit4} = 13.458 \cdot MPa$ EC 1995-1-1 Eq 8.26

$F_{ax.Rk} := \min\left(f_{ax.k} \cdot d_{nail} \cdot t_{pen}, f_{head.k} \cdot d_{head}^2\right) = 0.846 \cdot kN$

$t_p \le 0.5 \cdot d_{nail} = 0$ Not fulfilled -- > Thick steel plate

12.3 Control of load bearing capacity in the nails

Accordning to EC 1995-1-1, Eq 8.10 for thick steel plate

$\boxed{F_{x.MEd} \le F_{R.d.tot}}$ Criterion

$F_{v.Rk.c} := f_{h.k} \cdot t_{pen} \cdot d_{nail} = 5.219 \cdot kN$

$$F_{v.Rk.d} := f_{h.k} \cdot t_{pen} \cdot d_{nail} \cdot \left(\sqrt{2 + \frac{4 \cdot M_{y.Rk}}{f_{h.k} \cdot d_{nail} \cdot t_{pen}^{\,2}}} - 1 \right) + \frac{F_{ax.Rk}}{4} = 2.624 \cdot kN$$

$$F_{v.Rk.e} := 2.3 \cdot \sqrt{M_{y.Rk} \cdot f_{h.k} \cdot d_{nail}} + \frac{F_{ax.Rk}}{4} = 2.443 \cdot kN$$

$$F_{v.Rk} := \min\left(F_{v.Rk.c}, F_{v.Rk.d}, F_{v.Rk.e}\right) = 2.443 \cdot kN$$

$$F_{Rd} := \frac{F_{v.Rk} \cdot k_{mod.glulam}}{\gamma_{M.connection}} = 1.316 \cdot kN$$ Design capacity per nail and shear plane

$$F_{Rd.tot} := n_v \cdot n_h \cdot F_{Rd} = 19.736 \cdot kN$$

Utilization ratio

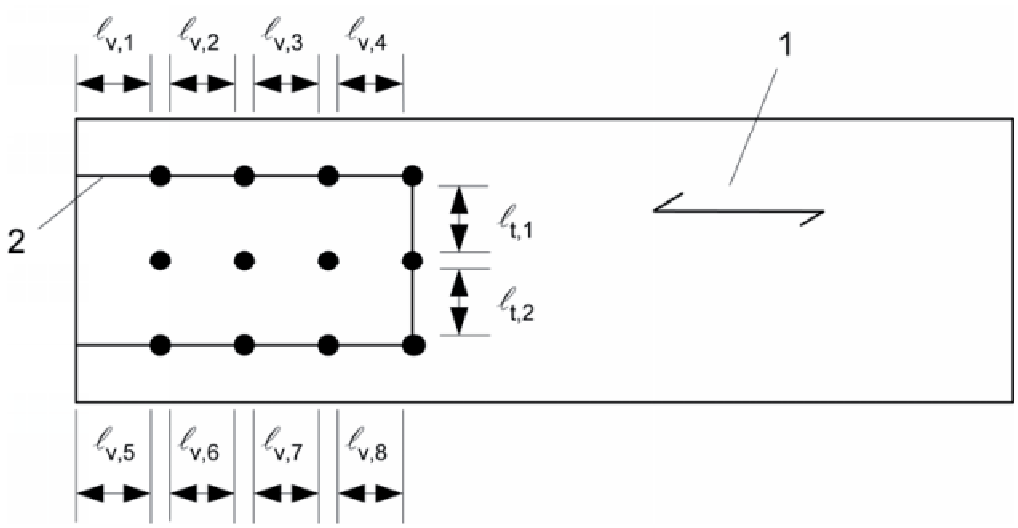
$$\frac{F_{x.MEd}}{F_{Rd.tot}} = 98.375 \cdot \% \qquad 0.984 \le 1 = 1$$ **OK !**

12.4 Control of block tearing

Control of block tearing according to Eurocorde 1995-1-1, Appendix A

$\boxed{F_{bs.Rd} \ge F_{x.MEd}}$ Criterion

The figure below illustrated the different lengths between the nails, parallel and perpendicular to the grain.
1. Parallel to the grain
2. Tensile strenght



$t_{ef.d} := t_{pen} \cdot \left[\sqrt{2 + \frac{M_{y.Rk}}{\left(f_{h.k} \cdot d_{nail} \cdot t_{pen}^2\right)}} - 1\right] = 23.451 \cdot mm$ The effective depth for the nail for failure mode d

$t_{ef.e} := 2 \cdot \sqrt{\frac{M_{y.Rk}}{f_{h.k} \cdot d_{nail}}} = 20.454 \cdot mm$ The effective depth for the nail for failure mode e

$t_{ef} := \min\left(t_{ef.d}, t_{ef.e}\right) = 20.454 \cdot mm$

$l_v := a_1 - d_{head} = 52 \cdot mm$ Net length between the nails parallell to the grain

$l_t := a_2 - d_{head} = 20 \cdot mm$ Net length between the nails perpendicular to the grain

$L_{net.v} := n_v \cdot l_v = 260 \cdot mm$ Total net length for the shear failure

$L_{net.t} := \left(n_h - 1\right) \cdot l_t = 40 \cdot mm$ Total net length for the tension failure

$A_{net.v} := \frac{L_{net.v}}{2} \cdot \left(L_{net.t} + 2 \cdot t_{ef}\right) = 0.011\, m^2$ Net area parallell to the grain

$A_{net.t} := L_{net.t} \cdot t_{pen} = 2.2 \times 10^{-3}\, m^2$ Net area perpendicular to the grain

12.4.1 Design capacity of block tearing

$F_{bs.Rk.1} := 1.5 \cdot A_{net.t} \cdot f_{t.0.k.GL} = 64.35 \cdot kN$

$F_{bs.Rk.2} := 0.7 \cdot A_{net.v} \cdot f_{v.g.k} = 25.769 \cdot kN$

$F_{bs.Rk} := \max\left(F_{bs.Rk.1}, F_{bs.Rk.2}\right) = 64.35 \cdot kN$

$F_{bs.Rd} := \frac{k_{mod.glulam} \cdot F_{bs.Rk}}{\gamma_{M.connection}} = 34.65 \cdot kN$

Utlization ratio

$\frac{F_{x.MEd}}{F_{bs.Rd}} = 56.031 \cdot \%$ $\quad 0.56 \le 1 = 1$ **OK !**

12.5 Design of the anchoring

For the anchoring a HST expander bolt from Hilti has been chosen according standard dimensions in Anchor Fastening Technology Manual. The dimensions have been chosen to resist the actual tension and shear loads.

HST anchoring from Hilti:

$l_{anchor} := 90mm$ Needed anchoring length

$d_{bolt} := 10mm$ Diameter of expander bolts

$c_{cr} := 90mm$ Critical edge distance from center of bolt to concrete edge

$c_{min} := 55mm$ Minimum distance to edge

$f_{yk} := 640 \frac{N}{mm^2}$ Shear yield strength for a expander bolt, HST

15.5.1 Number of bolts

$F_{Rd.bolt} := f_{yk} \cdot \left(\frac{\pi \cdot d_{bolt}^2}{4} \right) = 50.265 \cdot kN$ Shear capacity of a bolt

$n_{bolts} := \frac{F_y}{F_{Rd.bolt}} = 1.16$ Number of bolt needed for the anchoring of a column

Two bolts are needed for the anchoring of a column

15.5.2 Contol of combined shear and moment in the anchoring

$F_{up} := \frac{F_{x.MEd}}{2} = 9.707 \cdot kN$ Uplifitng force in the anchoring

$M_{anchor} := F_{up} \cdot c_{cr} = 0.874 \cdot kN \cdot m$ Moment in the anchor

$\tau_{anchor} := \frac{F_{up}}{A_{net}} = 20.437 \cdot MPa$ Shear in the anchor

$w_{anchor} := \frac{t_p \cdot w_p^2}{6} = 1.2 \times 10^{-5} \cdot m^3$ Bending resistance for the steel plate

$$\sigma_{anchor} := \frac{M_{anchor}}{w_{anchor}} = 72.806 \cdot MPa$$

Stresses in the steel plate

$$F_{anchor} := \tau_{anchor} + \sigma_{anchor} = 93.243 \cdot MPa$$

Worst case - Both shear stress and bendning moment in the anchor

Utilization ratio

$$\boxed{F_{anchor} \le f_y}$$

Criterion

$$\frac{F_{anchor}}{f_y} = 39.678 \cdot \%$$

$$0.397 \le 1 = 1$$

OK !

Appendix 13 - Estimation of self-weights

The calculations follow the principles of Eurocode 1991. The reason for estimating the self-weight for timber and steel was to back up the lower scoring for timber in the evaluation phase. Worth mentioning is that the calculations in this appendix are just rough estimations.

13.1 Calculation of loads

13.1.1 Snow load

$\mu_1 := 0.8$

Snow load shape coefficient, angle of roof less than 30 degrees.
From EC 1991-1-3, Table 5.2

$s_k := 1.5 \frac{kN}{m^2}$

Characteristic snow load in Gothenburg
From EC 1991-1-3, Table NB:1

$C_e := 1$

Exposure coefficient

$C_t := 1$

Thermal coefficient

$S := \mu_1 \cdot C_e \cdot C_t \cdot s_k = 1.2 \cdot \frac{kN}{m^2}$

Snow load, EC 1991-1-3 Eq 5.1

$\psi_{0.s} := 0.6$

Since 1.0<s.k<2.0

13.1.2 Imposed load

$q_k := 2.0 \frac{kN}{m^2}$

Imposed load for residental building
From EC 1991-1-1, Table 6.2

$\psi_{0.i} := 0.7$

13.1.3 Geometry

$A := 7.5m \cdot 5.5m = 41.25 m^2$

The largest tributary area resisted by the columns

$h_{column} := 2.8m$

The height of the column

13.1.4 Load combinations

According to EC 1990 eq 6.10a the followin combinations can be put up:

Snow load is main load

$Q_1 := A \cdot \left(1.5 \cdot S + 1.5 \cdot \psi_{0.i} \cdot q_k\right) = 160.875 \cdot kN$

Imposed load is main load

$$Q_2 := A\cdot\left(1.5\cdot q_k + 1.5\cdot\psi_{0.s}\cdot S\right) = 168.3\cdot kN$$

$$Q_{max} := \max\left(Q_1, Q_2\right) = 168.3\cdot kN$$

13.2 Estimation of total weight for a column

Acording to the maximum acting load Q.max, a VKR-profile with dimensions 80x80x4.0 mm is needed according to the tables from the manufacturer Tibnor.

$$q_{VKR} := 9.41\frac{kg}{m}\cdot g = 92.281\cdot\frac{N}{m}$$

The weight of one steel column due to the maximum load

For the same load for a timber column, a GL32c column with dimensions 140x135 mm is needed according to the webpage of Svenskt Trä.

$$\rho_{GL.32} := 390\frac{kg}{m^3}$$

Characteristic density of GL32c

$$q_{GL32} := 0.140m\cdot 0.135m\cdot\rho_{GL.32}\cdot g = 72.285\cdot\frac{N}{m}$$

The weight of one timber column due to the maximum load

The load is assumed to be doubled to find the influence of the weight when increased dimensions are needed. For this case, the dimensions to resist a doubled load are presented below.

VKR-profile 100x100x5.0 mm

$$q_{VKR.2} := 14.7\frac{kg}{m}\cdot g = 144.158\cdot\frac{N}{m}$$

The weight of one steel column due to a doubled maximum load

GL32c, 165x180 mm

$$q_{GL30.2} := 0.165m\cdot 0.180m\cdot\rho_{GL.32}\cdot g = 113.59\cdot\frac{N}{m}$$

The weight of one timber column due to a doubled maximum load

Appendix 14

Capcacity of the columns in the reference building at start

Column number	Capacity of the columns (kN)	Load at bottom columns (kN)	Utilization ratio due to buckling
P1	3206	1647	51,4%
P2	3206	2189	68,3%
P3	3206	2405	75,0%
P4	3206	1759	54,9%
P5	3206	1382	43,1%
P6	3206	1141	35,6%
P7	3206	1539	48,0%
P8	2324	1513	65,1%
P9	1902	652	34,3%
P10	3206	2188	68,2%
P11	3206	2843	88,7%
P12	3206	1946	60,7%
P13	3206	1531	47,8%
P14	3206	1759	54,9%
P15	3206	1325	41,3%
P16	3206	1360	42,4%
P17	3206	1500	46,8%
P18	3206	1548	48,3%
P19	2520	957	38,0%
P20	2520	1092	43,3%
P21	3206	1634	51,0%
P22	3206	1136	35,4%
P23	2324	809	34,8%
P24	3206	815	25,4%
P25	3206	668	20,8%
P26	3206	724	22,6%
P27	2324	1033	44,4%
P28	2324	829	35,7%

Appendix 15

Capacity of the columns in the reference building when floors 7,8 and 9 are removed

Column number	Capacity of the columns (kN)	Load at bottom columns (kN)	Utilization ratio due to buckling
P1	3206	1416	44,2%
P2	3206	1854	57,8%
P3	3206	2012	62,8%
P4	3206	1759	54,9%
P5	3206	1382	43,1%
P6	3206	1141	35,6%
P7	3206	1277	39,8%
P8	2324	1498	64,5%
P9	1902	637	33,5%
P10	3206	1771	55,2%
P11	3206	2102	65,6%
P12	3206	1563	48,8%
P13	3206	1531	47,8%
P14	3206	1759	54,9%
P15	3206	1273	39,7%
P16	3206	1109	34,6%
P17	3206	1166	36,4%
P18	3206	1182	36,9%
P19	2520	957	38,0%
P20	2520	1087	43,1%
P21	3206	1634	51,0%
P22	3206	1136	35,4%
P23	2324	600	25,8%
P24	3206	572	17,8%
P25	3206	668	20,8%
P26	3206	704	22,0%
P27	2324	1033	44,4%
P28	2324	829	35,7%

Appendix 16

Capacity of the columns in the reference building when five new floors were added

Column number	Capacity of the columns (kN)	Start load bottom columns (kN)	Utilization ratio due to buckling
P1	3206	1416	44,2%
P2	3206	1854	57,8%
P3	3206	2012	62,8%
P4	3206	1759	54,9%
P5	3206	1382	43,1%
P6	3206	1141	35,6%
P7	3206	1277	39,8%
P8	2324	1498	64,5%
P9	1902	637	33,5%
P10	3206	1771	55,2%
P11	3206	2988	93,2%
P12	3206	1563	48,8%
P13	3206	1531	47,8%
P14	3206	1759	54,9%
P15	3206	1273	39,7%
P16	3206	1109	34,6%
P17	3206	1166	36,4%
P18	3206	1182	36,9%
P19	2520	957	38,0%
P20	2520	1087	43,1%
P21	3206	1634	51,0%
P22	3206	1136	35,4%
P23	2324	600	25,8%
P24	3206	572	17,8%
P25	3206	668	20,8%
P26	3206	704	22,0%
P27	2324	1033	44,4%
P28	2324	829	35,7%

Appendix 17

- Control of the cassette floor in ULS
- Control of the cassette floor in SLS

Control of cassette floor - ULS

Initial calculation	Part	h	b	A	E_{mean}	EA	z	EAz	EA(Zna-Z)	EA(Zna-Z)²	Ebh^3/12	EI	G	GA
CLT, Cross laminated timber	1	0,082	0,6	0,0492	7,00E+09	3,44E+08	0,041	1,41E+07	2,90E+07	2,45E+06	1,93E+05	2,64E+06	4,40E+08	21648000
Glulam GL30c	2	0,211	0,045	0,009495	1,30E+10	1,23E+08	0,1875	2,31E+07	-7,67E+06	4,77E+05	4,58E+05	9,35E+05	7,60E+08	7216200
Glulam GL30c	3	0,056	0,15	0,0084	1,30E+10	1,09E+08	0,321	3,51E+07	2,14E+07	4,18E+06	2,85E+04	4,21E+06	7,60E+08	6384000
								z_{na} =	0,1253	m	$\sum EI_{tot}$=	7,79E+06		

Moment qL^2/8		
q_{uls} =	2,07	kN/m
q_{sls} =	1,87	kN/m
L =	7,50	m
M_{uls} =	14,54	kNm
M_{sls} =	13,16	kNm

Shear force		
V_{uls} =	7,76	kN
V_{sls} =	7,02	kN

Formulas for stresses	
$\sigma_{c,t}$ =	M/∑Eitot*Emean*yi
τ_d =	V*(S*Ei,d)/(∑Eitot*b)

Formula for neutral axis	
z =	(∑EA)/(∑EAz)

Check of stresses (initial) **Position of stress**	$\sigma_{c,t}$	y_i	$f_{c/t,d}$	τ_d	S1	S2	$f_{v,d}$	%	%	Type of stress
Top flange, mid	1,10E+06	0,08433	1,23E+07					9,0		1c (Bending, compression)
Top flange, bottom		0,0433		6,43E+05	4,15E-03	0	2,33E+06	27,6		1pv (Panel shear)
Web, top	1,05E+06	0,04333	1,37E+07	1,19E+06	4,15E-03	0	1,96E+06	7,7	60,9	2c,pv (Bending, panel shear)
Web, neutral axis		0,12533		6,55E+05	4,15E-03	0,000042	1,96E+06	33,4		2na,v (Panel shear)
Web, bottom	4,07E+06	0,16767	1,09E+07	4,73E+05	1,64E-03	0	1,96E+06	37,3	24,1	2t,pv (Bending, panel shear)
Bottom flange, top		0,16767		4,73E+05	1,64E-03	0	1,96E+06	24,1		3pv (Panel shear)
Bottom flange, mid	4,75E+06	0,19567	1,09E+07					43,5		3t (Bending, tension)

Initial deflection check	$u_{inst,M}$	$u_{inst,V}$	$u_{inst,tot}$	**Limit** < 20 mm	CHECK	%	
	9,90E-03	0,00219	0,012087	0,02	OK!	60,43	

Formulas for deflection	
$u_{inst,M}$ =	5/384*qsls* L^4/∑EItot
$u_{inst,V}$ =	1,2/8*qsls* L^2/(G*A)

Control of cassette floor - SLS

Final calculation	Part	h	b	A	E_{mean}	$\Psi_2 * k_{def}$	$E_{d,fin}$	EA	z	EAz	EA(zna-z)	EA(Zna-Z)²	Ebh^3/12	∑EI	G	GA
CLT, Cross laminated timber	1	0,082	0,6	0,0492	7,00E+09	0,24	5,6E+09	2,78E+08	0,041	11387419	23420950	1975002	155628,1	2130630	4,40E+08	2,16E+07
Glulam GL30c	2	0,211	0,045	0,009495	1,30E+10	0,24	1E+10	99544355	0,1875	18664567	6189041	384795,5	369317,9	754113,4	7,60E+08	7,22E+06
Glulam GL30c	3	0,056	0,15	0,0084	1,30E+10	0,24	1E+10	88064516	0,321	28268710	17231910	3371831	23014,19	3394846	7,60E+08	6,38E+06
						Ψ_2 =	0,3									
						k_{def} =	0,8						**∑EItot=**	6,28E+06		

Check of stresses (final) Position of stress	$\sigma_{c,t}$	y_i	$f_{c/t,d}$	τ_d	S1	S2	$f_{v,d}$	%	%	Type of stress
Top flange, mid	2,83E+05	0,02166	1,23E+07					2,3		1c (Bending, compression)
Top flange, bottom		0,04333		6,43E+05	4,15E-03	0	2,33E+06	27,6		1pv (Panel shear)
Web, top	1,05E+06	0,04333	1,37E+07	1,19E+06	4,15E-03	0	1,96E+06	7,7	60,9	2c,pv (Bending, panel shear)
Neutral axis		0,12533		6,55E+05	4,15E-03	4,22E-05	1,96E+06	33,4		2na,pv (Panel shear)
Web, bottom	4,07E+06	0,16767	1,09E+07	4,73E+05	1,64E-03	0	1,96E+06	37,3	24,1	2t,pv (Bending, panel shear)
Bottom flange, top		0,16767		4,73E+05	1,64E-03	0	1,96E+06	24,1		3pv (Panel shear)
Bottom flange, mid	4,75E+06	0,19567	1,09E+07					43,6		3t (Bending, tension)

Final deflection check	$u_{fin,M}$	$u_{fin,V}$	$u_{fin,tot}$	Limit < 20 mm	CHECK	%	
	0,01228	0,00219	0,014463	0,0375	OK!	38,57	

Formulas for deflection	
$u_{inst,M}$ =	5/384*qsls* L^4/∑EItot
$u_{inst,V}$ =	1,2/8*qsls* L^2/(G*A)

Appendix 18

Control of capacity of the shear walls

- Five added floors when façade L2 is exposed to wind loads
- Five added floors when façade L3 is exposed to wind loads
- Three added floors when façade L2 is exposed to wind loads
- Three added floors when façade L3 is exposed to wind loads

Control of capacity in the shear walls when facade L2 is exposed to wind

Five added floors

Legend

Legend	
Frequent indata	
Infrequent indata	
Partial results	
Results	
Critical results	

Table for the shear walls parallel to wind direction

Information about the walls					**Calculated values**				
Wall no.	Rel. stiffness	y [m]	d [m]	N [kN/m]	y*stiffn.	r [m]	rel. stiffness	I_wall	rel. stiffn, rot
Comments	*Relative*	*Location of wall*	*Depth of wall*	*Normalforce in the wall per m*	-	*Distance to rotationc.*	*Amount of shear stiffn. per wall*	*Steiners theory per wall*	*Rot.stiff.part in each wall*
1	1,7	0,0	4,1	277,9	0,00	26,58	0,08	1217,63	0,44
2	0,2	6,3	2,0	277,9	1,26	20,28	0,01	82,29	0,03
3	1,7	8,1	4,1	277,9	13,96	18,48	0,08	588,66	0,21
4	6,9	28,9	6,5	277,9	199,41	-2,32	0,30	37,02	0,01
5	0,2	31,1	2,0	277,9	6,22	-4,52	0,01	4,08	0,00
6	11,4	33,5	6,5	458,7	383,24	-6,92	0,50	547,23	0,20
7	0,4	7,2	2,5	277,9	2,81	19,38	0,02	146,53	0,05
8	0,4	9,3	2,5	277,9	3,63	17,28	0,02	116,50	0,04
Sum of walls 1-8	22,97				610,52			2739,95	

Wall no.	**Shear force (Floor 1)**		**Load from rotation (floor 1)**		**Total load (floor 1)**			**Check of the capacity**			**Anchoring**	
Wall no.	H_W [kN]	M_W [kNm]	H_rot [kN]	M_rot [kNm]	H_tot [kN]	M_tot [kNm]	Fc_M_tot [kN]	M_N [kNm]	M_rest [kNm]	Utilization	F_a [kN]	A_s_req
Comments	*Hor.load per wall*	*Moment of H_W*	*Hor.load from rotation*	*Moment of H_rot*	*H_W + H_rot*	*M_V + M_rot*	*Compr. at bottom due to M_tot*	*Resisting moment*	*Extra needed moment*		*Necessary anchoring*	*Necessary steel area*
1	294,5	6647,8	645,2	14566,4	939,7	21214,1	5174,2	2193,4	19020,8	967,2%	4639,2	1,07E-02
2	34,2	771,6	57,1	1290,1	91,3	2061,8	1030,9	486,3	1575,4	423,9%	787,7	1,81E-03
3	294,5	6647,8	448,6	10128,0	743,1	16775,8	4091,7	2193,4	14582,4	764,8%	3556,7	8,18E-03
4	1179,2	26621,9	-225,1	-5082,7	954,1	21539,2	3313,7	5644,9	15894,3	381,6%	2445,3	5,62E-03
5	34,2	771,6	-12,7	-287,3	21,5	484,4	242,2	486,3	0,0	99,6%	0,0	0,00E+00
6	1955,1	44138,3	-1114,6	-25162,3	840,5	18976,1	2919,4	9317,3	9658,7	203,7%	1486,0	3,42E-03
7	66,7	1504,7	106,5	2404,1	173,1	3908,8	1563,5	781,6	3127,2	500,1%	1250,9	2,88E-03
8	66,7	1504,7	95,0	2143,6	161,6	3648,4	1459,3	781,6	2866,8	466,8%	1146,7	2,64E-03
Sum of walls 1-8	3924,9	88608,5										

Geometry

h_b	-3,70	*m*	*Height over base of wall*
l_x	33,5	*m*	*Length of building perpendicular to load*
l_y	20,3	*m*	*Depth of building parallel to load*
x	26,58	*m*	*Rotationcenter in x-direction*
y	10,15	*m*	*Rotationcenter in y-direction*
h_F	22,6	*m*	*Equ. height of load resultant*
y_F	16,75	*m*	*Position of load resultant in y-direction*
M_rot	38596,6	*kNm*	*Total rotation moment*

Table with loads

Loads	*Wind load*	*Unintended inclination*		*Height floor*	*F_tot_ULS*	*M_ULS*
No. of floors	[kN/m]	[kN]		[m]	[kN]	[kNm]
11	6,39	1,33	0,00	37,87	215,36	8952,66
10	11,61	5,16	0,00	34,14	394,22	14915,86
9	10,79	5,16	0,00	30,40	366,59	12501,33
8	10,79	5,16	0,00	26,67	366,59	11132,50
7	10,79	5,16	0,00	22,93	366,59	9763,66
6	10,60	18,38	0,00	19,20	373,32	8548,92
5	9,25	23,12	0,00	16,00	332,86	6557,28
4	9,25	23,12	0,00	12,80	332,86	5492,14
3	9,25	23,12	0,00	9,60	332,86	4427,00
2	9,25	23,12	0,00	6,40	332,86	3361,86
1	9,25	23,12	0,00	3,20	332,86	2296,71
0	4,62	23,12	0,00	0,00	177,99	658,55
-1	0,00	0,00	0,00	-3,70	0,00	0,00
Total load/meter	111,82	179,03	0,00	-	117,16	2645,03
Total load building	3745,9	5997,5	0,0	-	3924,9	88608,5

Control of capacity in the shear walls when facade L3 is exposed to wind

Three added floors

Legend

Frequent indata	
Infrequent indata	
Partial results	
Results	
Critical results	

Table for the shear walls parallel to wind direction

Information about the walls					**Calculated values**					**Shear force (Floor 1)**		**Load from rotation (floor 1)**		**Total load (floor 1)**			**Check of the capacity**			**Anchoring**	
Wall no.	Rel. stiffness	y [m]	d [m]	N [kN/m]	y*stiffn.	r [m]	rel. stiffness	I_wall	rel. stiffn, rot	H_W [kN]	M_W [kNm]	H_rot [kN]	M_rot [kNm]	H_tot [kN]	M_tot [kNm]	Fc_M_tot [kN]	M_N [kNm]	M_rest [kNm]	Utilization	F_a [kN]	A_s_req
Comments	*Relative*	*Location of wall*	*Depth of wall*	*Normalforce in the wall per m*	*-*	*Distance to rotationc.*	*Amount of shear stiffn. per wall*	*Steiners theory per wall*	*Rot.stiff.part in each wall*	*Hor.load per wall*	*Moment of H_W*	*Hor.load from rotation*	*Moment of H_rot*	*H_W + H_rot*	*M_V + M_rot*	*Compr. at bottom due to M_tot*	*Resisting moment*	*Extra needed moment*		*Necessary anchoring*	*Necessary steel area*
9	13,8	20,3	8,2	277,9	279,73	-3,58	0,46	177,00	0,23	1469,4	33429,6	-1,3	-29,9	1468,1	33399,7	4073,1	9058,3	24341,4	368,7%	2968,5	6,82E-03
10	13,8	15,9	8,2	277,9	219,10	0,82	0,46	9,18	0,01	1469,4	33429,6	0,3	6,8	1469,7	33436,4	4077,6	9058,3	24378,1	369,1%	2972,9	6,83E-03
11	0,4	9,7	2,4	277,9	3,40	7,02	0,01	17,23	0,02	37,3	849,1	0,1	1,5	37,4	850,6	354,4	717,0	133,6	118,6%	55,7	1,28E-04
12	0,4	7,2	2,4	277,9	2,52	9,52	0,01	31,69	0,04	37,3	849,1	0,1	2,0	37,4	851,1	354,6	717,0	134,1	118,7%	55,9	1,28E-04
13	0,2	8,3	1,9	277,9	1,41	8,42	0,01	12,04	0,02	18,1	412,4	0,0	0,9	18,2	413,3	217,5	435,6	0,0	94,9%	0,0	0,00E+00
14	1,9	0,0	4,2	277,9	0,00	16,72	0,06	516,94	0,68	197,3	4488,0	0,8	18,7	198,1	4506,7	1073,0	2305,2	2201,5	195,5%	524,2	1,20E-03
Sum	30,28				506,16			764,08		3228,9	73457,7										

Geometry

h_b	-3,70	*m*	*Height over base of wall*
l_y	20,3	*m*	*Length of building perpendicular to load*
l_x	33,5	*m*	*Depth of building parallel to load*
y	16,72	*m*	*Rotationcenter in y-direction*
x	16,75	*m*	*Rotationcenter in x-direction*
h_F	22,8	*m*	*Equ. height of load resultant*
y_F	10,15	*m*	*Position of load resultant in y-direction*
M_rot	21201,0	*kNm*	*Total rotation moment*

Table with loads

Loads	*Wind load*	*Unintended inclination*		*Height floor*	*F_tot_ULS*	*M_ULS*
No. of floors	[kN/m]	[kN]		[m]	[kN]	[kNm]
11	6,37	0,00	0,00	37,87	192,88	8018,17
10	11,51	0,00	0,00	34,14	348,43	13183,27
9	9,57	1,33	0,00	30,40	291,02	9924,39
8	9,57	5,16	0,00	26,67	294,84	8953,82
7	9,57	5,16	0,00	22,93	294,84	7852,87
6	9,40	18,38	0,00	19,20	302,86	6935,58
5	8,20	23,12	0,00	16,00	271,35	5345,62
4	8,20	23,12	0,00	12,80	271,35	4477,30
3	8,20	23,12	0,00	9,60	271,35	3608,97
2	8,20	23,12	0,00	6,40	271,35	2740,65
1	8,20	23,12	0,00	3,20	271,35	1872,32
0	4,10	23,12	0,00	0,00	147,23	544,76
-1	0,00	0,00	0,00	-3,70	0,00	0,00
Total load/meter	101,06	168,72	0,00	-	159,06	3618,61
Total load building	2051,6	3425,0	0,0	-	3228,9	73457,7

Control of capacity in the shear walls when facade L2 is exposed to wind

Three added floors

Legend

Frequent indata	(yellow)
Infrequent indata	(orange)
Partial results	(blue)
Results	(green)
Critical results	(red)

Table for the shear walls parallel to wind direction

Information about the walls					**Calculated values**					**Shear force (Floor 1)**		**Load from rotation (floor 1)**		**Total load (floor 1)**			**Check of the capacity**			**Anchoring**	
Wall no.	Rel. stiffness	x [m]	d [m]	N [kN/m]	y*stiffn.	r [m]	rel. stiffness	I_wall	rel. stiffn, rot	H_W [kN]	M_W [kNm]	H_rot [kN]	M_rot [kNm]	H_tot [kN]	M_tot [kNm]	Fc_M_tot [kN]	M_N [kNm]	M_rest [kNm]	Utilization	F_a [kN]	A_s_req
Comments	*Relative*	*Location of wall*	*Depth of wall*	*Normalforce in the wall per m*	-	*Distance to rotationc.*	*Amount of shear stiffn. per wall*	*Steiners theory per wall*	*Rot.stiff.part in each wall*	*Hor.load per wall*	*Moment of H_W*	*Hor.load from rotation*	*Moment of H_rot*	*H_W + H_rot*	*M_V + M_rot*	*Compr. at bottom due to M_tot*	*Resisting moment*	*Extra needed moment*		*Necessary anchoring*	*Necessary steel area*
1	1,7	0,0	4,1	227,4	0,00	26,58	0,08	1217,63	0,44	223,9	4197,4	490,6	9197,3	714,4	13394,7	3267,0	1794,5	11600,1	746,4%	2829,3	6,50E-03
2	0,2	6,3	2,0	227,4	1,26	20,28	0,01	82,29	0,03	26,0	487,2	43,4	814,6	69,4	1301,8	650,9	397,9	903,9	327,2%	452,0	1,04E-03
3	1,7	8,1	4,1	227,4	13,96	18,48	0,08	588,66	0,21	223,9	4197,4	341,1	6394,9	565,0	10592,3	2583,5	1794,5	8797,8	590,3%	2145,8	4,93E-03
4	6,9	28,9	6,5	227,4	199,41	-2,32	0,30	37,02	0,01	896,6	16809,2	-171,2	-3209,2	725,4	13599,9	2092,3	4618,5	8981,5	294,5%	1381,8	3,18E-03
5	0,2	31,1	2,0	227,4	6,22	-4,52	0,01	4,08	0,00	26,0	487,2	-9,7	-181,4	16,3	305,9	152,9	397,9	0,0	76,9%	0,0	0,00E+00
6	11,4	33,5	6,5	379,0	383,24	-6,92	0,50	547,23	0,20	1486,5	27869,1	-847,4	-15887,6	639,1	11981,6	1843,3	7697,5	4284,1	155,7%	659,1	1,52E-03
7	0,4	7,2	2,5	227,4	2,81	19,38	0,02	146,53	0,05	50,7	950,1	81,0	1518,0	131,6	2468,0	987,2	639,5	1828,6	385,9%	731,4	1,68E-03
8	0,4	9,3	2,5	227,4	3,63	17,28	0,02	116,50	0,04	50,7	950,1	72,2	1353,5	122,9	2303,6	921,4	639,5	1664,1	360,2%	665,6	1,53E-03
Sum	22,97				610,52			2739,95		2984,1	55947,7										

Geometry

h_b	-3,70	m	*Height over base of wall*
l_x	33,5	m	*Length of building perpendicular to load*
l_y	20,3	m	*Depth of building parallel to load*
x	26,58	-	*Rotationcenter in x-direction*
y	10,15	-	*Rotationcenter in y-direction*
h_F	18,7	*m*	*Equ. height of load resultant*
y_F	16,75	*m*	*Position of load resultant in y-direction*
M_rot	29344,7	*kNm*	*Total rotation moment*

Table with loads

Loads	*Wind load*	*Unintended inclination*		*Height floor*	*F_tot_ULS*	*M_ULS*
No. of floors	[kN/m]	[kN]		[m]	[kN]	[kNm]
11	0,00	0,00	0,00	37,87	0,00	0,00
10	0,00	0,00	0,00	34,14	0,00	0,00
9	5,64	1,33	0,00	30,40	190,27	6488,66
8	10,19	5,16	0,00	26,67	346,45	10521,08
7	10,19	5,16	0,00	22,93	346,45	9227,43
6	10,01	18,38	0,00	19,20	353,55	8096,31
5	8,73	23,12	0,00	16,00	315,60	6217,41
4	8,73	23,12	0,00	12,80	315,60	5207,47
3	8,73	23,12	0,00	9,60	315,60	4197,54
2	8,73	23,12	0,00	6,40	315,60	3187,61
1	8,73	23,12	0,00	3,20	315,60	2177,67
0	4,37	23,12	0,00	0,00	169,34	626,57
-1	0,00	0,00	0,00	-3,70	0,00	0,00
Total load/meter	84,04	168,72	0,00	-	89,08	1670,08
Total load building	2815,4	5652,2	0,0	-	2984,1	55947,7

Control of capacity in the shear walls when facade L3 is exposed to wind

Three added floors

Legend

Frequent indata
Infrequent indata
Partial results
Results
Critical results

Table for the shear walls parallel to wind direction

Information about the walls					**Calculated values**					**Shear force (Floor 1)**		**Load from rotation (floor 1)**		**Total load (floor 1)**			**Check of the capacity**			**Anchoring**	
Wall no.	Rel. stiffness	y [m]	d [m]	N [kN/m]	y*stiffn.	r [m]	rel. stiffness	I_wall	rel. stiffn, rot	H_W [kN]	M_W [kNm]	H_rot [kN]	M_rot [kNm]	H_tot [kN]	M_tot [kNm]	Fc_M_tot [kN]	M_N [kNm]	M_rest [kNm]	Utilization	F_a [kN]	A_s_req
Comments	*Relative*	*Location of wall*	*Depth of wall*	*Normalforce in the wall per m*	-	*Distance to rotationc.*	*Amount of shear stiffn. per wall*	*Steiners theory per wall*	*Rot.stiff.part in each wall*	*Hor.load per wall*	*Moment of H_W*	*Hor.load from rotation*	*Moment of H_rot*	*H_W + H_rot*	*M_V + M_rot*	*Compr. at bottom due to M_tot*	*Resisting moment*	*Extra needed moment*		*Necessary anchoring*	*Necessary steel area*
9	13,8	20,3	8,2	227,4	279,73	-3,58	0,46	177,00	0,23	1158,5	21813,1	-1,3	-24,7	1157,2	21788,3	2657,1	7411,2	14377,2	294,0%	1753,3	4,03E-03
10	13,8	15,9	8,2	227,4	219,10	0,82	0,46	9,18	0,01	1158,5	21813,1	0,3	5,6	1158,8	21818,7	2660,8	7411,2	14407,5	294,4%	1757,0	4,04E-03
11	0,4	9,7	2,4	227,4	3,40	7,02	0,01	17,23	0,02	29,4	554,0	0,1	1,2	29,5	555,3	231,4	586,6	0,0	94,7%	0,0	0,00E+00
12	0,4	7,2	2,4	227,4	2,52	9,52	0,01	31,69	0,04	29,4	554,0	0,1	1,7	29,5	555,7	231,5	586,6	0,0	94,7%	0,0	0,00E+00
13	0,2	8,3	1,9	227,4	1,41	8,42	0,01	12,04	0,02	14,3	269,1	0,0	0,7	14,3	269,8	142,0	356,4	0,0	75,7%	0,0	0,00E+00
14	1,9	0,0	4,2	227,4	0,00	16,72	0,06	516,94	0,68	155,5	2928,5	0,8	15,5	156,3	2943,9	700,9	1886,0	1057,9	156,1%	251,9	5,79E-04
Sum	30,28				506,16			764,08		2545,6	47931,7										

Geometry

h_b	-3,70	*m*	*Height over base of wall*
l_y	20,3	*m*	*Length of building perpendicular to load*
l_x	33,5	*m*	*Depth of building parallel to load*
x	16,72	*m*	*Rotationcenter in x-direction*
y	16,75	*m*	*Rotationcenter in y-direction*
h_F	18,8	*m*	*Equ. height of load resultant*
y_F	10,15	*m*	*Position of load resultant in y-direction*
M_rot	16714,6	*kNm*	*Total rotation moment*

Table with loads

Loads	*Wind load*	*Unintended inclination*		*Height floor*	*F_tot_ULS*	*M_ULS*
No. of floors	[kN/m]	[kN]		[m]	[kN]	[kNm]
11	0,00	0,00	0,00	37,87	0,00	0,00
10	0,00	0,00	0,00	34,14	0,00	0,00
9	5,48	1,33	0,00	30,40	167,27	5704,12
8	9,89	5,16	0,00	26,67	304,62	9250,83
7	9,89	5,16	0,00	22,93	304,62	8113,36
6	9,11	18,38	0,00	19,20	294,29	6739,34
5	7,95	23,12	0,00	16,00	263,90	5198,88
4	7,95	23,12	0,00	12,80	263,90	4354,39
3	7,95	23,12	0,00	9,60	263,90	3509,90
2	7,95	23,12	0,00	6,40	263,90	2665,42
1	7,95	23,12	0,00	3,20	263,90	1820,93
0	4,37	23,12	0,00	0,00	155,29	574,57
-1	0,00	0,00	0,00	-3,70	0,00	0,00
Total load/meter	78,50	168,72	0,00	-	125,40	2361,17
Total load building	1593,5	3425,0	0,0	-	2545,6	47931,7