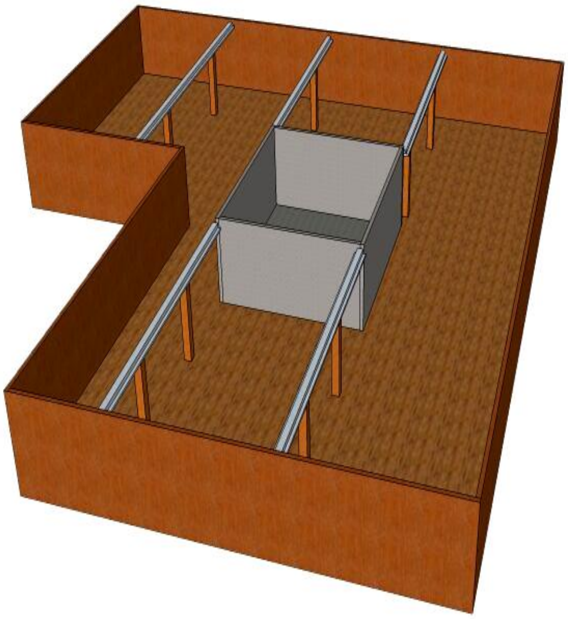

Consequences of Implementing Timber in Medium High-Rise Office Buildings

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

THOMAS ANDERSSON
LINA HAMMARBERG

Department of Civil and Environmental Engineering
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CHALMERS UNIVERSITY OF TECHNOLOGY
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Master's Thesis 2015:129

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Department of Civil and Environmental Engineering
Division of Structural Engineering

Chalmers University of Technology
SE-412 96 Göteborg
Sweden
Telephone: + 46 (0)31-772 1000

Cover:
A mixed system with timber walls, columns and floor elements, steel beams and a concrete core.

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Göteborg, Sweden, 2015

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ABSTRACT

Today many cities have an ambition of densifying the city centre and one solution is to build taller buildings. However, taller buildings usually require more material per unit area than lower buildings. In order to minimise the negative aspect of using more material the engineer can optimise the structure such that less material is needed or utilise less carbon dioxide intensive materials, such as timber.

Even though Sweden has a long tradition of using timber as a structural material, it is often disregarded as an option for tall buildings. Instead more conventional systems of concrete and steel are preferred. However, among clients, engineers and architects there is an increasing interest of utilising timber, but due to the lack of experience in using timber in multi-storey buildings it is hard to estimate what consequences an alternative solution with timber would have on the load bearing system. This makes it hard to advocate the use of timber in a structural system. The aim of this project was therefore to develop possible conceptual solutions for medium high-rise office buildings, where timber is implemented in a mixed structural system. The concepts developed in the project were evaluated with respect to sectional forces and global equilibrium, needed size of load bearing elements, differences in vertical displacements and dynamic response.

The project was carried out in collaboration with the consultant company WSP and Chalmers University of Technology. Literature studies of structural systems for tall buildings, timber buildings, timber components, design with regard to fire safety, dynamic response of buildings and vertical displacements due to creep and shrinkage were performed. A component study was performed where sizes for different materials and members were investigated and presented. In the development of structural systems an existing building in Göteborg was used as a reference building. Two promising concepts, stabilised with a concrete core, were analysed further and compared to the reference building.

A mixed solution proved to be promising provided that timber elements are treated and handled to reduce vertical displacements, the effective span of floor elements is limited and structural connections are designed to permit differential displacements. The mixed structural systems proved to obtain higher accelerations than the reference building.

Key words: Timber buildings, tall buildings, timber, mixed structural systems, conceptual design, vertical displacements, dynamic response, global equilibrium

Konsekvenser av att implementera trä i medelhöga kontorsbyggnader

Examensarbete inom masterprogrammet Structural Engineering and Building Technology

THOMAS ANDERSSON
LINA HAMMARBERG
Institutionen för bygg- och miljöteknik
Avdelningen för Konstruktionsteknik
Chalmers tekniska högskola

SAMMANFATTNING

Många städer har idag en ambition om att förtäta stadskärnan. En sätt att åstakomma förtätning är att bygga högre byggnader. Emellertid kräver höga byggnader mer material per ytenhet jämfört med lägre byggnader. För att minimiera de negativa aspekterna av att använda mer material kan konstruktören optimera stommen så att mindre material behövs eller välja mindre koldioxidintensiva material, så som trä.

Sverige har en lång tradition av träbyggnation, men för höga byggnader utesluts ofta trä som möjligt alternativ. Istället föredras mer konvetionella system av betong och stål. Det finns dock ett ökande intresse hos beställare, konstruktörer och arkitekter av att bygga mer med trä. På grund av bristande erfarenhet av att nyttja trä så är det svårt att uppskatta konsekvenserna av att implementera trä i högre byggnader. Målet med projektet var därför att utveckla möjliga konceptuella lösningar för höga kontorsbyggnader där trä implementerats i en blandad stomme. Koncepten som togs fram i projektet utvärderades med hänsyn till snittkrafter och global jämvikt, erforderliga dimensioner hos de olika lastbärande elementen i stommen, skillnader i vertikala förskjutningar och byggnadens dynamiska respons.

Projektet utfördes i samarbete mellan konsultföretaget WSP och Chalmers tekniska högskola. Litteraturstudier avseende stomsystem för höga byggnader, träbyggnadssystem, träelement, dimensionering mot brand, vertikala förskjutningar på grund av krypning och krympning samt dynamisk respons hos byggnader utfördes. I en komponentstudie undersöktes och presenterades erforderliga dimensioner hos olika komponenter i olika material. Under utvecklingen av stomsystem användes en befintlig kontorsbyggnad i Göteborg som referensbyggnad. Två lovande koncept, båda stabiliserade med en betongkärna, analyserades ytterligare och jämfördes med referensbyggnaden.

En blandad stomme visade sig vara ett lovande alternativ förutsatt att träelement behandlas och hanteras så att vertikala förskjutningar minimeras, spännvidder för golv begränsas och att anslutningar utformas så skillnader i vertikala rörelser möjliggörs. De blandade stommarna visade sig få högre horisontell acceleration jämfört med referensbyggnaden.

Nyckelord: Träbyggnader, höga byggnader, trä, blandade stomsystem, konceptuell design, vertikala förskjutningar, dynamisk respons, global jämvikt

Contents

Preface

In this project the consequences of implementing timber into the structural systems for medium high-rise buildings have been investigated. The project has been carried out in collaboration between the consultant company WSP and the Divison of Structural Engineering at Chalmers University of Technology.

We would like to thank our two supervisors at WSP Construction Design, Peder Bodén and Arnoud Vink for their guidance and support during the project. They have always given time and dedication to our questions. Further on we would like to thank all of the employees at WSP Construction Design in Göteborg. Everyone has welcomed us and contributed to making our time at WSP educational and pleasant.

Furthermore we want to thank Robert Kliger, professor in steel and timber structures at the Division of Structural Engineering at Chalmers University of Technology, for all his inputs regarding timber engineering. He has been an important support for the development of our project.

Finally we would like to give a special thanks to our examiner Björn Engström, professor at the Division of Structural Engineering at Chalmers University of Technology, for his patience and support throughout the project.

Göteborg June 2015

Thomas Andersson, Lina Hammarberg

Notations

Presentation of all variables occurring in the report, listed alphabetically.

Roman upper case letters

A, A_c, A_t	Cross-sectional area of a member, a concrete member and a timber member
E	Elastic modulus
$E_{0,mean}$	Mean value of the elastic modulus for timber
$E_{c,d}$	Mean value of the elastic modulus for concrete
$E_{c,d}$	Final mean value for the elastic modulus for concrete
$E_{t,d}$	Final mean value for the elastic modulus for timber
E_{cm}	Mean value for the elastic modulus for concrete
$EI(z_1)$	Bending stiffness at the height x_1
EI_{ef}	Effective stiffness of a composite section
F_i	Equivalent lateral load at the i:th storey
$F_{v,d}$	Applied shear load on the connectors in a composite floor
$G_k, G_{k,j}$	Characteristic value for the permanent load
H	Height of building
H_u	Equivalent horizontal force from unintended inclination
I	Second moment of inertia
I_{tot}	Second moment of inertia of a timber-concrete composite section
$I_v(z)$	Turbulence intensity of the wind at the height h
L	Length of a member
$M(z_1)$	Bending moment in buildings at the height x_1
$M_{0,Ed}$	Moment for the first order effects from unintended inclinations
M_{Ed}	Design moment that takes the second order effects into account for concrete columns
M_f	Field moment
M_s	Support moment
N_{Ed}	Applied axial design load
$N_{t,Rd}$	Tension capacity of a steel member
N_b	Theoretical buckling load based on a nominal stiffness and buckling length of a concrete column
R	Factor taking the resonance response into account when calculating acceleration of a building due to wind load
T	Averaging time for the mean wind velocity
V	Vertical load from a specific storey when calculating the equivalent horizontal force from unintended inclination

W_i	Weight of the i:th storey
Q	Designing load combination
Q_{fire}	Value of the applied load in fire load case
Q_k	Characteristic value for the variable load
Q_{quasi}	Quasi-permanent load combination

Roman lower case letters

c_f	Force coefficient factor of a building
b	Width of the building
$d_{char,0}$	Charring depth for unprotected timber members during fire
e_0	Intended initial eccentricity, first order
e_c	Eccentricity of the concrete slab in a composite floor section
e_i	Eccentricity due to unintended inclination, first order
e_t	Eccentricity of the timber beam in a composite floor section
$f_{c.c.d}$	Design compression strength for concrete
$f_{c.t.d}$	Design tension strength for concrete
$f_{c.0.d}$	Design compression strength for a timber member, parallel to its grains
$f_{d,fi}$	Load bearing capacity for a timber member subjected to fire
f_k	Characteristic strength of timber
f_{md}, $f_{t,m,d}$	Design bending strength of a timber member, parallel to its grains
$f_{m,y,d}$	Design bending strength of a timber member, parallel to its grains, in y-direction
f_n	Natural frequency of a building
$f_{t,0,d}$, $f_{t,t,d}$	Design tension strength for a timber member, parallel to its grains
$f_{v,d}$, $f_{t,v,d}$	Design shear strength of timber
$f_{v,mean,d}$	Design strength of the connectors in a composite floor
g	Gravitational constant
g_r	Self-weight of the roof
i	Radius of gyration
k_c	Reduction factor of the strength for slender timber columns
$k_{c,y}$	Reduction factor of the strength for slender timber columns, in the y-direction
k_{def}	Deformation factor for timber, creep coefficient
k_{fi}	Modification factor for fire, timber design
$k_{mod,fi}$	Conversion factor for timber, fire design

k_p	Peak factor, used when calculating the along-wind acceleration
l_0	Buckling length
m	Equivalent mass of a building per unit area
n	Number of supporting columns/walls in the structural system vertically loaded
n_{ct}	Ratios between the modulus of elasticity for concrete and timber
$q_m(z)$	Wind velocity pressure at the height h and for a return period of 5 years
s_b	Snow load on the balcony beam
s_r	Snow load on the roof
t	Time a timber member is exposed to fire
u	Elongation/shortening
$u(z_1)$	Lateral deflection of buildings at the height x_1.
u_1	Original moisture content
u_2	New moisture content
u_f	Fibre saturation point for timber materials
u_i	Lateral deflection at the i:th storey
u_{max}	Maximum allowed lateral top deflection of buildings
v	Up-crossing frequency
$\ddot{x}_{\max}(z)$	Along-wind acceleration of a building

Greek upper case letters

$\Delta\alpha$	Shrinkage of timber
ΔL	Shortening/Elongation due to moisture change
$\phi_{1,x}(z)$	Deflecting modal shape of a building
Ψ_0	Combination coefficient for loads combinations
Ψ_1	Combination coefficient for loads combinations
Ψ_2	Combination coefficient for loads combinations

Greek lower case letters

α_{md}	Total unintended inclination angle
α_0	Systematic part of the unintended inclination angle
α_d	Random part of the unintended inclination angle
α_f	Maximum shrinkage in a certain direction
β	Factor that depends on the moment distribution from the first and second order effect, used when calculating the design moment for concrete columns
β_0	Design charring rate

γ	Effectiveness of the connection in a composite floor
$\gamma_{M,fi}$	Partial factor for fire in wood
$\gamma_{G,j}$	Partial safety factor for permanent load
γ_Q	Partial safety factor for variable load
ε	Strain
λ	Slenderness of a column
ξ_j	Reduction factor of the permanent load for specific load combination
σ	Stress
$\sigma_{c,c}$	Applied compression stress on a concrete member
$\sigma_{t,c}$	Applied tension stress on a concrete member
$\sigma_{c,t}$	Applied compression stress on a timber member
$\sigma_{c,0,d}$	Applied compression stress on a timber member, parallel to its grains
$\sigma_{t,t}$	Applied tension stress on a timber member
σ_{md}, σ_{tm}	Applied bending moment on a timber member
$\sigma_{m,y,d}$	Applied bending moment on a timber member, in y-direction
$\sigma_{t,0,d}$	Applied tension stresses parallel to a timber members grains
$\sigma_{t,v,d}$	Applied shear stress on a timber member
$\sigma_{\ddot{x}}(z)$	Standard deviation of the along-wind acceleration
τ_d	Maximum shear stress in a timber beam
$\varphi, \varphi(t, t_0)$	Creep coefficient for concrete
ψ_0	Combination coefficient for variable loads in office buildings, ULS
ψ_1	Combination coefficient for variable loads in office buildings, load case fire

1 Introduction

In this chapter the background, problem description, aim, objectives, limitations and methodology of the project are presented.

1.1 Background

Since the industrial revolution people have been moving from the countryside to cities. As more people move into cities the need for densification increases. This is also the case for Göteborg where the municipality has an ambition to densify the city centre. As a result of this ambition, some new tall buildings have been built lately. A building can be considered as tall when it is higher than the normal height of the surrounding buildings; in Göteborg this is 8-10 storeys (Samuelsson, et al., 2012). Examples of tall buildings are Skanska's office building Gröna Skrapan, Ullevi Office where WSP are located and ÅF's new office building. There are also plans for a 17-storey high apartment building at Johanneberg in Göteborg, (Skanska, n.d). At Lindholmen there are plans of building Göteborg's first sky scraper, Karlavagnstornet, which will reach at least 230 metres.

Densification of the city is related to a higher usage of public transportation and cycling. Together with the possibility to utilise central energy services, densification can be considered as a sustainable development of the city (SOM, 2013). In addition, high buildings tend to create densification of people in the streets, which is beneficial for commerce, public services and entertainments (Samuelsson, et al., 2012). However, there are also negative aspects concerning sustainability. For example tall buildings demand more material per unit area. According to SOM (2013) the engineer has two choices regarding the structural system;

"First, the engineer can try to design a building which minimizes structural material [...]. Secondly, the engineer can design a building which uses less carbon intensive materials such as timber."

In Sweden there have been a long tradition of using timber as a construction material and today 90 % of the single-family houses are built with timber. However, in total, half of all housings in Sweden are built with a timber system (Naturvetarna, 2013). Despite of the strong tradition of using timber and Sweden's good supplies of timber, multi-storey buildings in timber are not very common. This is partly due to the restriction of building timber houses with more than two floors. The restriction lasted for over a decade until it was abolished in 1994 (Svenskt trä, 2014:b). One way to increase the use of timber in high buildings might be to combine timber with steel and concrete in mixed structural systems.

According to Svenskt trä (2014a) the benefits of using timber as a construction material are numerous. It is accessible, resistant and strong in relation to its density. Furthermore, timber is generally considered as renewable, sustainable, environmentally friendly and climate smart.

1.2 Problem description

In Sweden, in spite of its benefits, only 10 % of the multi-storey buildings utilises timber elements as part of the structural system (Ekenberg, 2013). Instead more conventional structural systems with concrete and steel are preferred. However, among clients, engineers and architects there is an increasing interest of utilising timber.

Due to lack of experience in using timber in multi-storey buildings it is hard to estimate what consequences an alternative with timber would have on the load bearing system. This makes it hard to advocate the use of timber in a structural system; hence timber is often excluded as an option to the advantage for concrete and steel. Therefore the possibility of implementing timber into mixed structural system for tall buildings was investigated in this project.

1.3 Aim and objectives

The aim of the project was to develop possible mixed structural systems for medium high rise office buildings where timber is implemented in the structural system. Furthermore the consequences of implementing timber should be evaluated. Finally the project should give recommendations concerning where timber is best suited in such structural systems.

The conceptual solutions with timber should be evaluated with respect to:

- needed size of the load bearing elements.
- sectional forces and global equilibrium.
- differences in vertical deformations of different materials.
- the dynamic response.

1.4 Limitations

The main focus should be to develop structural systems suitable for office buildings in the range of 15 storeys. The effect of implementing timber into other types of buildings such as residential buildings, schools and hospitals was only to be treated briefly.

The arguments for and against the usage of a certain timber members were based on their required dimensions and their structural performance. No investigation should be made regarding the environmental and economic consequences of choosing timber to the favour of another material. In addition the construction phase of the building was not to be investigated. However, proposed structural systems should be possible to construct.

For the structural system preliminary calculations were to be performed to ensure the plausibility of the system. The aim should be to develop a conceptual solution, not a final solution. No calculations were to be made regarding the foundation of the building. Moreover, architectural aspects and the effect of implementing timber into non-structural members, such as façades and non-load bearing inner walls, were not to be treated in this project.

1.5 Methodology

The project contained a literature study and three sub studies which are a component study, development of structural systems and a more detailed analysis of the developed systems. Choices of sub studies have been made together with supervisors from WSP and from Chalmers.

Literature studies were to be performed in order to increase the knowledge regarding structural systems for tall buildings, timber buildings and structural timber members. In addition the literature studies should also include studies of composite floors of timber and concrete, fire safety, acoustics, vertical displacements and dynamic response of buildings. The literature study should be the basis for the investigation of needed sizes, the development of structural systems and when analysing the consequences of implementing timber in structural systems. As a complement to the literature studies consultation with experienced engineers at WSP and Chalmers were to be performed.

A component study was to be performed, aiming to present needed sizes of members for different materials. The component study should be made as general as possible by investigating dimensions for different load cases. This study should also provide an additional basis for the development of structural systems.

When developing structural systems some demands from a reference building should be used in order to narrow the number of concepts. Lyckholms, a 14-storey office building, in Göteborg was to be used as the reference building. This building has a relatively simple geometry and is stabilised by a concrete core. By using a reference building as benchmark the developed concepts should be more realistic and enable a comparison with the original structural system of the reference building.

The developed systems should be analysed and compared to each other qualitatively. The solutions regarded as the best ones were to be investigated further in order to understand the consequences of implementing timber with respect to other aspects than the size of the members. Main focus should be to investigate differences between the concepts and the reference building and not between the different concepts.

2 Timber in Tall Mixed Structural Systems

In this chapter arguments for why tall buildings with mixed structural systems are of interest is presented. Firstly some of the benefits of higher buildings are treated followed by a section where mixed structures are defined and their benefits are presented. Thereafter advantages and disadvantages of using timber in the structural system are presented.

2.1 Densification of cities

Today the global urban population is growing compared to the rural population and more than 50 % of the population is living in urban areas (WHO, 2015). Sweden has a higher amount of its population living in cities than the global average. In 2050 the expected percentage of the urban population in Sweden is around 90% (WHO, 2014a). Today, the three biggest cities in Sweden have a population of 1.7 million people together. In 2053 this population is expected to have grown to 2.4 million people, an increase with 37% (Karlsson, 2015).

A problem is that the growing cities will create a need for more area. Therefore many municipalities have as a goal to densify their cities. Densification enables more land for agriculture use and reduces the need of transportation (Moström, 2013). However, the population in cities still needs to have access to recreational areas such as parks and nature. They also need a certain amount of private space. So, when densifying a city, the process and design of the densification need to be thoroughly planned, according to Skovbro (2002). There are different ways of densifying a city. It can be performed by placing buildings closer to each other, exploit uninhabited areas within the city or making apartments smaller. Each of these solutions is not always comprehensive with the above mentioned aspects that need to be taken into account when densifying a city. Therefore, in a city with limited parks and already small apartments, one solution is to build taller buildings.

2.2 Definition of tall buildings and mixed structures

A building is commonly defined as high if it has at least ten storeys or if it raises a height of 30 metres. However, whether a building is considered high or not depends of the context in which the building is located. As can be seen in Figure 1a, a building with a certain height can be considered as high, if it is built in a city with where it is taller than the urban norm. Nevertheless, in a high-rise city the same building is considered to be low. In addition, the appearance of a tall building also depends on its slenderness. A slender building is often experienced as higher than a stocky building, see Figure 1b. The definition of a tall building also depends on whether the height of the building influences the design and planning. For example a tall building experience higher lateral load due to increasing wind load and therefore bracing is a product of the tallness (CTBUH, n.d).

Figure 1 How the appearance of a building depends on its slenderness and context. (a) Height relative to context (b) height relative to proportion. Inspiration is taken from CTBUH (2015).

According to Elliot, et al. (2002) published by the International Federation for Structural Concrete (fib) a mixed structure can be defined as *"the use of different materials and design approaches so that the whole is greater than the sum of its parts, e.g. precast concrete façade used to stabilise a structural steel frame"*. In this report a mixed structure is defined as a structural system that consists of components of different materials. Elliot, et al. (2002) also states that today 50% of all new multi-storey buildings in the western world have mixed structures and according to Vambersky (2004) the most common mixed structures contain concrete and steel.

In Sweden, one of the tallest timber buildings, Limnologen in Växjö, has a structural system that can be considered as a mixed structure. The first storey is made of concrete to increase the self-weight and thereby improve the stability of the building, see Section 2.4.2.

Terms like hybrid structures can also be used as a synonym to mixed structures. However, hybrid systems can also refer to the usage of more than one bracing system for lateral forces. It is also important to distinguish between mixed structures and composite structural members. In a composite member, materials are combined to interact in the sectional response. In mixed structures, units made of different materials are combined. Also composite members can be included.

According to Vambersky (2004) mixed structures are in many cases the solution needed to meet the demands from architects, reduce floor depths, to create structures that are sustainable and to enable a rapid construction. Further on, mixed structures are more or less by their definition cost-effective, because materials are used where they are best suited.

Today, steel and concrete are mainly used in mixed structures. In this project the possibility of utilising timber was investigated. According to Hein (2014) timber is best used in mixed structures. It is important to acknowledge the weaknesses of timber to be able to optimise the structure. By using a mixed structure with timber, the total amount of material may be reduced in comparison with a timber building. This is mostly the case for taller timber buildings. Hein argues that the best solution regarding cost-efficiency is to use a stabilising concrete core in a mixed structure with timber.

2.3 Timber in structural systems

Today 70% of Sweden's land area is covered by forest and the forest areal is increasing since the growth of the forest is higher than the felling. In 2011 Sweden was the second largest exporter of sawn timber, pulp and paper in the world (Föreningen Sveriges Skogsindustrier, 2013).

For small houses timber is often regarded as the best structural material. Wood is a high performance material with high load bearing capacity in relation to its weight. It is also highly available, easy to handle and possess good thermal properties. Due to its many benefits, timber has become more and more competitive even for taller buildings (Föreningen Sveriges Skogsindustrier, 2013).

Despite its many benefits timber is not used as a structural member for more than 10 % of the multi-storey buildings in Sweden today (Ekenberg, 2013). In Roos, et al. (2009) a qualitative study based on interviews with engineers and architects in order to map their attitude to using timber in structural systems is made. They mean that the two main reasons for not using timber is lack of education in the field and that there is a strong tradition of building with concrete and steel. Engineers and architects believe that they do not have sufficient knowledge of timber for advocating it for the client or the contractor (Roos, et al., 2009).

According to Ekenberg (2013) another reason for the absence of multi-storey buildings in timber is that production capacity is too small. He states that there are too few market actors with knowledge in timber construction and that the prices are high. However, there is an increasing interest from clients of building with timber.

2.3.1 Environmental benefits

Today 30 % of the annual greenhouse emissions in the world can be attributed to the building sector, as well as 40 % of the energy used in the world (UNEP, 2009). Figure 2 shows that the energy used during the manufacturing of materials for a building is about 22 % of the total energy used during the life cycle of the building (CEI-Bois, 2010). As stated in Section 1.1 the engineer has two choices regarding the structural system; minimize the structural material and/or use less carbon intensive materials such as timber.

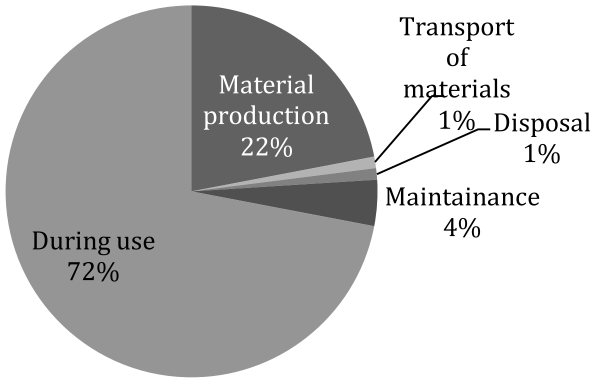


Figure 2 *Distribution of the energy consumption for a building. Inspired by CEI-Bois (2010).*

If considering carbon dioxide emissions timber is a good substitute to non-renewable materials such as concrete, steel and masonry. When manufacturing non-renewable building materials both the extraction and processing demand energy. These building materials give a positive carbon footprint, while timber gives a negative carbon footprint. This is since the emissions due to felling, transportation and processing are small compared to the amount of carbon the timber product itself can store (Föreningen Sveriges Skogsindustrier, 2013).

Figure 3 illustrates the emissions of carbon dioxide during the manufacturing of different building materials. The diagram does not account for storing of carbon in timber products (Föreningen Sveriges Skogsindustrier, 2013).

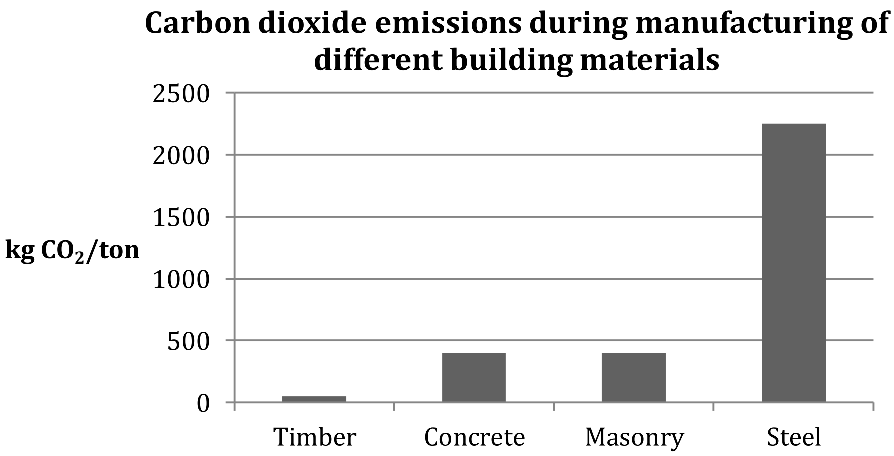


Figure 3 *Approximate values for the carbon dioxide emissions during manufacturing of different building materials Inspired by Föreningen Sveriges Skogsindustrier (2013).*

The Technical Research Institute of Sweden (SP) has investigated the environmental benefits of using timber in multi-storey timber buildings by performing a life cycle assessment of different kinds of building systems with the same functions. From the results it could be seen that the carbon emission during the construction phase was 60 % higher for a concrete building than for a timber building. This is since timber members have a lower material weight, which means that they require less energy for construction, foundation work and transportation (Diego, et al., 2013). However, it is important to be aware of that the construction phase is contributing to 22 % of the total energy consumption of a building.

Timber products can easily be recycled after they have fulfilled their purpose in buildings. For example it can be used for manufacturing of fibre boards. In the final stage products made of timber can be used as biomass fuel. From an environmental point of view this is an important advantage, since biomass fuel helps replacing fossil fuels (Föreningen Sveriges Skogsindustrier, 2013).

2.3.2 Economic and social benefits

Due to its low self-weight timber is cost efficient when it comes to transportation (Östman & Gustafsson, 2009). Of the same reason the construction of buildings can be faster and thereby more cost efficient. The low weight is also beneficial for the foundation work.

Since multi-storey timber buildings often are constructed with prefabricated members, the environment around the construction site is less affected. The noise level at the construction site is often low and since the construction time is short problems with traffic is limited to a shorter period. This is preferable when building within cities (Swedish wood, 2012).

In addition, the properties of timber remain unchanged in freezing temperatures and the construction of timber buildings is thereby possible during winter (Östman & Gustafsson, 2009). Moreover, wood has good thermal insulation properties, which reduce the need of insulation (CEI-Bois, 2010).

2.3.3 Difficulties with timber as a structural member

Timber is today often regarded as a good substitute to concrete and steel due to its environmental benefits. It is true that timber has less climate impact than for example concrete, but still timber is also combined with some difficulties that are important to remember.

When choosing timber for main structural members there are some main difficulties that need attention. It can be hard to obtain satisfying performance regarding impact sound insulation in floors. Impact sound arises from footsteps on the floors. These vibrations have low frequency and therefore are hard to insulate.

Vibrations are limited by increasing the stiffness of the floor. Since timber floors do not provide the same stiffness as concrete floors, timber floors tend to become high for long spans which also affect the total height of the building. This is an important issue to handle for buildings that demand open floor plans, for example office building in contrast to apartment buildings. For more information about acoustics and vibrations the reader is referred to Section 4.7.

Further on, it is important to consider the global equilibrium of timber buildings. The density of timber is less than the density of concrete; therefore timber buildings are lighter. This can be problematic when designing tall timber buildings, since a light structure cannot resist wind loads as good as a heavy building. In many cases it is therefore necessary to provide such light structures with extra weight or with anchorage in order to prevent lifting and tilting of the building.

Timber has a high strength in relation to its weight. However, the actual compressive strength is less than the compressive strength of concrete or steel. The tensile strength parallel to the grains are greater than for concrete but smaller than for steel. It is usually necessary to design timber members with larger dimensions than corresponding members in concrete or steel, which is shown in Chapter 5. Therefore, by choosing timber members the total building height, the available area on each floor and the impression of the building might be affected.

In cities it is common with regulations regarding how high buildings are allowed to be in different areas within the city. If choosing timber there is a risk of losing one storey due to larger height of the floor structure (Kliger, 2015-02-06). This would affect the rentable area of the building.

Timber has different mechanical properties in different directions. For example, timber has higher compression strength parallel to its fibres than perpendicular to its fibres. Consequently, larger global vertical deformation arises if there are a lot of members subjected to compression perpendicular to the grains throughout the building.

When, for example, mixing a concrete core with a timber frame, differences in vertical movements between the different systems are important to consider. Shrinkage strain for timber is, as the strength, depending on the direction relative to the grains and on the change in moisture content. Typical values for the timber shrinkage strain for each change of 1 % of the moisture content are 0.001 longitudinally and 0.03 tangentially to the fibres (Crocetti, et al., 2011).The shrinkage is in other words mainly effecting the transverse movements, but for long members the longitudinal movements can be important to take into account. A common change in the moisture content can be in the order of 7 %, giving for example a final shrinkage strain of 0.007 longitudinally to the grains. According to Al-Emrani, et al. (2011) concrete has a final shrinkage strain in the range of 0.0001-0.0005 which is smaller than the final shrinkage strain for timber.

Another phenomenon that is affecting the deformations is creep, hence the stiffness of the material is an influencing factor for deformations. Both timber and concrete creep when subjected to long term load. Timber is creeping more in a moister environment or higher temperature. In Eurocode 5 the creep coefficient is a factor named k_{def}, the deformation factor. Common values for this factor are 0.6-0.8, but for the highest service class it has a value of 2. This factor is used to reduce the elastic modulus. The corresponding creep coefficient for concrete is usually in the range of 1 to 3, larger than the creep coefficient for timber (Al-Emrani, et al., 2011).

2.4 Existing and planned tall timber buildings

In this section some existing timber buildings and some visions for future buildings are presented. Even though the main structural material of the buildings is timber, most of the high multi-storey timber buildings have structural components in concrete or steel making the buildings mixed according to the definition used in this report.

2.4.1 Former timber warehouse in Eslöv

The tallest timber building in Sweden is located in Eslöv and reaches 31 metres. It was built in 1918 and initially it was a warehouse for cereal grains, but in 1984 the activity ended. There was no clear plan for further use of the warehouse and after many uncertainties the final suggestion was to reconstruct it into an apartment building. The structural system is composed of massive timber columns and floors. Originally all storeys had timber in the structural system but due to water damage the three top floors were replaced by a steel frame (Salomon-Sörensen & Blomqvist, 2011).

2.4.2 Limnologen in Växjö

Another, more recently constructed, apartment building is Limnologen in Växjö with its 8 floors in total, see Figure 4a. The first floor is made of concrete to give the structure more weight and simplify the anchorage of the light-weight timber floors above. Vertical load are carried by the outer cross-laminated walls and some of the inner walls. In some places columns and beams have been used to reduce long spans. The building is stabilised by the exterior cross-laminated walls by in plane action. The floors distribute the horizontal loads to the walls by diaphragm action. From the top floor down to the first floor there are 48 tension rods going inside of the inner walls in order to anchor the building with regard to tilting (Stehn, et al., 2008).

(a) (b)

Figure 4 Limnologen, (a) photo of the building (Albrecht, 2009)(b) how the floor is placed between two wall elements.

One consequence in Limnologen is considerable vertical deformations in the load bearing CLT-walls. In Limnologen the floors are supported by the CLT-walls as shown in Figure 4b. This makes the floor loaded in compression perpendicular to the grain. Vertical deformations arising from this are great in proportion to the total vertical deformations. Approximately 25 % of the deformations come from deformations in the floor and the rest in the wall elements (Engquist, et al., 2014).

2.4.3 Parking garage in Skellefteå

Not only residential buildings use timber as structural members. In 2009 a garage in Skellefteå was constructed. The garage consists of four timber floors above the ground and two basement floors in concrete. All the columns and beams are made of glulam and the load bearing slabs are composed of cross laminated timber, solid boards that are glued together crosswise (Martinsons, 2009).

2.4.4 Statdhaus in London

With eight floors entirely built in timber and one floor with reinforced concrete, the Stadthaus in London is the tallest residential building in the world today. The structural system is made of walls and floors constructed with prefabricated cross-laminated timber panels. In order to facilitate openings and removal of internal walls, no beams or columns were used in the structure. One of the aims of the project was to create a sustainable building. This motivated the choice of using cross-laminated timber, since it provides good insulation, lowers the energy use and is easy to demolish and reuse (TRADA Technology, 2009).

2.4.5 Treet in Bergen

In Bergen, Norway, a 14 floors and 51 metres high building for apartments is under construction and will be finished during the autumn 2015, see Figure 5. The entire load bearing system will be in timber (Sweco, n.d). Glulam trusses are used along the façade in a framework to stabilise the building against lateral loads. CLT walls, which do not contribute to the stability or main load bearing system, are used for the elevator shaft and some internal walls (Abrahamsen & Malo, 2014).

Prefabricated modules of timber framework are stacked on each other in sections of four and one modules. The first four modules are not connected to the surrounding load bearing structure. Every fifth floor is strengthened by a glulam truss and the modules at this storey are connected to the truss and are not supported by modules below. This strengthened storey carries a prefabricated concrete slab which the following next 4 storeys are connected to. These storeys are not connected to the external framework in any other way than through the concrete slab. This pattern continues for the rest of the building, see Figure 5.

The truss together with the framework and the extra weight from the concrete slabs provides the building with sufficient rigidity and good dynamic properties. Tension forces are transferred down to the foundation by the external glulam framework. The typical dimensions of the columns are 405x650 mm and 495x495 mm. A common dimension for the diagonals, which only works in tension, is 405x405 mm. These columns and diagonals form the external framework which is mostly covered by glass or metal sheeting. These covers protected the framework from rain and sun which resulted in a climate class 1 for most of the members (Abrahamsen & Malo, 2014).

Figure 5 Model of the timber building Treet in Bergen, Norway. (Abrahamsen & Malo, 2014).

2.4.6 Plans and visions for future timber buildings

Outside of Stockholm there are plans of building higher than 14 floors. This building that would have 22 floors and rise 65 metres can be a reality in 2018. It is an apartment building designed by Wingårdh Arkitekter (Wachenfeldt, 2014).

One of Sweden's largest housing associations (HSB) wants to build a spectacular apartment building for their 100 year celebration in 2023. The winning concept in the architectural competition is a 34 storey high building with timber as main structural member and concrete for the stabilising stairwell and elevator shaft (C.F. Møller, n.d).

3 Structural Systems and Timber Components

In this chapter the common structural systems for high-rise buildings are presented followed by a description of common structural systems for timber buildings. Thereafter structural components in timber are presented.

3.1 Loads effects in tall buildings

The loads acting on high-rise structures are vertical loads such as permanent loads, imposed loads and snow loads and lateral loads such as wind loads. Other effects that need to be accounted for in the design of structural systems are unintended inclination, accidental actions. The effect from earthquakes is usually not considered in Sweden.

3.1.1 Vertical loads

The components resisting vertical loads are columns, cores and load bearing walls. These components need to be designed for different load combinations since the vertical load accumulates from the roof level down to the foundation. Hence, the total vertical load acting on the foundation is the sum of the loads from all the floors above. The dead weight of a structure can be decreased by using steel and timber instead of concrete. However, the weight is also an advantage since it helps in resisting against overturning (Ching, et al., 2009).

If walls and columns are aligned through the whole building the most efficient load path can be utilised. Deviation from a straight load path results in a redirection of the load horizontally.
Figure 6 shows some examples of how the vertical load can be carried through the structure of high-rise buildings (Ching, et al., 2009).

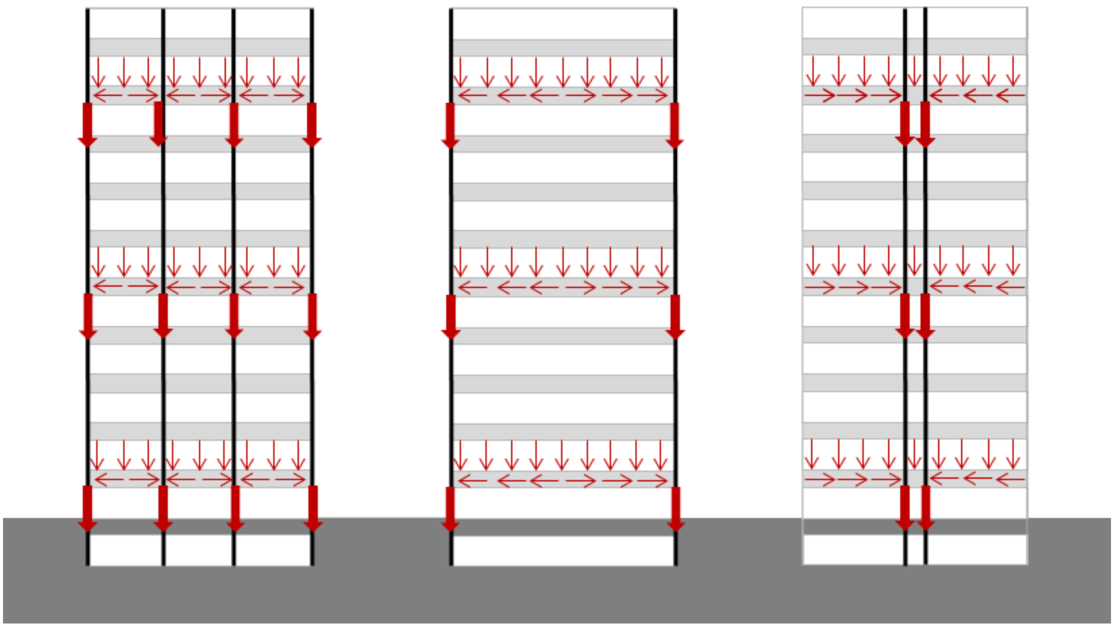


Figure 6 Principle sketch over straight vertical load paths in structural systems.

3.1.2 Lateral loads

With increasing height the lateral loads become more important for the design. In areas where the seismic activity is low, wind is the load that affects the design of high-rise buildings the most. In this report, the consequences of earthquakes are not considered. The magnitude of the wind pressure acting on a structure increases with increasing height. In design the wind loads are assumed to act perpendicular to the vertical loads.

The wind load will induce shear forces and moments in the load bearing members of the structure. If the members are not able to resist these load effects locally and globally the structure can tilt or slide. High and slender buildings are more prone to tilt than stocky buildings. Sliding can be a problem if structures are not able to resist the induced shear forces between the building and the foundation. These shear forces can move the whole building laterally if the shear resistance at the foundation is insufficient. When considering tilting and sliding of structures the self-weight of buildings is favourable. This is since the self-weight counteracts the moment and shear forces. If the self-weight is too low the moment and shear forces can be resisted by anchoring the structure to the foundation.

For tall buildings there are two types of lateral deformations that need to be considered in design; shear deformations and flexural deformations. The total impact of each deformation mode depends on the slenderness of the building. A slender building is more influenced by flexural deformations and a stocky building is more influenced by shear deformations, see Figure 7. For buildings with a height-to-width ratio greater than five, the deformations due to shear can be neglected (Neuenhofer, 2006). However, it is important to be aware of that this simplification is not justified for rigid frame systems, since they mostly respond to lateral load by shear deformation regardless of the slenderness.

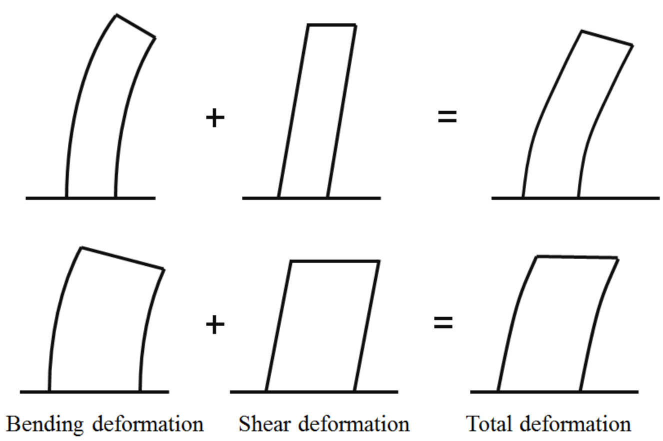


Figure 7 Deformations due to bending and shear for a slender and a stocky building.

Wind will also induce a torsional moment in the structure which is resisted by the torsional stiffness of the stabilising members. The torsional stiffness depends on the geometry and the size of the stabilising members, and is higher if the mass centre and centre of rigidity do coincide. A structure can be provided with additional torsional stiffness by bracing units. The torsional stiffness of a structure becomes more important with increasing height (Ching, et al., 2009).

When the wind load is considered as a static load the deflections of a high-rise building can be modelled as a cantilever beam. The taller and more slender the building is, the larger the horizontal displacement becomes. Wind gusts cause dynamic effects, which contributes to additional displacements. This action can result in oscillation, which causes vibration of the building. These vibrations can be experienced as uncomfortable for people living or working in the building (Ching, et al., 2009). The dynamic effect from wind load is described further in Section 7.5.

3.1.3 Load effects from unintended inclination

A building is never perfectly straight due to unintended inclination of vertical members. The global unintended inclination is the sum of the unintended inclination from all the vertical members throughout the building. The probability that all members incline at the same direction is small. Therefore, the more members, the less global unintended inclination is expected. Equation (1) can be used to estimate the angle of the inclination for the global tilting.

$$\alpha_{md} = \alpha_0 + \frac{\alpha_d}{\sqrt{n}} \tag{1}$$

α_{md} Total inclination angle
α_0 Systematic part of the inclination angle
α_d Random part of the inclination angle
n Number of supporting columns/walls in the system loaded with vertical loads.

The load effect is determined by changing the inclined building to a perfectly straight building by adding additional equivalent horizontal forces to account for the effect from the unintended inclination. When the inclination angle is known the equivalent forces can be calculated with Equation (2).

$$H_u = V \cdot n \cdot \alpha_{md} \tag{2}$$

H_u Equivalent horizontal force for a specific floor
V Vertical force from the specific floor
α_{md} Total inclination angle

3.2 Structural systems for tall buildings

In the design of structural systems for tall buildings it is important to acknowledge the lateral loads. The taller a building is the more effect from e.g. wind loads is induced in the building. Table 1 show the most common structural systems used for tall buildings and for which heights they are assumed to be efficient. It is important to remember that these heights are for buildings stabilised made of concrete and/or steel. The efficient height, if using timber or a combination where timber has an important role, is not known. If stabilising with only timber members a normal height today is 8-10 storeys. In Bergen however, a timber building of 14 storeys is being built, see Section 2.4.5.

Table 1 Efficient height for different common structural systems (Ali & Sun Moon, 2007)

Structural system	**Efficient height**
Shear wall and core braced structures	Up to 35 storeys
Rigid frame structures (non-braced)	Up to 30 storeys
Braced-frame structures	Up to 80 storeys
Tube structures	Up to 110 storeys
Core structures with outriggers	Up to 150 storeys

3.2.1 Systems with shear walls, coupled walls and cores

Walls that resist lateral loads are referred to as shear walls. They are mostly made of reinforced concrete, but can also consist of masonry or timber (Cook, 2005). Reinforced concrete has high stiffness compared to timber, which is an advantage when designing tall buildings. Examples of timber shear walls are stud walls or massive timber walls, see Section 3.4.3. Lateral loads are transferred by diaphragm action of floors to the walls. Concerning diaphragm action distinction can be made between flexural and rigid actions. The flexural action distributes the horizontal load with respect to tributary area, while the rigid action distributes the load with respect to stiffness and location of the walls (Ching, et al., 2009).

When loaded horizontally walls need to resist an applied moment through a force couple. The stress distribution depends both on the applied moment and the vertical loads, hence the stress distribution can include both tension and compression or just one of them (Ching, et al., 2009). Examples of stress distributions are shown in Figure 8. Shear walls are connected to the foundation slab and usually goes all the way up to the top floor, making the walls very slender. Therefore, shear walls mainly act in bending when resisting lateral load, as a cantilever beam. Because of the walls high moment of inertia in their own directions, they provide high stability through bending stiffness. Shear deformation is almost negligible (Stafford Smith & Coull, 1991).

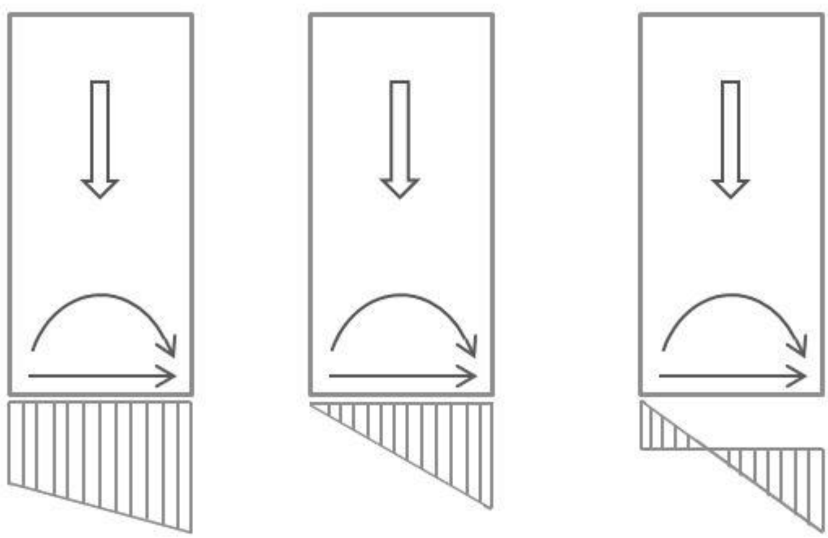


Figure 8 Examples of stress distributions in horizontally loaded walls

As previously described a wall resists bending moments from lateral loads with a force couple. When using concrete, which has low tensile strength, the need of reinforcement can be high. One way to reduce the amount of reinforcement is to place the walls such that they carry as much permanent load as possible. This will reduce the tensile stresses in the wall and hence also the amount of reinforcement that is needed (Eisele & Kloft, 2002).

Shear walls can be placed in different arrangements creating different layouts (Ching, et al., 2009):

- Separately
- Connected in angles (cores)
- Coupled by floors or beams

Separate walls need to be arranged in a certain layout to provide stability. At least there have to be three walls intersecting in two different points, see Figure 9. The walls can also be connected to each other forming composed sections. The same as for separate walls applies, which means at least three wall units intersecting in two points (Eisele & Kloft, 2002).

When walls are connected and coupled together to form one unit the joints need to have sufficient rigidity, so that the connected walls act as one unit. The degree of interaction between wall units can vary from acting as one unit to act as separate walls, depending on the rigidity of the coupling.

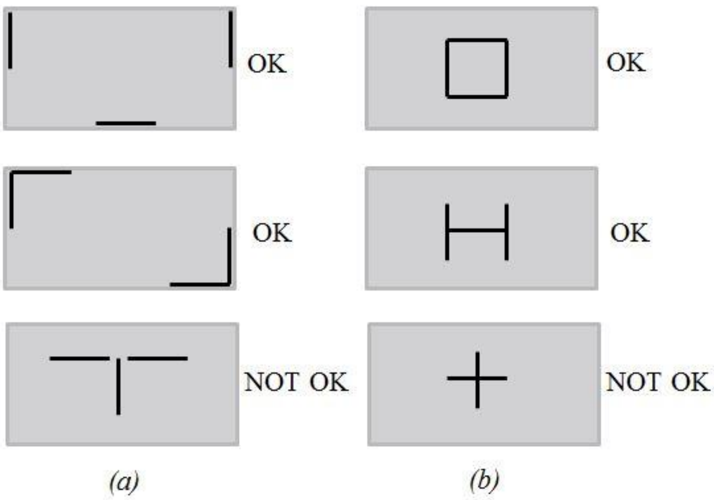


Figure 9 Examples of shear wall arrangements, (a) single walls and (b) composed sections.

Composed cores of interacting walls have higher flexural and torsional stiffness than a corresponding core without interaction. The flexural and torsional stiffness depends also on the rigidity of the connections. The more openings a shear wall has, the more it will act like a frame (Ching, et al., 2009). It is therefore suitable to utilise cores that enclose elevator shafts, stairwells or machinery rooms. This is also good according to fire safety due to the requirement of withstanding fire a certain period of time for stairwells (Eisele & Kloft, 2002).

Shear walls placed parallel to each other and coupled to each other by floors are called coupled shear walls. The provided stiffness from coupled shear walls is higher than the sum of the uncoupled walls, due to the imposed interaction, which makes the walls act more or less like one unit.

3.2.2 Rigid frame structures

Rigid frames consist of beams and columns with moment resisting connections. To achieve frame action the connections must have enough stiffness to keep the angles between the members constant during load increase (Cook, 2005).

The appearance of rigid-frame systems is often similar to column-and-beam systems, but the structural behaviour differs a lot. As a result of the moment resisting connections in a frame structure, lateral loads can be resisted without bracing. Column-and-beam systems, oppositely, need bracing units that resist the lateral loads (Shodek & Bechthold, 2008).

Vertical loads acting on a rigid frame are first resisted by the beams that are supported by the columns. The load is further on resisted by the columns and finally by the foundation. The loading on the beams create a need for end rotations, but since the ends are rigidly connected to columns, the rotation cannot occur freely. Due to this restraint, beams in a frame structure almost behave like fixed ended instead of simply supported, see Figure 10. This is beneficial, since fixed beams are more rigid, which leads to less deflection and less bending moments compared to corresponding simply supported beams. Thereby the beams in a frame structure require smaller dimensions. However, due to the restraint, the columns need to take both bending moments and axial load, leading to larger dimensions of the columns (Shodek & Bechthold, 2008).

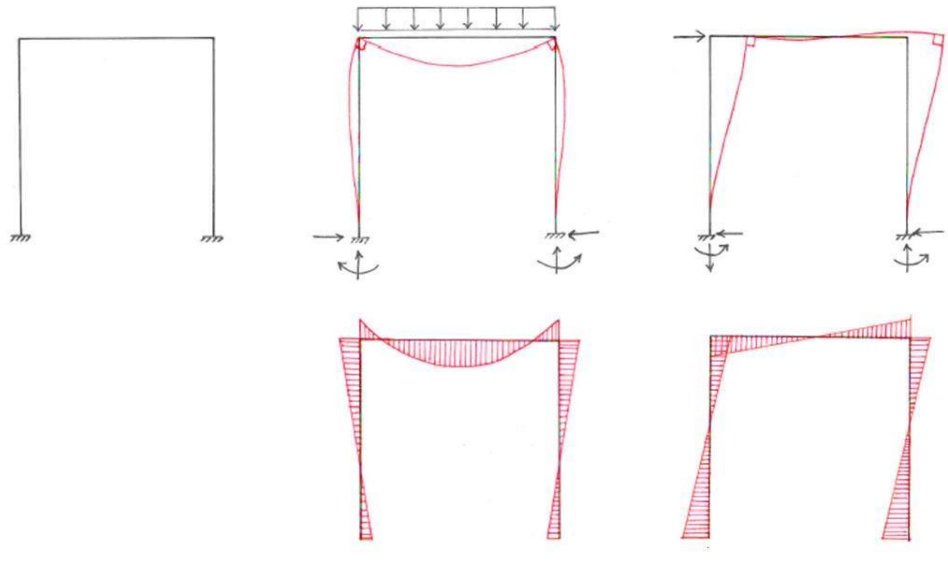


Figure 10 A loaded frame and the corresponding reaction forces, moment distribution and shear force distribution.

A consequence of using rigid frame systems is that the foundation needs to carry an additional lateral force, since the vertical loads do not only create vertical reaction forces at the ground supports, but also horizontal reaction forces, see Figure 10. Due to the frame action the vertical load pushes the columns outwards. This movement is prevented by fixing the frame to the ground and thereby creating a restraint (Shodek & Bechthold, 2008).

Rigid-frame structures can resist horizontal loads provided that their connections are properly designed; hence they need to be rigid. The more stiff the beam is, the more horizontal load the structure can withstand (Shodek & Bechthold, 2008). Rigid frame systems mostly respond to lateral loads in a shear mode regardless of the slenderness. It is therefore important to consider both the effects of shear deformations and flexural deformations when analysing rigid-frame systems (Neuenhofer, 2006).

The economic height limit of a rigid frame system is about 30 storeys. Nonetheless, taller rigid frame buildings can be designed, but since moment resisting connections are complicated to design, such structures tend to be more expensive when the lateral load increases (Stafford Smith & Coull, 1991). Only utilising frame action to resist lateral loads is often inefficient and therefore rigid frame systems often complemented with bracings (Shodek & Bechthold, 2008). From an architectural point of view framed-structures are favourable, since they provide the design with minimal obstruction in the layout.

3.2.3 Braced-frame structures

A braced-frame structure consists of a beam-column system and additional bracing units. In Figure 11 an example of a braced frame structure and its components are showed. In a rigid-frame system, the lateral load creates shear in the beams and bending in the columns. With increasing height of the structure, the bending moment in the columns become higher, leading to large dimensions (Eisele & Kloft, 2002). An efficient way to improve the resistance to lateral loads and thereby reduce the amount of material in the structure is to add additional diagonal bracings. By doing so, the bracing diagonals resist the lateral load in axial action, which is more efficient than resisting load in bending or shear. A braced-frame structure can thereby resist the same loads with less material (Cook, 2005). An economic benefit of using braced-frame structures instead of rigid-frame structures is that the connections are easier to manufacture. Instead of moment resisting connections, pin joints can connect the members in braced-frame structures (Ching, et al., 2009).

When subjected to lateral loads, braced-frame structures behave like a beam with webs and flanges. The beams and diagonals resist the shear and thereby resemble the web, while the columns acts like flanges, resisting the moment by a force couple. As for rigid-frame structures, both shear and flexural deformations need to be considered in design. Concerning bracing units a distinction can be made between centric and eccentric bracings.

Braced-frame systems are used for both low-rise buildings and high-rise buildings. Pure braced-frame structures can reach up to 80 storeys (Ching, et al., 2009).

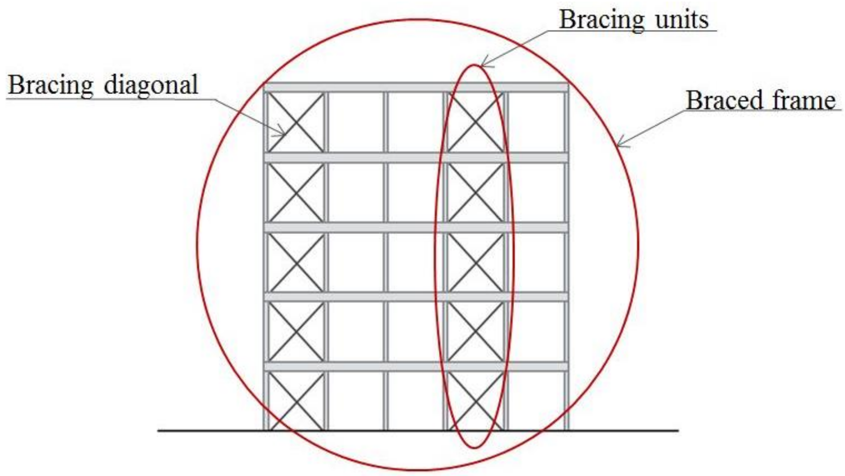


Figure 11 An example of a braced-frame structure.

Centric bracing

In a centric bracing system columns, beams and diagonals intersect in one point. A centric bracing system resists lateral load entirely by the bracing diagonal, hence the column-and-beam system only needs to be designed for vertical loads. This means that the dimensions of the beams can be the same independently of the height of the building (Merza & Zangana, 2014).

There are several types of bracings used for centric bracing systems. Figure 12 illustrates some common ways to brace a structure. The continuous lines symbolise tension and the dotted lines symbolise compression. Figure 12 has been inspired by Ching, et al. (2009). The bracing units can resist the lateral loads in tension, compression or in a combination of both, depending on the design.

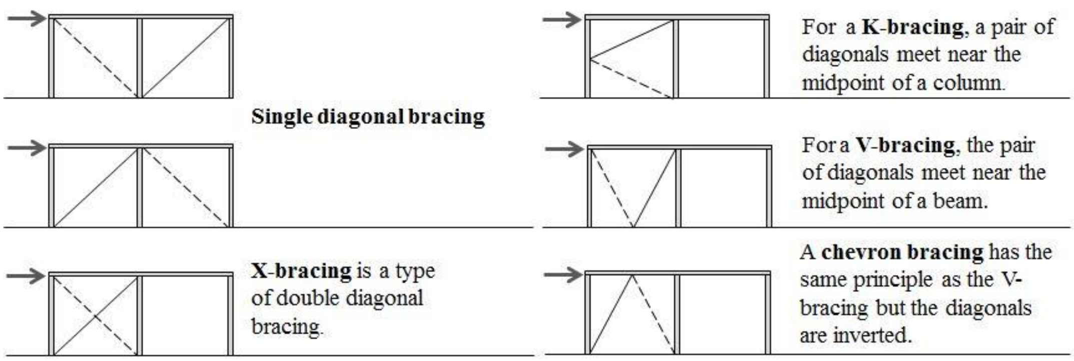


Figure 12 Different types of centric bracing.

Figure 13 illustrates how some of the configurations resist horizontal loads when there is more than one storey.

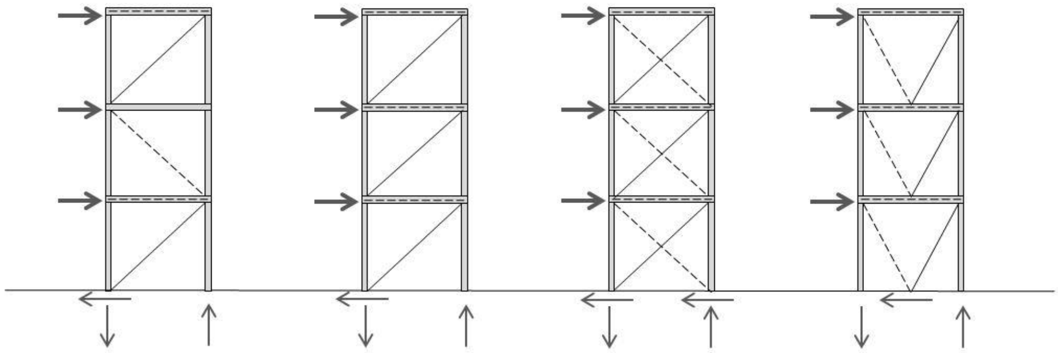


Figure 13 Principle of how different configurations resists lateral load. Dashed lines symbolises compression and solid lines symbolises tension.

Eccentric bracing

In an eccentric bracing system there is an offset from the connection of the diagonal brace and the connection between the column and the beam, see Figure 14. This means that the lateral load is not only resisted axially by the beam, but also via short link-beams that are formed between the braces and the columns or between two opposing braces (Ching, et al., 2009). By utilising eccentric bracing some of the frame action of rigid frames is used to resist lateral load in bending and shear; hence the forces in the diagonal bracing are reduced. This increases the ductility of the structure and therefore eccentric bracing systems are preferred in seismic zones where large plastic deformation capacity is needed. They can also be good when wide openings are required. However eccentric bracing provides the system with lower stiffness and is less efficient than centric bracing (Ali & Sun Moon, 2007).

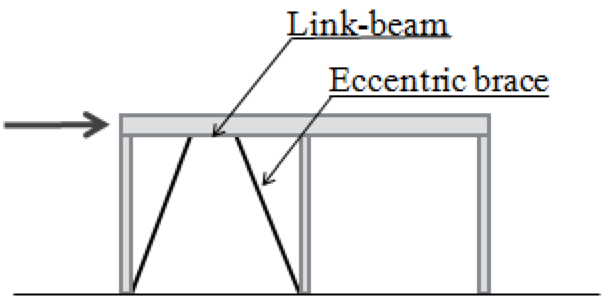


Figure 14 Example of eccentric bracing.

3.2.4 Tube structures

The idea of a tube structure is to place the stabilising members in the façade. Assumed that the coupling between the composed walls is sufficiently, a large flexural stiffness can be obtained partly by the great lever arm. The tube behaves like a large composed section in cantilever action, which is the basic model of a tube structure. When a tube is loaded by a horizontal force the walls parallel to the force will act as webs and those perpendicular will act as flanges. Webs resist shear and the flanges resist bending by their axial response.

For continuous box tubes the stress distribution between the two flanges varies linearly and the stress in the flanges are constant. This is, however, not the case for tubes which always have a behaviour between the ”real tube” and a frame, due to openings in the façade or use of columns and beams. This leads to shear lag, which makes the middle columns less loaded and the columns in the corners more loaded (Eisele & Kloft, 2002). Shear lag can be a problem if the normal forces in the corner columns are becoming too large (Eisele & Kloft, 2002).

For buildings that reach 40-110 storeys, tube structures are a good solution. ”The Willis Tower” in Chicago is built with the bundled tube- technique and was the tallest building in the world between 1973 and1998.

Table 2 different tube systems are described.

Table 2 Description of different tube systems.

Tube system	Structural behaviour
Exterior tubes	Perforated façades, which are not as stiff as shear walls used for cores, but the greater level arm is compensating (Eisele & Kloft, 2002).
Rigid frame tubes	Rigid frame tubes are made of a rigid frame, see Section 3.2.2.
Tube-in-tube	Two tubes where the exterior tube can be either a perforated concrete tube or a rigid steel frame and the interior tube is either a core or a braced frame. The exterior and interior tubes are coupled by floors to act more as one unit, providing higher stiffness (Ching, et al., 2009).
Bundled tubes	Two or more tubes that are connected to each other forming one unit. This provide a larger stiffness because the bundled tube have more than two webs, resulting in less shear lag and higher contribution from the flange planes in resisting bending (Ching, et al., 2009).
Braced tubes	Braced tubes are using diagonals which are by axial stresses stabilising the system against lateral load and is considered as the most efficient of all the different tubes (Eisele & Kloft, 2002).

3.2.5 Core structures with outriggers

An efficient way of stabilising tall slender buildings is to use a core structure with outriggers. The idea with outriggers is to couple the exterior column to the core, increasing the capacity of resisting lateral load, see Figure 15. Outriggers increase the effective depth of the effective cross-section which adds stiffness by forcing the exterior columns to act in tension and compression. Thereby, outriggers reduce the bending deformation of the core resulting in less bending moment and lateral deflection. Different location of the outriggers has different effects on the behaviour of the system. Outriggers located in the upper part of the building reduce the lateral deflection and outriggers placed in the lower part of the building resist bending moment (Stafford Smith & Coull, 1991).

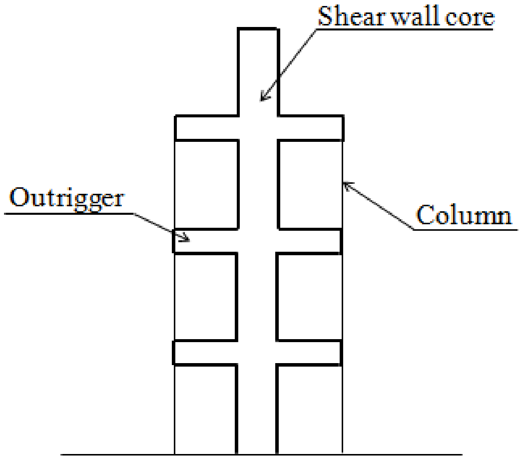


Figure 15 Core structure with outriggers connecting the façade to the centric located core.

Core structures with outriggers often provide a flexible floor plan. Outriggers can on the other hand be relatively large and often takes up the space of two storeys. It can also be hard to couple all the exterior columns to the core. One solution is to use a truss beam around the exterior columns. Therefore, floors where the outriggers are placed have limitations regarding the floor plan and windows; hence these storeys are often used for machinery and installations (Samulesson & Svensson, 2007). However, outriggers can be an economic solution for very tall buildings, 40 to 150 storeys (Ali & Sun Moon, 2007). Flexural behaviour is the major response to lateral load. In other words, core structures with outriggers do not get significant shear deformations (Stafford Smith & Coull, 1991). As mentioned in Section 3.1.2, the effect of shear deformation can often be neglected for slender buildings.

3.3 Conventional structural systems for timber buildings

When choosing structural systems for timber buildings it is of interest what kind of buildings that are to be designed. Different buildings, such as small houses, sports auditoriums and office buildings have different requirements concerning spans, load carrying capacity, fire resistance and acoustics. According to Crocetti, et al. (2001) timber systems can be categorised into three different main categories:

- Panel systems
- Modular systems
- Beam-column systems

If no other source is indicated the information in Section 3.3 is taken from TräGuiden (2015).

3.3.1 Panel systems

Load bearing parts in a panel system are plane elements such as wall elements and floor elements. Panel systems can be subdivided into two different structural systems, light frame panel systems and solid wood panel systems (Crocetti, et al., 2011). The main principle for both of the systems is the same. Load bearing walls resist both vertical and horizontal loads. Floors distribute the effect of horizontal loads from the façade to the stabilising walls through diaphragm action.

Light frame panel systems

In Sweden light frame panel systems are common for single-family houses but can also be utilised in taller buildings. However, today there is a technical limit of seven storeys due to unaccepted vertical deformations perpendicular to the grain (Crocetti, et al., 2011).

Light frame walls are composed of vertical studs connected to a top and bottom rail. The studs have a rectangular cross-section where the thickness in the direction of the wall is thinnest. Boards are usually placed on both side of a frame wall. The boards can either be nailed or screwed onto the vertical studs preventing the studs from buckling in their weak direction. Horizontal intermediate studs can also be used in order to prevent buckling of the vertical studs. If light frame walls are to be used as stabilising walls, the studs need a thickness of at least 45 mm. Normally studs are placed with spacing of 600 mm. Higher stiffness and vertical load bearing capacity can be obtained with a smaller spacing between the vertical studs.

Since the vertical studs are prevented from buckling in their weak direction the load bearing capacity is determined with regard to buckling in their strong direction .The load bearing capacity of light frame walls are calculated with the assumption that the studs alone resist the normal forces and bending moments.

When designing floor structures in light frame systems the acoustic performance is usually decisive. In order to prevent disturbing sound and vibrations the floor spans need to be limited (Crocetti, et al., 2011).

Solid wood panel systems

Solid wood panel systems are composed of solid load bearing wall elements and floor elements. Often cross-laminated timber is used. According to Crocetti, et al. (2001) solid wood panel systems are today common in Germany, Austria and Switzerland. When using solid wood panel systems in areas with a colder climate, such as in the Nordic countries, solid wood walls are often complemented with insulated light frames in order to achieve a proper thermal performance.

Solid wood panel systems are well suited for buildings with high demands on load bearing capacity and sound insulation, for example multi-storey residential buildings. When using solid wood walls and floors a high load bearing capacity can be achieved, which enables longer spans and a better stabilisation compared to light frame panel systems. Another benefit is that walls and floors can be prefabricated to a great extent.

Today floor elements in solid wood panel systems can span approximately 12 metres. For offices and schools that sometimes demand longer spans the solid wood panel system can be combined with interior beam/column lines.

The Swedish timber component manufacturer Martinson is producing a system called ‘KL-trä’ (KL-wood), which consists of cross-laminated timber members. Today, Martinson is able to design buildings with KL-trä up to eight storeys. The maximum span for floors is 12 metres in their residential buildings. The ‘KL-trä’ system is stabilised by shear walls (Martinsons, 2014).

3.3.2 Modular systems

Modular systems are composed of prefabricated box modules that are assembled on top of each other on site. The prefabricated modules enable a fast construction, but the choice of transportation results in a size limitation of the modules. Commonly a module measures 8-13 metres in length, 4.1-4.2 metres in width and 3.10 metres in height.

Another limitation of the modules is the size of openings in the walls. For a four storey building the length of an opening is limited to 3-3.5 metres. This limitation occurs due to problems of resisting concentrated forces that occurs in the corners.

Modular systems do not allow for long spans, since the size of the modules and the possibility of removing internal walls is limited. The system is therefore suitable for student housings and hotels since these types of buildings generally do not require large open areas. Today it is possible to build houses with modular systems up to seven storeys (Crocetti, et al., 2011). However, a 14 - storey building with this system is under construction in Bergen, Norway. The building is composed of modular systems within an outer bracing timber frame, see Section 2.4.5.

According to Crocetti, et al. (2001) modular systems can be subdivided into two different structural systems; light frame modular systems and solid wood modular systems. Solid wood modular systems are more stable, since the stabilising walls are stiffer. Another advantage is that solid wood has a higher self-weight, which enables less coupling between the storeys.

A company that builds modular systems in Sweden is Setra and their system is called ‘Plusshus Moduler’. The modules usually represent one room and are often equipped with flooring, drywalls, kitchen and bathroom when delivered to the construction site. According to Setra (2015) the system is fast to construct, faster than panel systems.

3.3.3 Beam-column systems

The principle of beam-column systems is that the vertical load effects are resisted by columns and beams. This system is normally used, when open spaces are required, which is common for commercial buildings and office buildings. Timber beams, such as glulam beams, can be used for spans up to 80 metres. However, timber beams used for spans around 80 metres are only used for arenas or swimming halls, where the beams can have large cross-sections (Crocetti, et al., 2011).

In schools and office buildings a combination of solid wood panel systems and beam-column systems are often used. For such systems horizontal loads are resisted by diaphragm actions in massive timber floors and bracing units such as shear walls or systems with diagonal bracings. If open spaces are required shear walls can be placed in the façade or in stairwells and elevator shafts.

The total construction height of a floor structure with continuous beams is often higher than floor structures with simply supported beams. This is because floor elements need to be placed above the beams if continuous beams are used. Otherwise the beams can be connected to the floor element according to Figure 16. If timber beams are of interest, it is usually good to use glulam beams due the possibility of increasing their width and thereby decreasing their height (Martinsons, 2006).

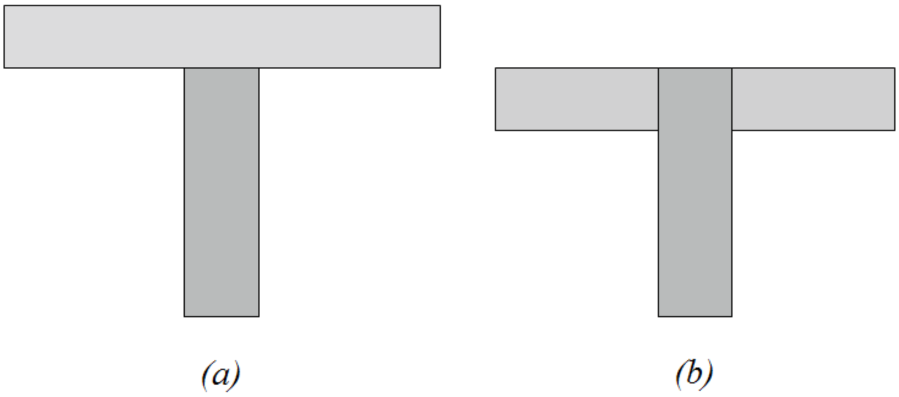


Figure 16 Example of how beams can be placed, (a) continuous beam, (b) simply supported beams.

If timber is loaded parallel to its grains, the deformations are relatively small compared to the deformations when loaded perpendicular to the grains.
Glulam members usually do not require extra cover for fire safety. Additionally glulam has high load bearing capacity compared to other timber products. Therefore timber could be a good solution for columns.

An example of a beam-column system for multi-storey timber buildings is ‘Trä8’, which was developed by the company Moelven Töreboda and was introduced in 2009 (Moelven Töreboda, 2015). The system is especially adapted for buildings with four storeys.

The’ Trä8-system’ is prefabricated and contains timber elements only. Columns and beams are made of glulam and bracing units are made of glulam combined with the veneer material Kerto. The floor and roof elements are made of panels of Kerto-Q and beams of Kerto-S. This provides the floor and roof structures with good properties regarding deflections, vibrations and acoustics. Insulation is placed in the voids between the beams. The floor elements are 8×8 metres, which enables an open and flexible structural system. In Figure 17 the ‘Trä8-system’ is shown.

Figure 17 The structural system 'Trä8' developed by Moelven Töreboda (2015).

3.4 Structural members in timber

In this section conventional structural members in timber are presented. The specific design of the members depends on the supplier and therefore only the principles behind the different members are described. If no other source is indicated the information in Section 3.4 is taken from TräGuiden (2015).

3.4.1 Timber beams

Timber beams are often made with solid sections of structural wood, glulam or veneer such as Kerto-S. Another type of beams is light beams, which usually have I-sections or box-sections. The webs and flanges of a light beam are often composed of different materials. Boards are often used for the flanges and solid wood for the webs. Structural wood is suitable for short spans up to five metres. When longer spans are demanded, it is better to use beams of glulam, veneer or light beams.

Rectangular timber beams are often produced with a height-to-width ratio of 4:1or 5:1. The dimensions of a cross-section with a high height-to-width ratio is often governed by shear capacity, while a cross-section with a low height-to-width ratio often is governed by the long term deflections.

3.4.2 Timber floors

Floor structures need to resist permanent loads, imposed loads and horizontal loads from wind. Load effects from the horizontal loads are distributed by floor structures to the bracing units. However, it is often the performance in the service state that is decisive. The major challenge is often to limit the height of the floor. The height depends on several factors in the service state such as vibrations, deflections, fire safety and acoustics. According to Eurocode 5, CEN (2009), it is therefore important to design floors such that their eigenfrequency is larger than 8 Hz. It is usually easier to obtain an acceptable eigenfrequency by having continuous beams and floors compared to having simply supported beams and floors, due to less deflection. However, continuous beams affect the acoustic performance of the building negatively due to an increase of flanking transmissions.

There are several ways to design timber floor structures for timber buildings. Generally timber floors can be subdivided into two main systems.

- Solid wood floor elements
- Light weight floor elements

Solid wood floor elements

For solid floors in residential buildings and office buildings the utilisation ratio in the ultimate limit state is seldom higher than 0.5. Instead floor elements are designed according to requirements in the serviceability limit state, such as limited deformations and vibrations (Martinsons, 2006).

Solid wood floor elements are predominantly consisting of massive wood and can be manufactured in various ways. The different types can generally be subdivided into plane element floors, cassette floors and composite floors. Common for them all is that they can be prefabricated to a great extent. According to Martinsons (2006) solid wood floors are often made of sawn timber, glulam or laminated veneer lumber (LVL).

Plane floor elements are composed of several timber panels that are glued or mechanically fastened to each other. The number of boards is uneven and every other panel is placed crosswise, which provides the floor with a higher stiffness than if all boards were placed in the same direction. Another benefit is that movements due to moisture changes are reduced. Plane floor elements can be either continuous or simply supported. According to Martinsons (2006) the largest recommended span for glued plane floor elements with one field is 4.6 metres for residential buildings and 4.4 metres for offices.

In order to fulfil the acoustic requirements plane floor elements need to be complemented with a sound-insulating ceiling. Thermal insulation and services such as ventilation and sewer can then be placed between the floor and the ceiling (Martinsons, 2006). Figure 18 presents a plane floor element.

Figure 18   Example of a solid wood floor element (Swedish wood, n.d).

Cassette floor elements have a composite section composed of a top flange of crosswise glued panels of solid wood and webs and bottom flanges of glulam, see Figure 19. Normally a cassette floor element has a width of 2.4 metres and a height of 0.3-0.65 metres. The maximum span of the cassette floor elements that are available today is 12 metres. The void between the webs can be filled with insulation and services for ventilation and sewer. Martinsons (2006) states that, since insulation and services can be hidden in the floor structure, the total height is lower than for plane floor elements. Cassette floor elements are often complemented with a ceiling that hides the insulation that is placed between the webs. Compared to plane floor elements cassette floor elements have a lower self-weight due to the use of webs.

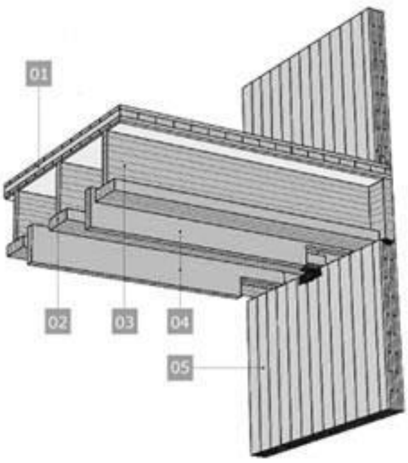


Figure 19 Example of a cassette floor element (Swedish wood, n.d).

Timber-concrete composite floors consist of timber beams with a concrete slab cast on top, see Figure 20. The principle for these floors is that the timber members resist tension, the concrete slab resists compression and the connection resists shear forces in the joint interface. It is important that the connection is stiff enough in order to obtain composite action. The connection is often crucial for the performance of the floor structure (Lukaszewska, 2009).

By providing the floor structure with a concrete slab the load bearing capacity can be increased and spans up to 12 metres can be obtained. The span can be increased further by the use of reinforcement (Crocetti, et al., 2011). In addition, composite floors with timber and concrete have higher stiffness and thereby less deflections and also improved fire safety, acoustics and thermal properties compared to other timber floors. However, the mass of the floor is increased resulting in a lower natural frequency (Lukaszewska, 2009).

According to Kliger (2015-03-12) composite floors are common in southern Europe, especially when strengthening existing timber floors. Today all composite floors of concrete and timber are manufactured by casting concrete on top of timber beams. However, the best performance would be obtained if using prefabricated concrete since shrinkage effects would be minimised. Timber-concrete composite floors with pre-cast concrete are not available on the Swedish market today.

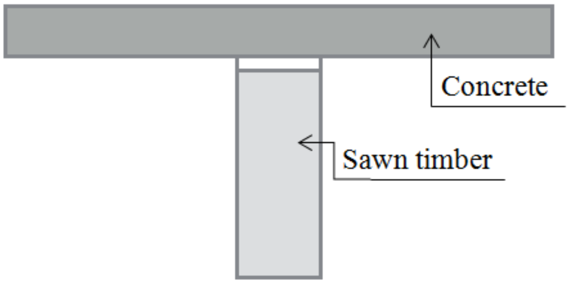


Figure 20 Example of a timber-concrete composite floor element.

Light weight floor elements

Light-weight floors are often composed of a load-bearing part and a suspended ceiling with its own load-bearing structure. The load bearing part of the floor is typically composed of boards and beams with insulation in between. Structural wood, glulam, light beams and solid wood are often used for the beams. Examples of boards are fibre boards, particle boards or oriented strand boards (Crocetti, et al., 2011). Figure 21 shows a typical light-weight floor. Light weight floors are often used in single family houses and are not commonly used between apartments.

According to Crocetti, et al. (2011) the separation of the load-bearing part and the ceiling is performed in order to ensure sufficient acoustic performance. If there is no contact between the ceiling and the load-bearing part vibrations cannot be transmitted. Thereby a good sound insulation is obtained.

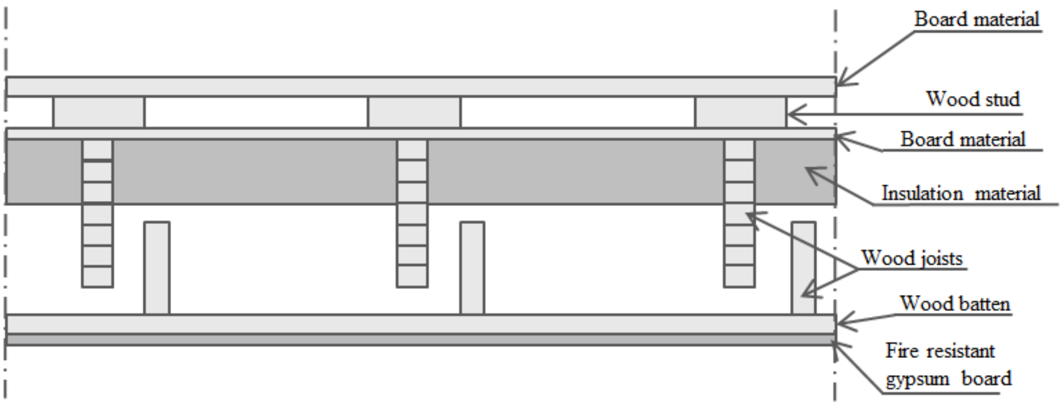


Figure 21 Section of a light weight floor element.

3.4.3 Timber walls

There are two types of load bearing timber walls available today; light frame walls and solid timber walls. The latter provides more stability to the structure and higher vertical load bearing capacity. On the other hand, light frame walls are easier to insulate, since insulation can be placed inside between the studs. Therefore, if solid timber walls are used as outer walls, they are often combined with insulated light frame walls (Crocetti, et al., 2011).

Solid timber walls elements consist of glued or screwed layers of cross-laminated timber panels, CLT-panels. Every other panel is rotated 90° to obtain higher stiffness in the wall, see Figure 22.

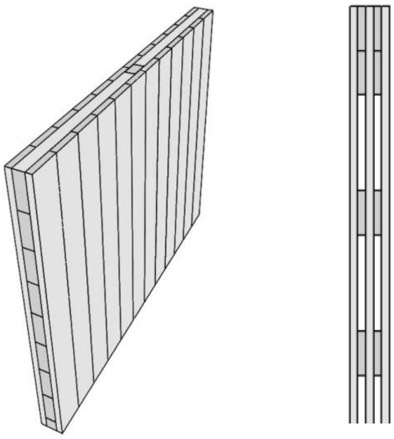


Figure 22 Solid timber wall elements (Swedish wood, n.d).

Light frame walls are mostly made of structural timber, but glulam and veneer can also be used. The studs are prevented to bend out in their week direction by the wall boards. Studs placed in walls that are considered as stabilising walls should be at least 45 mm thick. The width of structural timber is usually 95-220 mm, while glulam and veneer studs can be made wider if necessary. In many cases when designing low-rise buildings, it is not the loads that govern the width of the studs. Instead it is the need of insulation that determines the width, especially in countries with cold climate. Stud spacing is usually 600 or 450 mm, but can be decreased to increase the load bearing capacity if needed. Figure 23 shows a typical light frame wall.

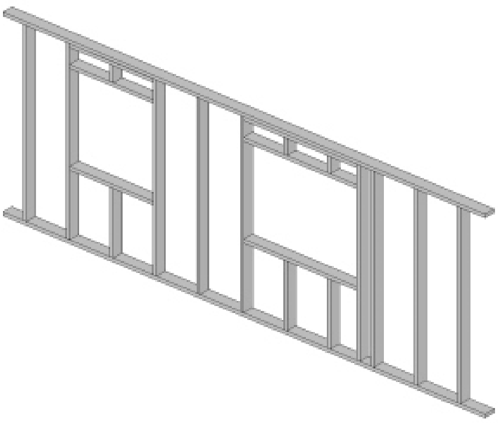


Figure 23 A light frame wall element (Swedish wood, n.d).

As described in Section 2.3.3 timber subjected to compression perpendicular to the grain results in large deformations. Therefore stud walls which have two members subjected to compression perpendicular to the grains for each element may be less suited than CLT-walls for taller timber buildings.

3.4.4 Timber columns

Columns are often used in timber buildings as a part of the primary load bearing system. Columns that are placed in walls can be braced against buckling in one or two directions. Detached columns on the other hand need to be designed with regard to buckling in all directions. Because of this, it is often beneficial to choose a square or circular shape of the cross-section for a detached column. However, structural timber and glulam are mostly provided in rectangular shapes. These are the two timber components that are the most common ones for columns. Structural timber can only have cross-section dimensions up to 250 mm. Glulam, on the other hand, can be made with larger cross-sections. The width of the lamellas is limited to 215 mm. However, according to Carling (2001) lamellas can be nailed, screwed and/or glued together in order to obtain larger widths. The cross-section of glulam columns can also be made in different shapes; I-, T- or L-shapes are possible.

4 Design Principles

The calculations presented in Chapter 5 and Chapter 6 have been performed in accordance with the general principles presented in this chapter. Design principles with regard to fire, acoustics and vibrations are also included in this chapter. For more information about the detailed design procedures the reader is referred to Eurocode unless another source is specified. In this chapter the design principles of members that have been designed on the basis of tabulated values are not described.

4.1 Compressive resistance of slender members

When designing columns according to Eurocode the columns can be considered as either slender or non-slender. If a column is slender load effect increases due to structural deformations, which is called second order effect. The slenderness of columns is calculated using the buckling length according to Equation (3).

$$\lambda = \frac{l_0}{i} \tag{3}$$

l_0	Buckling length
$i = \sqrt{\frac{I}{A}}$	Radius of gyration

According to part 1-1 of Eurocode 5, CEN (2009), timber columns that are not loaded horizontally should be designed to fulfil Equation (4).

$$\frac{\sigma_{md}}{k_c \cdot f_{c,0,d}} \leq 1 \tag{4}$$

σ_{md}	Bending moment
$f_{c,0,d}$	Compressive strength
k_c	Reduction factor of the strength taking the slenderness of the column into account

According to part 1-1 of Eurocode 2, CEN (2008b), slender concrete column sections should be designed for the combined effect of normal force and bending moment where the design moment should take the second order effects into account, see Equation (5) and Equation (6).

$$M_{Ed} = \left[1 + \frac{\beta}{\frac{N_b}{N_{Ed}} - 1}\right] M_{0,Ed} \tag{5}$$

$$M_{0,Ed} = N_{Ed}(e_0 + e_i) \tag{6}$$

N_b is the theoretical buckling load based on a nominal stiffness and the buckling length of the column. M_{0Ed} is the first order moment due to unintended inclination and intended eccentricities and transverse loads. N_{Ed} is the normal force and β is a factor which depends on the moment distribution from the first and second order effects. If there is no intended eccentricity or horizontal load e_0 becomes zero.

4.2 Tensile resistance of members

Columns and especially bracing components can be subjected to tension. For members that experiences both compression and tension, the former is in most of the cases the governing effect. However, the tensile capacity needs to be checked. For steel and timber this is done in similar ways. According to part 1-1 of Eurocode 3 CEN (2008a) and part 1-1 of Eurocode 5, steel is designed with respect to an axial force, while timber is designed with respect to an axial stress. Equation (7) presents the design criterion for steel and Equation (8) presents the design criterion for timber.

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1 \tag{7}$$

$$\sigma_{t,0,d} \leq f_{t,0,d} \tag{8}$$

N_{Ed}	Applied normal force
$N_{t,Rd}$	Tensile resistance
$\sigma_{t,0,d}$	Tensile stress
$f_{t,0,d}$	Design tensile strength

4.3 Design of timber beams

The design process of timber beams includes verification of deflections, bending resistance and shear resistance. According to part 1-1 of Eurocode 5 timber beams are designed with regard to bending according to Equation (9).

$$\frac{\sigma_{md}}{f_{md}} \leq 1 \tag{9}$$

σ_{md}	Bending moment
f_{md}	Bending capacity

The maximum normal stress in timber beams can be obtained by Navier's formula. For a simply supported beam with uniform load the maximum stress is found in the mid-span. The shear stresses are checked according to Equation (10).

$$\tau_d \leq f_{v,d} \tag{10}$$

τ_d maximum shear stress in the beam

The final deflections with regard to time dependent effects are obtained by multiplying the initial deflections with a factor of $1+k_{def}$ taking the creep deformations of the timber beam into account. The coefficient k_{def} depends on the service class. In this project the criterion for the deflections was set to the effective span divided by 400.

4.4 Design of timber-concrete composite floors

In this project design recommendations developed by Linden (1999) were used in the design of timber-concrete composite floors. The basis of this method is presented in this section. For further description of the method, the reader is referred to Linden (1999).

The design recommendations are based on linear material behaviour and cover composite action between solid timber beams and concrete slabs. In order to obtain linear material behaviour the timber beams must be of ordinary strength class. If higher strength classes are used, nonlinear behaviour must be considered, preferably by FEM models. The design recommendations are valid for concrete strength classes between C15 and C35.

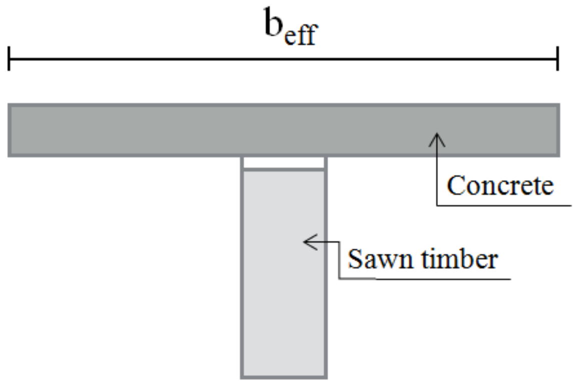


Figure 24 Example of a timber-concrete composite floor

For the verification of the load capacity, the floor structure is regarded as a simply supported T-beam with an effective width of the concrete slab, see Figure 24. For timber-concrete composite beams the dimensions are often governed by creep deformations. Therefore, creep is taken into account by reducing the mean value of the elastic modulus. The creep coefficients are determined according to Eurocode 2 and Eurocode 5. For timber, the creep coefficient can be taken as k_{def} in combination with long term load. The expressions for the effective elastic modulus at a certain time are presented in Equation (11) and Equation (12).

$$E_{t.d} = \frac{E_{0,mean}}{1 + k_{def}} \tag{11}$$

$$E_{c.d} = \frac{E_{cm}}{1 + \varphi} \tag{12}$$

Since the beam is composed of two materials the effective bending stiffness needs to be computed. This is performed according to Linden (1999). The principle behind the equation is that the transformed cross-section is defined on the basis of one of the materials, in this case timber. The other material, the concrete slab, is weighted by the factor n_{ct} and thereby transformed into equivalent timber material, see Equation (13).

$$EI_{ef} = E_{t.d}[I_{tot} + \gamma \cdot (n_{ct}A_c{e_c}^2 + A_t{e_t}^2)] \quad (13)$$

$E_{t.d}$	Modulus of elasticity of timber
I_{tot}	Second moment of inertia of timber and transformed timber section
γ	Effectiveness of the connectors
n_{ct}	Ratio between the moduli of elasticity of concrete and of timber
A_c	Cross-section area of the concrete slab
e_c	Eccentricity of the concrete slab in relation to the neutral axis
A_t	Cross-section area of the timber beam
e_t	Eccentricity of the timber beam in relation to the neutral axis

When the effective bending stiffness is obtained, compression, tension and shear stresses in the composite beam section can be calculated and checked in the ultimate limit state, see Figure 25.

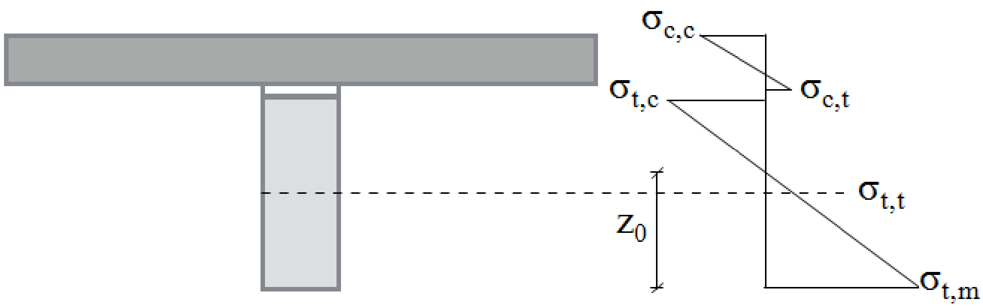


Figure 25 Example of stress distribution in a timber-concrete composite beam.

$\sigma_{c,c} \leq f_{c,c,d}$	Design compressive strength of concrete
$\sigma_{c,t} \leq f_{c,t,d}$	Design tensile strength of concrete
$\sigma_{t,c} \leq f_{t,m,d}$	Design bending strength of solid timber, parallel to grain
$\sigma_{t,t} \leq f_{t,t,d}$	Design tensile strength of solid timber, parallel to grain
$\sigma_{t,m} \leq f_{t,m,d}$	Design bending strength of solid timber, parallel to grain
$\sigma_{t.v.d} \leq f_{t.v,d}$	Design shear strength of solid timber

It is also important to check the shear capacity of the connection at the joint interface. The maximum shear force in the connection should fulfil the requirement below.

$f_{v,d} \leq f_{v,mean,d}$ Design strength of the connectors

In addition floor structures have to be designed with regard to the performance in the serviceability limit state, which concerns final and instantaneous deflections, eigenfrequency and velocity response. For timber-concrete composite beams this can be performed according to part 1-1 in Eurocode 5, CEN (2009). In this project the limit of deflections was set to the span length divided by 500 and the eigenfrequency for timber-concrete composite floors had to be higher than 7 Hz.

4.5 Design of walls

For the component study the thickness of the wall was determined under the assumption that the walls are carrying load vertically and they are not contributing to the global stability of the structure. However, the wall needs to resist the load effects out of its plane from wind that acts locally on the wall.

When designing walls for vertical compression, the region between the windows can be assumed to behave as a column. The calculation principle is the same as described in Section 4.1, but for outer walls the out of plane effect of horizontal loads from the wind needs to be considered. Hence the resistance for the combined axial load and bending needs to be checked. The column part of timber walls should be designed to fulfil the criterion for combined axial compression and bending according to part 1-1 in Eurocode 5, see Equation (14).

$$\frac{\sigma_{c,0,d}}{k_{c,y} * f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1 \tag{14}$$

$\sigma_{c,0,d}$	Applied compression stress parallel to grains
$\sigma_{m,y,d}$	Applied bending moment
$k_{c,y}$	Reduction factor taking slenderness into account
$f_{c,0,d}$	Compression strength parallel to grains
$f_{m,y,d}$	Bending strength

The load bearing capacity of the column part of concrete walls needs to be checked with regard to second order moment according to Equation (5) in Section 4.1. However, the effect of the wind results in an intended initial eccentricity (e_0) that is added to the eccentricity due to unintended inclination (e_i). Hence the first order moment is calculated according to Equation (6).

The part of the wall above the window can be regarded as a beam and needs to be checked with regard to bending, shear and deflection. For the component study the load effects from wind was then disregarded. This simplification was made, since the wind load proved to have a small influence on the total load and the utilisation of the beams proved to be low.

4.6 Design with regard fire

If no other source is indicated Thor (2012) has been used as source.

When designing buildings with regard to fire load they can be divided into different building classes depending on the required need of protection. The building classes are Br0, Br1, Br2 and Br3 and are depending on the number of storeys and the activity in the building. Br0 has the highest requirements and Br3 has the lowest.

Moreover , the different components of a load bearing structure is divided into different fire safety classes, which depends on the risk of serious injury in case of a collapse of a certain structural member. The fire safety class of a structural component is ranked from one to five, where one equals minor damage and five equals very large damage. When deciding the fire safety class for a certain structural member, the building class needs to be taken into account. This means that the same structural member can have different fire safety classes depending on the type of building it is intended for.

Each fire safety class gives a requirement for the load bearing capacity in case of a fire. This requirement is indexed R (resistance) and the number after indicates how many minutes the resistance has to be maintained during a fire scenario that follows a standard fire curve. Furthermore, the required number of minutes depends on the expected amount of fire load in the building and the fire safety class. A fire load less than 800 MJ/m^2 is often assumed for offices, schools, residential buildings and comparable fire cells.

Therefore, in the component study and in the development of structural systems, the fire load was assumed to be less than 800 MJ/m^2. All components were assumed to be in safety class 5. Hence, the components needed to be designed for the standard fire resistance R90.

4.6.1 Load combination in design with regard to fire

The load case for verifying resisting in case of fire assumes lower loads, partial coefficients and load factors than the load combinations for design in the ultimate limit state, see Equation (15). The load combination for the load case fire is found in Eurocode 0, CEN (2010a).

$$Q_{fire} = \sum_{k\geq 1} G_k + \psi_1 \, Q_k \tag{15}$$

G_k Characteristic value for the permanent load
Q_k Characteristic value for the imposed load
ψ_1 0.5 for office buildings

4.6.2 Behaviour of building materials subjected to fire

General for all materials is that the strength is reduced when the material is heated. In the case of fire the cross-section of steel and concrete remains constant, while the load bearing capacity of the component is reduced.

During a fire the temperature in the concrete and the reinforcement increases and the load bearing capacity is reduced. Since a concrete member often has a high mass and heat capacity, the heating is often slow, especially when compared with the heating of steel. The core of a concrete member remains cool longer than the outer layers and thus retains its strength.

For concrete members loaded in bending, such as floor slabs and beams, the reduction of the strength of the reinforcement is decisive. In order to fulfil the requirements for load bearing capacity the concrete cover needs to be thick enough.
By ensuring that the concrete cover is thick enough the temperature rise in the reinforcement can be delayed, which means that it takes longer time before the concrete member loses load bearing capacity. For members in compression, such as

walls and columns, it is the reduction of the strength of the concrete that is decisive. In contrast to beams and slabs the thickness and dimensions of the member is then of importance.

Steel has a high thermal conductivity and in comparison with concrete the cross-section gets an evenly distributed temperature relatively fast. Therefore steel members often demand some kind of additional fire protection, especially if the demand for the standard fire resistance is high.

The behaviour of timber members differs from steel and concrete, since the cross-section does not remain constant. Instead the cross-section is charred gradually. The charred part of the cross-section is assumed to lack load bearing capacity, while the non-charred part has full capacity. Hence, the remaining part of the cross-section after a certain time period needs to be able to withstand the loads.

4.6.3 Design of concrete members with regard to fire

In Eurocode 2 there are three different design methods for the load bearing capacity with regard to the load case fire; table values, simplified calculation methods and advanced calculation methods. According to Thor (2012) the most common alternative is to use table values and therefore this method was used in the component study. The tables provide the minimum cross-sectional dimensions and cover thickness for a concrete member with regard to a certain standard fire resistance and utilisation ratio at the load case fire.

For the component study values for the dimensions were taken directly from Thor (2012), these values are based on values according to part 1-2 in Eurocode 2. For further details on the conditions for the tabulated values the reader is referred to Thor (2012).

4.6.4 Design of steel members with regard to fire

As for the design of concrete members there are several methods presented in Eurocode for determining the dimensions of a steel member with regard to fire. These are described in part 1-2 in Eurocode 3, CEN (2010b). For the component study and the development of structural systems tabulated values for fire protection of steel members were used.

Due to the high thermal conductivity of steel, the whole cross-section becomes heated fast. In order to prevent heating and thereby delay the reduction of the load bearing capacity steel members are often covered with either board material such as gypsum or rock wool or painted with fire prevention colours. It is also a possibility to integrate the steel members into walls or floor structures. In this project it was assumed that fire gypsum boards are used in order to ensure that the steel members fulfil all requirements regarding fire protection. Values for the thickness of the fire protection were taken from Gyproc (2010).

4.6.5 Design of timber members with regard to fire

The load bearing capacity of a timber member during fire is calculated according to part 1-2 in Eurocode 5, CEN (2010c), by reducing the cross-section with a charring depth. The remaining cross-section needs to withstand the design load from the load combination for fire.

The charring depth is calculated with regard to the fire exposure time and the charring rate. For a timber member that is unprotected during the whole fire exposure time the charring depth is calculated according to Equation (16).

$$d_{char,0} = \beta_0 \times t \tag{16}$$

β_0	Design charring rate [mm/min]. (0.65 for glulam, LVL and massive timber made of coniferous wood or beech)
t	Exposure time [min]

The design strength of a timber member subjected to fire is determined according to part 1-2 in Eurocode 5, see Equation (17).

$$f_{d,fi} = k_{mod,fi} \cdot k_{fi} \frac{f_k}{\gamma_{M,fi}} \tag{17}$$

$k_{mod,fi}$	Conversion factor for timber
k_{fi}	Modification factor for fire (1.25 for solid wood, 1.15 for glulam and 1.1 for LVL)
f_k	Characteristic strength
$\gamma_{M,fi}$	Partial factor for wood in fire design (recommended value $\gamma_{M,fi} = 1$)

4.7 Design with regard to acoustics and vibrations

If no other source is specified the information in this section is taken from Hagberg (2010).

Sound is a phenomenon caused by pressure differences in the air. The sound level is measured in decibel (dB), where 0 dB is approximately the lowest audible sound level for humans and an increase of 10 dB corresponds to a perceived doubling of the sound. Furthermore, different frequencies for the same sound level results in different perceptions of humans. A sound source that is located 0.5-1.0 metres from a person gives a sound pressure level, which has a larger part coming from reflecting sounds than from the direct sound. Therefore, the amount of absorbers decides how the sound environment in a room is perceived (Ljunggren, 2011).

Sound can be divided into two parts, airborne sound and impact sound. Airborne sound comes from talking, stereos and TVs, while impact sound arises from footsteps, scratches from chairs and thuds. Light weight buildings, such as timber buildings, can obtain good insulation against airborne sound for higher frequencies. In contrast, it is hard to obtain a good solution that has acceptable performance regarding impact sound and low frequency airborne sound.

When a person walks or jumps on floors, vibrations are induced and the floors sag. Timber floors are more prone to vibrate and sag than concrete floors, due to a lower flexural stiffness. From the vibrations in the floor impact sound will arise. For timber floors this sound will usually obtain lower frequencies than sound from concrete floors. Further on, the sound level from timber floors will be higher due to larger vibrations.

Floors are regarded as light structures, if the self-weight is around 100 kg/m^2, and heavy, if the self-weight is approximately 300-350 kg/m^2. Timber floors are usually regarded as light floors, while concrete floors are regarded as heavy. Hence, timber-concrete composite floors are somewhere in between heavy and light-weight floors.

The acoustic performance of light weight buildings is hard to predict, since there are no standardised calculation methods. A sufficient acoustic performance is generally easier to obtain for walls than for floors. When designing light weight floors it is mostly hard to obtain a good impact sound insulation and reduce oscillations, especially if the floor has a long span or has a low fundamental frequency. Therefore, it is good to reduce the floor span, if possible, and to increase the fundamental frequency of the floor by increasing the stiffness or decreasing the mass. A thumb of rule in an early design stage is to assume that the height of the floor is at least 500 mm.

Generally it is recommended that the design with regard to acoustics and vibrations includes calculations concerning deflection and dynamic response for all light-weight floors. The deflection should be determined both for a static load and a dynamic impulse load. For the static load the deflection should be less than the span divided by 500. According to part 1-1 of Eurocode 5, CEN (2009), a good timber floor design concerning vibrations is to ensure a

- fundamental frequency of at least 8 Hz
- maximum instantaneous vertical deflection of 1 mm with 1 kN point load
- unit impulse velocity response less than $120^{f\xi-1}$ m/(Ns^2) where f is the fundamental frequency and ξ is the modal damping ratio

Hagberg (2010) sets other criteria:

- fundamental frequency of at least 16 Hz
- maximum instantaneous deflection of 1 mm with 1 kN concentrated load

For large spans the criteria from Hagberg (2010) are hard to fulfil without extensive measures. The span can either be shortened, the height of the beam can be increased or transverse stiffeners can be used.

Another phenomenon that needs to be considered is flanking transmissions, which arise when a sound travels through structural components into an adjacent room. The flanking transmissions can be prevented by avoiding continuous members between rooms and by avoiding stiff connections.

5 Component Study

In this chapter the results from the component study, where needed dimensions for different structural members were analysed, are presented. The outcome of the component study formed the basis for the development of the mixed structural systems. Dimensions are presented in tables and more information about the components, utilisation ratios and calculations are presented in Appendix A. Principles for the design calculations are presented in Chapter 4.

5.1 Methodology and assumed conditions

The component study was performed according to methods in Eurocode. However, standardised values for dimensions have been used, when such are available. Values for prestressed concrete members were taken from Svensk Betong (2015a) and values for steel members were taken from design tables from Tibnor AB (2011). In addition, values for the structural height for timber and concrete floors were taken from Martinsons (2006) and Svensk Betong (2015b). These standardised values are determined according to Eurocode. On the other hand, for composite floors there exist no tables or diagrams over standardised dimensions. Calculations were therefore necessary in order to obtain dimensions of the composite floor. A method based on the research of Linden (1999) was then used, see Section 4.4.

Design values for loads in the ultimate limit state were calculated according to Equations 6.10a and 6.10b in Eurocode 0, CEN (2010a). The calculations include permanent actions, such as weight from load bearing components and installations and variable loads, such as imposed load from office areas including load from partition walls. All loads were considered to be unfavourable and the design value for the loads was calculated as the maximum of the two expressions in Equation (18).

$$Q = max\begin{cases}\sum_{j\geq1}\gamma_{G,j}G_{k,j} + \gamma_Q\psi_0 Q_k \\ \sum_{j\geq1}\xi_j\gamma_{G,j}G_{k,j} + \gamma_Q Q_k\end{cases} \tag{18}$$

$G_{k,j}$	Permanent actions: weight of load bearing parts + installations (0.5 kN/m^2)
Q_k	Imposed loads: office load (2.5 kN/m^2) + partition walls (0.5 kN/m^2)
$\gamma_{G,j} = 1.35$	Partial safety factor for permanent load
$\gamma_Q = 1.5$	Partial safety factor for variable load
$\xi_j = 0.89$	
$\psi_0 = 0.7$	Category B: offices

For the calculations in the serviceability limit state a quasi-permanent load combination was used according to Eurocode 0, see Equation (19).

$$Q_{Quasi} = \sum_i G_{k,j} + \psi_0 Q_k \tag{19}$$

All dimensions were compiled with regard to normal spans, influence areas and loads from office buildings. However, in order to cover more cases, a couple of spans and loads were evaluated for each structural member. The weight of the floor structure was assumed to be 2 kN/m^2, which corresponds to a heavy timber floor.

The different members of the component study were analysed with regard to their performance in the ultimate limit state, the serviceability limit state and in the case of fire. For timber and concrete members, the dimensions were increased in cases where the members did not fulfil the requirements for fire. On the other hand, steel members are covered with fire gypsum boards with a thickness of 15.4 mm. The amount of layers needed for a certain steel member depends on the circumference and the thickness. Values for the fire protections were taken from tables from Gyproc (2010). The number of gypsum boards needed for a certain steel profile is presented in Appendix A6.

5.2 Columns

Dimensions of timber, concrete and steel columns were evaluated with regard to a number of vertical loads from 0.5 MN to 7 MN. By varying the load, the effect of the choice of material can be evaluated. For example, 6 MN corresponds to the load from a tributary area of 52 m^2, i.e. a span of 7.2 metres, on a column on the bottom floor in a 15-storey building. The height and buckling length was set to 3.6 metres.

Initially, both glulam and solid timber were evaluated according to the principles in Section 4.1. However, the load bearing capacity of solid timber columns proved to be too small for all the loads investigated. Strength class L40c was chosen for the quality of the glulam columns and standardised sizes of the lamellas were used, see Figure 26. For the concrete columns, strength class C30/37 and 16 mm reinforcement bars of type B500B were chosen. The resistance against axial load and bending moment including second order effects was calculated according to the principles in Section 4.1. All the concrete columns were assumed to be quadratic with reinforcement arranged according to Figure 26. Dimensions of the steel columns, HEA and VKR, were taken directly from tables from Tibnor AB (2011). The dimensions were chosen with regard to buckling in the weak direction.

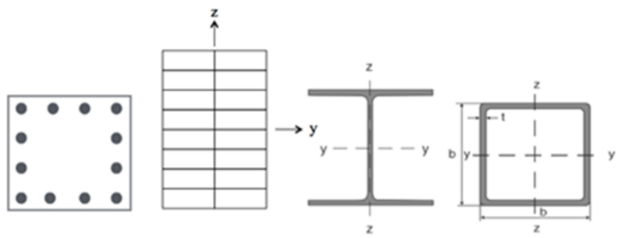


Figure 26 Cross-sections of the different columns investigated, reinforced concrete, glulam, HEA/HEB and VKR.

Concerning resistance against fire, steel columns were covered by fire gypsum boards on all four sides. For vertical loads of 0.5 MN and 1.0 MN the dimensions had to be increased with regard to fire for the concrete and glulam columns. The concrete columns have a cover thickness of 50 mm and the utilisation ratio for load case fire is around 0.5. According to Thor (2012) the minimum column width of 300 and a cover thickness of 45 mm are enough to satisfy the demands in class R90.

A summation of the dimensions for the different columns is presented in Table 3. The size of the VKR columns is the smallest for all load cases. For a vertical load up to 2 MN the dimensions of glulam, HEA and concrete columns are approximately the same, but then the dimensions of the glulam columns become considerably larger. However, for the HEA columns the dimensions increase fast with increasing loads. This is since Tibnor AB (2011) does not provide widths larger than 300 mm. The dimensions of the VKR-profile increases up to 5 MN and then the dimensions decreases, since thicker profiles are chosen and fewer gypsum boars are necessary according to Gyproc (2010). If cross section area is the decisive parameter, the same profile used for 6 MN can be used for 5 MN. Appendix A1 presents the calculations performed for the timber columns. In Table 3 some dimensions are followed by an (f), which means that the load case of fire was the desicive design situation.

Table 3 Summation of the needed dimensions for different columns depending on the vertical design load.

Vertical loads	Glulam L40c	HEA S355	VKR S355	Concrete C30/37
0.5 MN	280×270 (f)	222×214	162×162	300 ×300 (f)
1.0 MN	330×270 (f)	262×252	212×212	300 ×300 (f)
2.0 MN	330×360	322×312	242×242	324×324
3.0 MN	430×405	362×352	312×312	374×374
4.0 MN	430×540	362×452	312×312	412×412
5.0 MN	570×540	362×502	412×412	458×458
6.0 MN	645×540	362×652	331×331	495×495
7.0 MN	645×630	362×852	381×381	534×534

According to the results from the component study, the timber columns have the largest dimension for almost all design loads investigated. It is thereby hard to advocate timber columns just by studying the table, since a larger cross section area may result in a reduced open area in the building. In order to visualise the spatial consequences of using timber instead of concrete or steel the percentage of the column area in a fictive building was investigated.

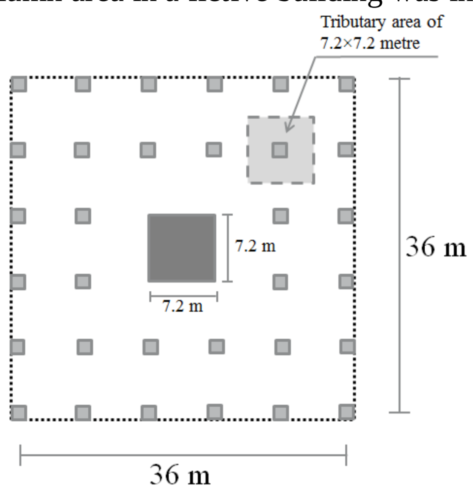


Figure 27 Layout of the fictive building.

The sides of the fictive building measures 36 metres and the height of one floor is 3.6 metres. Columns are assumed to be placed according to Figure 27, with a spacing of 7.2 metres. In the middle of the building there is a central core that measures 7.2×7.2 metres. The total floor area is1244 m^2 with 32 columns in total. The load acting on a column with a tributary area of 7.2×7.2 was calculated for different storeys and values for the dimensions could then be taken directly from Table 3. In Table 4 the total area of the columns on different storeys are presented. In Table 5 the area that the column sections cover is presented in percentage of the total floor area.

Table 4 Total area of the columns in the fictive building on different storeys.

Columns area	Corresponding load	Glulam [m^2]	HEA [m^2]	VKR [m^2]	Concrete [m^2]
1st floor	6 MN	11.2	7.6	3.5	7.8
5th floor	4 MN	7.4	5.2	3.1	5.4
10th floor	2 MN	3.8	3.2	1.9	3.4
13th floor	1 MN	2.9	2.1	1.4	2.9

Table 5 Percentage of the column area in the fictive building on different storeys.

Percentage of floor area	Corresponding load	Glulam [%]	HEA [%]	VKR [%]	Concrete [%]
1st floor	6 MN	0.90	0.61	0.28	0.63
5th floor	4 MN	0.60	0.42	0.25	0.44
10th floor	2 MN	0.31	0.26	0.15	0.27
13th floor	1 MN	0.23	0.17	0.12	0.23

As can be seen in Table 4, VKR-profiles are the most areal effective columns and timber columns are the least areal effective. If only comparing the total area, it is hard to advocate timber columns. However, by studying Table 5, it can be seen that the area that timber columns cover is still small compared to the total area, less than 1 %. It can also be seen that the difference decreases for the higher part of the building, where the loads on the columns are lower.

5.3 Beams

The beams were designed to resist the loads from floors within an influence width of 6 and 10 metres respectively, see Figure 28. In order to obtain the worst case with regard to deflections, moment and shear force, the beams were assumed to be simply supported. Five different spans were evaluated for each influence width; 4, 6, 8, 10 and 12 metres. For the influence width of 6 metres the beams were designed for an evenly distributed load of 45 kN/m and for the influence width of 10 metres the design load from the floor was 75 kN/m. The self-weight of the beams were added to these loads individually.

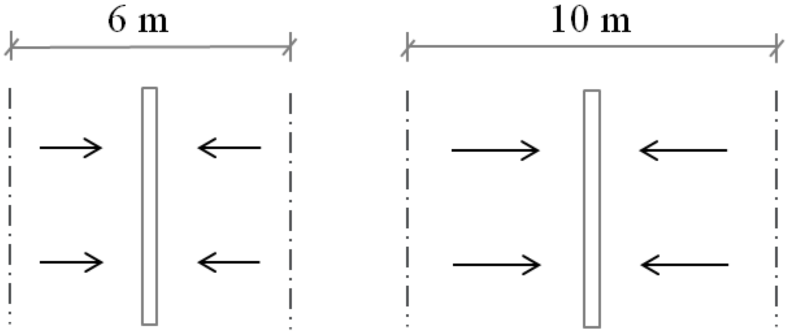


Figure 28 Assumed width of the influence area of the floor for which the beams were designed.

The dimensions of the timber beams were calculated according to the principles described in Section 4.3, with regard to deflections, moment and shear capacities. The limit for deflections was set to the span divided by 400 for all the beams investigated. Two different timber materials were investigated, glulam L40c and LVL Kerto-S. Dimensions of the steel beams, HEA and HEB, were on the other hand obtained from tables from Tibnor AB (2011) as for the steel columns. Prestressed concrete beams were designed on the basis of diagrams from Svensk Betong (2015a). In Appendices A2a and A2b the calculations performed in the design of the timber beams are presented.

For the beams three sides were assumed to be subjected to fire. Four timber beams needed increased dimensions due to fire; all of them were beams with a height-to-width ratio between four and five. The problem for these beams was that their widths were too small and they did thereby not fulfil the criterion for the load case fire. Therefore their widths were increased; giving height-to-width ratios that somewhat deviated from the ratio that is usual.

The steel beams were assumed to be covered with one or two gypsum boards depending on the circumference and thickness of the material. Dimensions of the concrete beams were taken from Svensk Betong (2015a) and therefore assumed to be designed also with regard to fire.

The timber and concrete beams were designed with a rectangular cross sections and the steel beams with an I-section. HEB steel beams have thicker flanges than HEA beams. For illustrations of the cross-sections, see Figure 26. The results for the beams are presented in Table 6 and Table 7, giving the total heights and widths of the beams.

As can be seen in Table 6 and Table 7, there are two sets of values for the glulam beams and the LVL beams. The beams referred to as Glulam I and LVL I are optimised with regard to the structural height of the floor. However, it is more common to design timber beams with a height-to-width ratio around four and five. Therefore, beams within this ratio were dimensioned as well and referred to as Glulam II and LVL II.

Table 6 Summation of needed beam dimensions for an influence width of 10 metres.

Span	Glulam I	Glulam II	LVL I	LVL II	Concrete	HEB	HEA
4 m	495×330	810×190	430×300	800×225(f)	400×200	291×322	301×342
6 m	630×380	1080×215	600×375	850×225	500×300	391×362	405×331
8 m	810×380	1125×280	800×375	1120×225	600×400	515×331	465×331
10 m	990×430	1395×280	940×450	1100×300	700×400	665×331	705×331
12 m	1170×430	1440×330	1130×450	1290×300	800×400	815×331	915×331

Table 7 Summation of needed beam dimensions for an influence width of 6 metres.

Span	Glulam I	Glulam II	LVL I	LVL II	Concrete	HEB	HEA
4 m	405×230	675×165(f)	400×225	500×225(f)	300×200	270×302	281×322
6 m	540×330	765×190	540×300	750×225(f)	500×200	331×362	361×362
8 m	720×330	855×215	720×300	800×225	500×300	465×331	455×331
10 m	855×380	1035×230	840×375	1000×225	600×400	565×331	605×331
12 m	1035×380	1170×280	1020×375	1200×225	700×400	665×331	705×331

According to Table 6 and Table 7 the HEB beams provide the smallest cross-sections and lowest heights for all spans. However, concrete beams are comparable with steel for longer spans. The timber beams are significantly larger for all spans and influence lengths. However, the total structural heights of the floors depend on how the beams are integrated into the floor and the beam/floor connection. More information about this is given in Section 5.5. If timber beams are to be used the span should be limited so that the height of the beams do not become too high. It may indicate that timber beams are more suitable for residential buildings than for office buildings for example. Office buildings usually have demands of open floor plans and hence longer spans are often a necessity in comparison to residential buildings that more often have shorter spans.

For the timber beams it was the shear capacity that was decisive. LVL-beams could have some smaller cross-sections than glulam beams. The dimensions of the steel beams were governed by the deflections. In Table 6 and Table 7 some dimensions are followed by an (f), which means that the fire was the decisive design situation.

5.4 Floor elements

Four different types of floor elements were investigated; timber cassette floors, concrete hollow core floors, concrete TT-floors and timber-concrete composite floors. Cassette floors were investigated instead of light weight floors and plane element floors, since they are able to span longer. In addition, the total height of a cassette floor is often lower than for a plane element floor, since the insulation and some installations can be hidden between the webs. The timber-concrete composite floor was designed according to the method described in Section 4.4 and the other floors types were designed with help of diagrams and tables from Martinsons (2006) and Svensk Betong (2015b). The resulting height for each floor type and for the spans, 6, 8, 10 and 12 metres is presented in Table 8.

Table 8 Floor height for different spans.

Span	Cassette [mm]	Timber/concrete [mm]	HD/F [mm]	TT/F [mm]
6 m	260	290	200	200
8 m	370	390	200	200
10 m	480	550	270	300
12 m	600	725	270	400

A decisive parameter when designing timber-concrete composite floors is the effectiveness of the connection at the joint interface, which is calculated using the slip modulus of the connectors and their spacing. The value of the slip modulus depends on what kinds of timber product and connections that are used. Linden (1999) provides some values for the slip modulus for different cases. In this project nail plates and sawn timber were assumed. The smallest slip modulus resulted in an effectiveness of the connections of 0.90 and the highest value of the slip modulus gave 0.96. Therefore an average value of 0.93 was assumed.

The composite floor was designed with a limit of the fundamental frequency of 7 Hz. Part 1-1 in Eurocode 5, CEN (2009), states 8 Hz for timber floors, but since a timber concrete composite floor has a higher weight, the limit of the fundamental frequency was set to 7 Hz. All composite floors were designed with a concrete slab with a thickness of 70 mm. In Appendix A3 the calculations regarding the timber-concrete composite floors are presented.

According to Martinsons (2006), their cassette floors have a fundamental frequency between 8-10 Hz. The maximum deflection for a static distributed load is between L/400 and L/600.

Another aspect that needs to be considered is the needed room for installations. Usually an extra height of 400-600 mm is needed. In the case of hollow core slabs all this extra height is needed to be placed underneath the floor elements. For cassette timber floors, timber-concrete composite floors and concrete floors with TT-section, some of the installations can be placed in the spaces between the webs. If assuming that 100 mm of the installations can be placed within the floor height and that in total 500 mm is needed for the installations, the following structural heights for the floor elements are obtained, see Table 9.

Table 9 Total height including installations and insulation for different spans.

Span	Cassette	Timber/concrete	HD/F	TT/F
6 m	660	685	700	600
8 m	770	790	700	600
10 m	880	950	770	700
12 m	1000	1125	770	800

These heights are also assumed to be sufficient with respect to the requirements for airborne sound insulation and fire protection. In Martinssons, (2006) it is stated that the cassette floors should have a height between 400-1000 mm, with respect to sound insulation and fire safety, depending on different solutions. As described in Section

4.7, Hagberg (2010) states that in the preliminary design stage timber floors should be assumed to have a height of 500 mm.

One reflection made after studying Table 9 is that the difference between the heights of the floor structures increases with longer spans. Concrete floor structures are more efficient regarding height for longer spans than timber floor structures, but if studying spans in the order of 8-9 metres, timber floor structures do not result in palpable higher heights than concrete floor structures. Moelven is using timber floors that spans 8 metres in their structural system called Trä8 (Moelven Töreboda AB, 2015). Martinsons (2006) is stating that their floors can be used up to 12 metres. However, the height of the cassette floors Martinsons uses for 12 metres is noticeable higher than the concrete alternatives.

Timber-concrete composite floors were in this project not chosen as a promising floor structure. The in Sweden more common cassette timber floor was instead used in the development of structural systems. The timber cassette floors can provide lower heights than the timber-concrete composite floor. In addition the timber-concrete composite floors were designed with the assumption that the concrete is prefabricated. This type of timber-concrete floor is not available on the Swedish market today.

However, it should be noted that the material strengths of the timber and concrete parts of the composite floor elements designed in this project could have been higher. Thereby the height of the floor elements would have been decreased. Timber and concrete with lower strength classes were chosen, since Linden (1999) states that the model used is only applicable if the timber and concrete members are of ordinary strength classes. If higher strength classes are to be used, nonlinear behaviour must be considered.

5.5 Integration of floor elements and support beams

The longer, spans the higher beams and floor structures are needed in order to fulfil requirements in the serviceability limit state and the ultimate limit state. As can be seen in Table 6, Table 7 and Table 9 the structural height becomes high in dependently of the material, especially when placing floor elements on top of beams.

One solution to this is to use HSQ-beams, where the floor is placed on the bottom flange, see Figure 29a and c. The HSQ-beam can be combined with concrete or timber floor elements. Another solution is to provide a heel on a glulam or LVL beam with the floor element on top of it, see Figure 29c. A third solution can be seen in Figure 29d, where a projecting upper flange of the floor rests on top of the beam. It can be necessary to make holes in the beam for Figure 29c and Figure 29d in order to allow for installations.

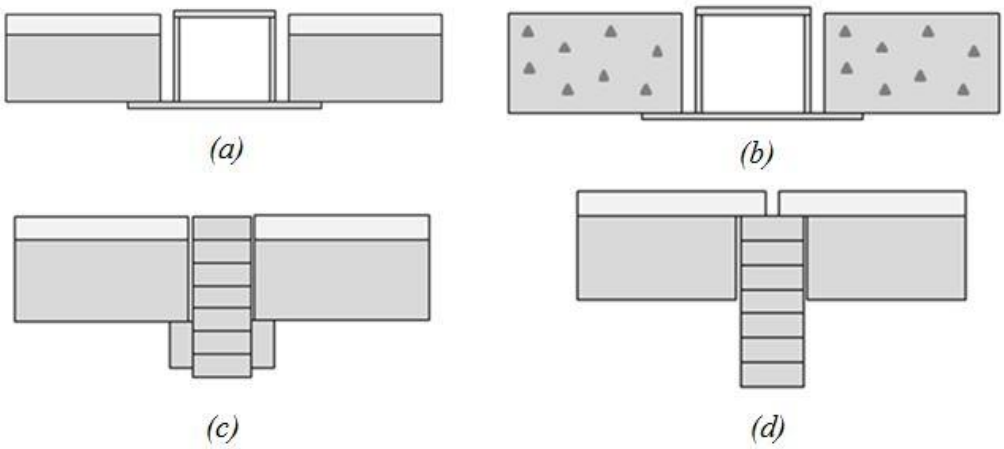


Figure 29 Different solutions to reduce the structural height of the floor. (a) and (b) is using HSQ-beams, (c) is using heels that are mechanically fastened to the beam and (d) is showing a timber floor using its upper flange to rest on the beam.

5.6 Walls

The thickness of the wall was calculated for concrete and cross laminated timber for four different load situations. It was assumed that the walls have an insulation of 200 mm and an extra cover outside the insulation of 50 mm that do not contribute to the vertical load bearing capacity. These extra layers, however, contributes to the weight of the walls. Parameters that change are the width of the influencing floor area which varies between 4 and 6 metres, and the length between the windows which varies between 0.8 and 1.0 metres, see Figure 30. The size of the windows was set to 1.7×1.6 metres and the height of the wall is 3.6 metres.

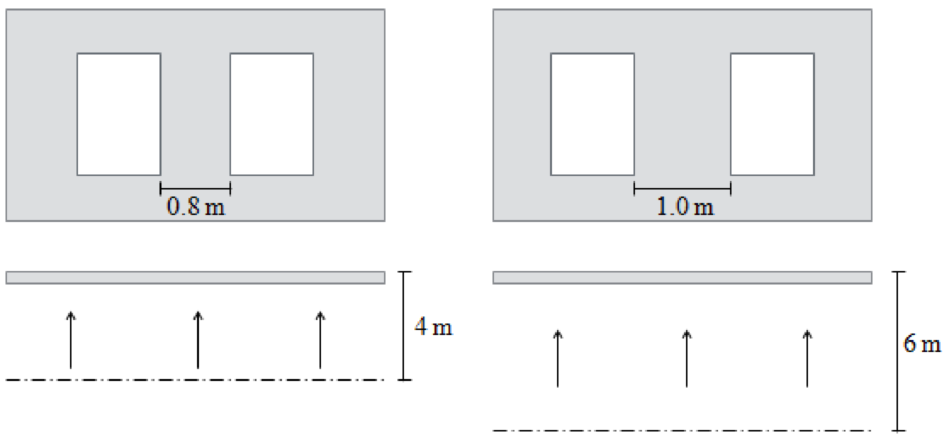


Figure 30 Walls and the influence widths investigated.

The load bearing capacity of the wall was checked for a wall on the first, sixth and eleventh floor in a fictitious 15 storey building according to the calculation principles described in Section 4.5. This was in order to cover the dimensions needed for different loads, since the vertical loads accumulate through the structure and the effect from the wind load increases with height.

For the timber walls cross laminated timber was chosen and the capacity was calculated with respect to material data from Martinsons (2014). Figure 31 shows the cross-sections of cross laminated timber (CLT) walls with different thicknesses.

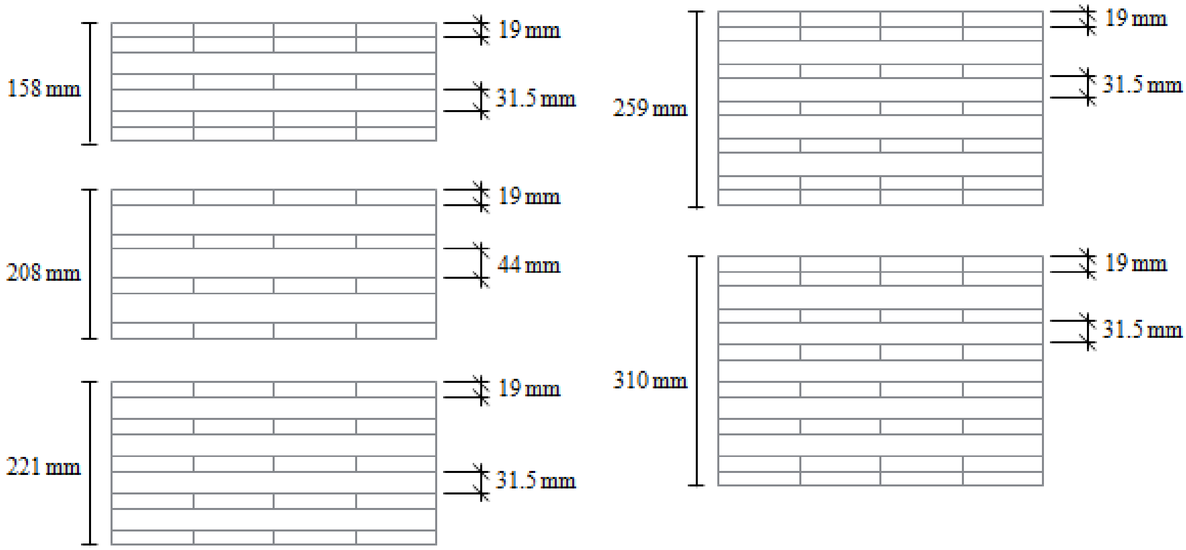


Figure 31 Cross-sections of CLT-walls with different thicknesses.

The strength class of the concrete was chosen as C35/45 with 12 mm vertical reinforcement bars. Figure 32 illustrates the cross-section of the column part of the wall between the window openings. The part of the concrete wall above the opening was designed as a deep beam, using the strut and tie method to determine the needed amount of reinforcement. The thickness was then checked to be sufficient to cover all reinforcement necessary with a minimum cover of 30 mm.

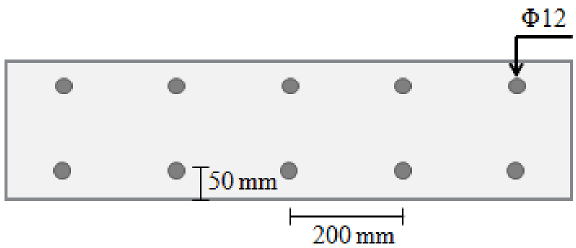


Figure 32 Cross-section of the column part of the concrete wall.

For all load situations the capacity of the column part of the wall was designed for both materials. Imposed load was the main load in the design load combination. The performance of the beam part proved to have a small influence on the overall load bearing capacity. The highest utilisation ratio for the timber beam (15.7 %) occurs, when the column part of the cross laminated timber wall has an utilisation ratio of 95.2 %. In Table 10, Table 11, Table 12 and Table 13 the results from the analysis for the load bearing parts of the walls are presented. Calculations are presented in Appendix A4.

Table 10 Needed thickness of walls with an influence width of 4 metres and 0.8 metres between the windows.

Influence width 4 m Length between windows 0.8 m	Concrete [mm]	Timber [mm]
1st floor	220	274
6th floor	185	236
11th floor	140	173

Table 11 Needed thickness of walls with an influence width of 4 metres and 1.0 metres between the windows.

Influence width 4 m Length between windows 1.0 m	Concrete [mm]	Timber [mm]
1st floor	205	274
6th floor	175	223
11th floor	140 (f)	173

Table 12 Needed thickness of walls with an influence width of 6 metres and 0.8 metres between the windows.

Influence width 6 m Length between windows 0.8 m	Concrete [mm]	Timber [mm]
1st floor	240	325
6th floor	205	274
11th floor	155	173

Table 13 Needed thickness of walls with an influence width of 6 metres and 1.0 metres between the windows.

Influence width 6 m Length between windows 1.0 m	Concrete [mm]	Timber [mm]
1st floor	225	274
6th floor	190	274
11th floor	145	173

The relation between the design loads in the load case fire and in the ultimate limit state is around 0.7. According to Thor (2012) a concrete wall exposed to fire at one side with class REI90 and an utilisation ratio of 0.7 requires a thickness of 140 mm and a cover thickness of 25 mm. This only affects the concrete walls on the 11th to the 14th floor in Table 11. When designing this wall with respect to the resistance in the ultimate limit state the required thickness is 135 mm.

The timber walls were calculated according to the method described in Section 4.5. It was found that some walls did not fulfil fire safety demands but the utilisation ratio in the fire case was close to 1. Therefore it is assumed that an extra fire gypsum board of 15.4 mm is enough to provide the fire safety class of REI90. This extra gypsum board was added to all timber walls.

As can be seen from the results the thickness of the concrete walls is thinner than for timber walls for all load cases. The largest difference occurs on the first floor when having an influence width of 6 metres and a distance of 0.8 metres between the windows. In this case the thickness of the concrete is 26 % less than the thickness of the timber wall. Even though the timber wall elements become thicker than the concrete wall elements, the floor area of a building do not need to be reduced, because the walls can in the planning stage be moved somewhat further out, compensating for the extra thickness.

5.7 Bracing components

Steel and timber bracing diagonals were designed with regard to wind loads. The wind load was calculated for a fictitious 15-storey building with a total height of 54 metres. The cross-section of the building was assumed to be rectangular, where each side measured 36 metres.

Two configurations of centric bracing diagonals were evaluated, single diagonal bracing and chevron bracing. The dimensions were calculated assuming that the bracing diagonals extend over one storey and a width of 6 metres and 9 metres respectively, see Figure 33.

For the component study, the diagonal's ability to resist the load effect of lateral loads was considered. The global performance of the complete bracing units, such as compression in the beam and axial forces in the columns was not examined.

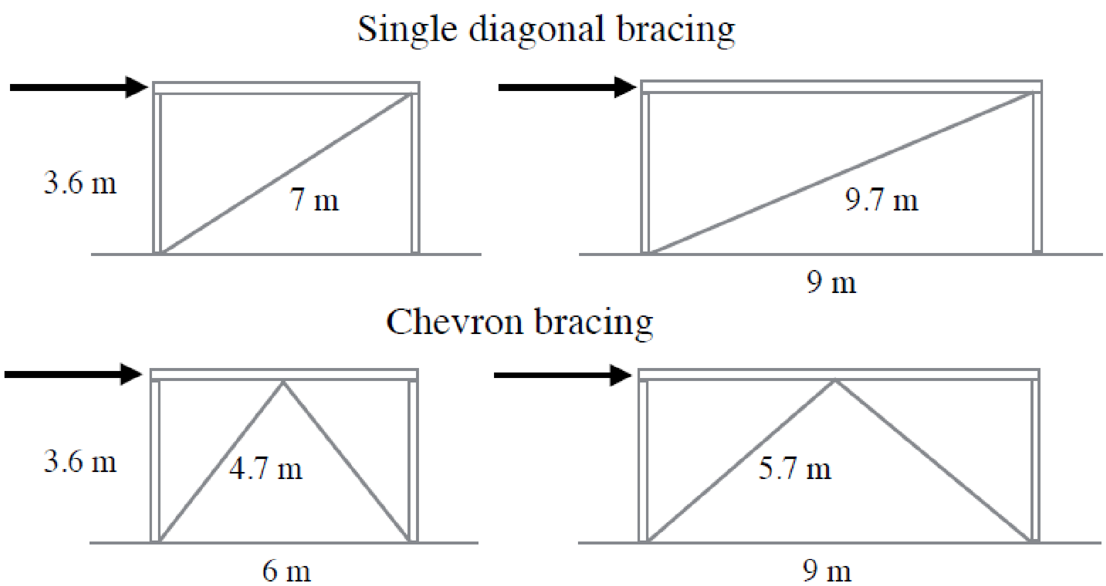


Figure 33 Single diagonal bracing and chevron bracing.

Since single diagonal bracing and chevron bracing can resist horizontal forces in both compression and tension the capacity of both has been checked. The capacity has been checked for bracing on the first, fourth, seventh, tenth and thirteenth floor of the fictitious building. This was in order to cover the dimensions needed for different loads since the lateral loads accumulate through the structure.

The dimensions of the timber bracing diagonals were calculated in accordance with part 1-1 in Eurocode 5, CEN (2009). For the timber bracing, glulam with the strength class GL30h and standardised sizes of the lamellas were used. Dimensions for the steel bracing, VKR and KCKR profiles with steel quality S355, were taken directly from tables from Tibnor AB (2011), see Figure 34. Interpolation between the buckling capacities was carried out in cases where the buckling length did not match with the standard lengths in the tables. The tensile capacity of the steel bracing was calculated according to part 1-1 in Eurocode 3, CEN (2008a).

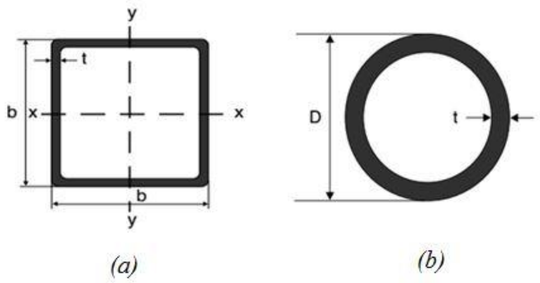


Figure 34 Different steel profiles, (a) VKR-profile (b) KCKR-profile

In Table 14 and Table 15 the dimensions of the diagonal members in a single diagonal bracing unit are presented. For single diagonal bracings the timber members that had the smallest dimensions were governed by fire, in total three of them needed to get an increased cross-section due to fire safety. The single diagonal bracings made of steel have to be provided with two or three fire gypsum boards on each side. Appendix A5 presents performed calculations for both single diagonal and chevron bracing members.

Table 14 Needed dimensions for single diagonal bracing with a buckling length of 7 metres. The dimensions of the steel profiles include the gypsum boards.

Compressive force	Storey	Glulam	VKR	KCKR
287 kN	13	230×315 (f)	233×233, 5	230, 6
626 kN	10	280×315 (f)	212×212, 10	255, 10
933 kN	7	330×315	242×242, 10	281, 10
1240 kN	4	330×315	262×262, 10	306, 10
1547 kN	1	330×360	262×262, 12.5	335, 10

Table 15 Needed dimensions for single diagonal bracing with a buckling length of 9.7 metres. The dimensions of the steel profiles include the gypsum boards.

Compressive force	Storey	Glulam	VKR	KCKR
265 kN	13	280×360 (f)	242×242, 6.3	230, 8
578 kN	10	330×315	242×242, 10	306, 8
862 kN	7	330×360	262×262, 12.5	335, 10
1145 kN	4	380×360	262×262, 16	335, 12.5
1429 kN	1	380×405	312×312, 10	386, 10

In Table 16 and Table 17 the dimensions for the diagonal members in a chevron bracing unit are presented. All timber chevron bracings were governed by the fire load combination. The dimensions were increased due to this. In the same way as for single diagonal bracings, steel chevron bracings had to be provided with two or three fire gypsum boards on each side to be able to fulfil the fire safety class REI90. The amount of fire gypsum boards depends on the cross-sectional area and the thickness of the member. Therefore the second VKR-profile in Table 16 and the first VKR-profile in Table 17 gets a larger dimension. The KCKR-profiles that have a circular profile are assumed to have the same thickness of fire gypsum board as the corresponding VKR-profile.

Table 16 Dimensions for chevron bracing with a buckling length of 4.7 metres. The dimensions of the steel profiles include the gypsum boards.

Compressive force	Storey	Glulam	VKR	KCKR
192 kN	13	215×225 (f)	192×192, 5	201, 4
419 kN	10	230×270 (f)	212×212, 6.3	201, 8
625 kN	7	230×315 (f)	202×202, 6.3	230, 8
830 kN	4	280×270 (f)	202×202, 8	255, 8
1036 kN	1	330×270 (f)	212×212, 10	255, 10

Table 17 Dimensions for chevron bracing with a buckling length of 5.7 metres. The dimensions of the steel profiles include the gypsum boards.

Compressive force	Storey	Glulam	VKR	KCKR
158 kN	13	215×225 (f)	212×212, 4.5	201, 4
344 kN	10	230×270 (f)	182×182, 8	230, 6
512 kN	7	280×270 (f)	202×202, 8	230, 8
681 kN	4	280×270 (f)	202×202, 10	255, 8
850 kN	1	330×270 (f)	212×212, 10	255, 10

The single diagonal bracings have similar dimensions as the diagonals used in the building Treet, see Section 2.4.5. However, there are some differences between how Treet's diagonals were designed and the design used in this project. In Treet the diagonals were designed to carry only tension, with a maximum tensile force of 930 kN (Abrahamsen & Malo, 2014). In this project the diagonals were also designed to resist compression. This leads to that the diagonals in Treet are able to span over two storeys, while in this project they span over one storey to provide a shorter buckling length.

It can be concluded that the chevron diagonal bracings have somewhat smaller dimensions than the single diagonal bracings. This is because the buckling length is shorter for the chevron diagonal bracings than for the single. The dimensions of all the diagonals are such that they may be hard to hide inside of walls.

6 Development of Structural Systems

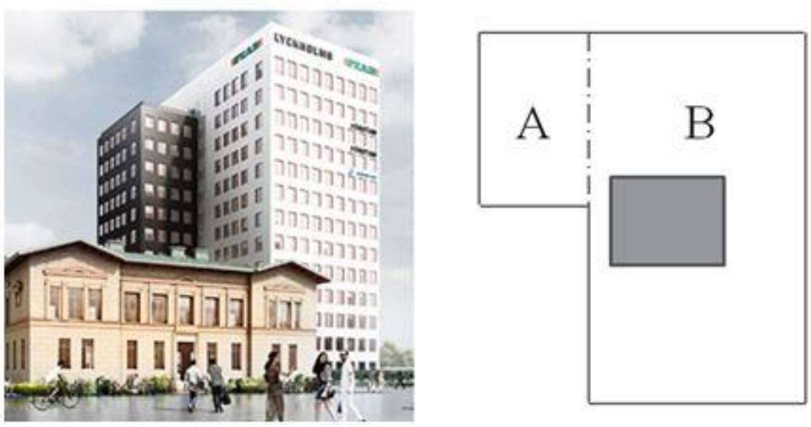
This chapter presents the results from the development of mixed structural systems. The chapter is introduced with a description of the reference building that was used as a benchmark when developing the structural systems. Thereafter the methodology, conditions and demands used in the development are described followed by a description of six different mixed systems that were developed. These systems are then summarised and evaluated in a concluding section.

6.1 Description of the reference building

The reference building worked as a benchmark in the development of the mixed structural systems. Everything presented in this section describes the existing building.

6.1.1 Introduction to the reference building

The reference building used in this project is Lyckholms, situated in Göteborg south of Liseberg in the district Lyckholms see Figure 35a. It is a 14 storey high building whereof 13 storeys are used for offices and the 14th storey contains installations. The building has 11 000 m^2 office area. Each storey has an area of 860 m^2 except the two top floors, which are somewhat smaller. In Figure 35b the plan of the building is illustrated. The basement and all storeys up to the twelfth storey contain both part A and part B while the thirteenth and fourteenth storeys only contain part B. On top of part A, at the 13th storey, there is a terrace. The beam supporting the terrace is further on referred to as the 'balcony beam'.



(a) *(b)*

Figure 35 The reference building Lyckholms, (a) the existing building (b) plan of the building and notation for the different parts of the building where part A reaches the 12th storey and part B reaches the 14th storey.

The height of the storeys differs along the building according to Table 18. This table also shows how the area differs between the storeys. Most of the storeys have a total height of 3.6 metres and a free height of approximately 2.8 metres. The office areas in the building are open and flexible in order to suit different demands. Therefore all the inner walls are possible to move.

Table 18 Height of each storey and which parts, according to Figure 35b, the storey contains.

	Height of storey	Contains area
Basement	3.15 m	A+B
1st storey	4.3 m	A+B
2nd-11th storey	3.6 m	A+B
12th storey	4.0 m	A+B
13th storey	3.6 m	B
14th storey	4.75 m	B

According to the architectural drawings all storeys contain open office areas, office rooms, conference rooms and a kitchen area, see Appendix B. The plan arrangement from the architectural drawings was used in the development of alternative structural systems.

6.1.2 Loads acting on the reference building

Table 19 shows the loads that were taken into account by the structural engineer of the structural system for Lyckholms. The structural system was designed assuming safety class 3 and a service life of 100 years.

Table 19 Characteristic values for loads and combination factors used by Integra when designing the reference building.

	Load	Ψ_0	Ψ_1	Ψ_2
Imposed load				
Basement	2.5 kN/m^2	0.7	0.7	0.6
Floor structure in office areas	2.5 kN/m^2	0.7	0.5	0.3
Floor structure on entry-level	4.0 kN/m^2	0.7	0.7	0.6
Floor structure on the terrace	5.0 kN/m^2	0.7	0.7	0.6
Partition walls	0.5 kN/m^2			
Permanent loads				
Weight of structural components	-			
Installations	0.3 kN/m^2			
Wind		0.6	0.2	0
Snow (ground value, S_0)	1.5 kN/m^2	0.7	0.5	0.2
Accidental load				
Column on ground level, x-dir.	150 kN			
Column on ground level, y-dir.	75 kN			
Column in garage, x-dir.	50 kN			
Column in garage, y-dir.	25 kN			

The weight of structural components is not presented in Table 19 due to the variation between different members. The weight of installations was assumed to be 0.3 kN/m^2 and of partition walls to be 0.5 kN/m^2.

The imposed loads vary between different storeys of the building. For the basement and the floors of the office areas the imposed load is 2.5 kN/m^2. The floor on the entry-level was designed for an imposed load of 4 kN/m^2 and the floor structure on the terrace was designed for 5 kN/m^2.

Since the building is situated in Göteborg the reference wind velocity is 25 m/s and the building was designed assuming terrain type III. The snow zone is 1.5 with a ground value for the snow load of 1.5 kN/m^2.

In addition, the columns were designed for accidental load. The columns on the ground level were designed for collision by truck and the columns in the basement were designed for collision by cars. In the case of fire all the load bearing parts must fulfil fire class EI60.

6.1.3 The structural system of the reference building

The structural system of Lyckholms was designed by the consultant company Integra and consists of both exterior load bearing walls and a beam-column system. The building is stabilised by an inner concrete core. The outer walls could be part of the stabilising system due to their considerable in-plane stiffness. However, on the bottom floor the walls are supported by columns which cannot be considered to contribute to the stabilising system. Therefore, the outer walls are not contributing to the lateral stability.

There are two beam lines, which together with the exterior walls are supporting the floors, see Figure 36. The largest floor span in the building is 10.8 metres. The beams are supported by exterior walls or columns. Almost all the columns are made of steel, see Figure 37a. In the basement however, concrete columns are used, see Table 20. All walls, including the stabilising core, are prefabricated. However, the basement walls and floors are cast in situ.

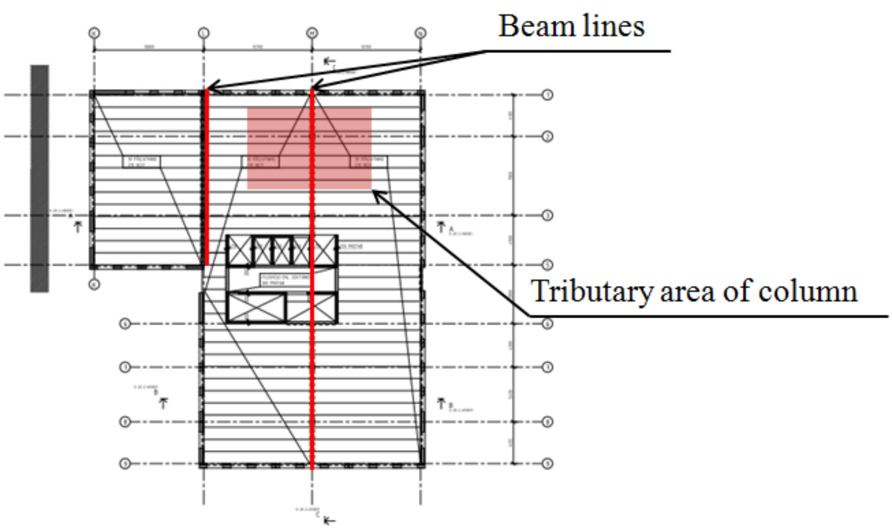
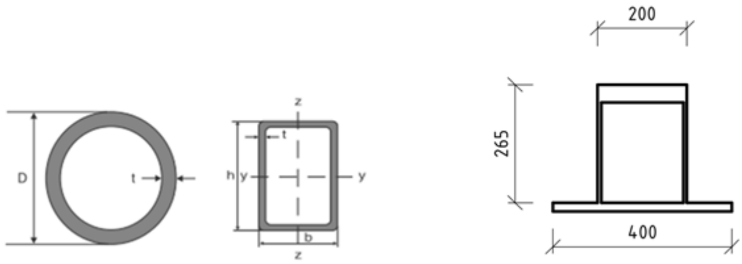


Figure 36 Floor plan with beam lines and the largest tributary area in the reference building.



(a) *(b)*

Figure 37 Different steel profiles used in the reference building for the columns and the beams (a)Steel columns (b) Steel beam

Table 20 Type of column used on different storeys in the reference building.

Storey	Column type
Basement, storey 0	Concrete, 600×600
Entry-level, storey 1	Steel, K-CKR 470
Storey 2-8	Steel, VKR 400×400
Storey 9-12	Steel, VKR 300×300
Storey 13	Steel, K-CKR 193.7, VKR 200×200

The beams are carrying load from an influence area with a maximum width of 10.75 metres and the maximum span for the beams is 7.8 metres. All beams have a cross section according to Figure 37b. The tributary area of the columns is largest for the column that supports the beam with the largest span, 63.67 m^2, see Figure 36.

The floor consists of hollow-core elements spanning in one direction. For all floors except the 14^{th} floor HD/F 120/27, which has a height of 265 mm, is used. On the 14^{th} floor HD/F 120/32 with a height of 320 mm is used. The steel beams are integrated in the floor.

Drawings of the reference building are shown in Appendix B.

6.2 Methodology in the development of structural systems

Six alternative concepts were developed based on the drawings of the reference building, the literature study and the component study. The architectural drawings gave a proposed layout of offices and open spaces and thereby also the activity of the reference building. These drawings were used as a benchmark when replacing columns in different concepts. Columns should not interfere with the layout of the building.

6.2.1 Development of components in the concepts

As a limitation the same stabilising core as in the reference building was used for all the concepts. Other stabilising systems are not investigated in this project.

The dimensions and type of columns used in the basement and the entrance floor were chosen to be the same as for the reference building for all concepts. These columns need to be designed with respect to collision and as a limitation of this master's project, this was not treated. Also the roof was chosen to be the same as for the reference building.

The results of the component study were used when designing the components of the structural system. When performing the floor design the height of the cassette floors was obtained by interpolating between the different heights in the component study. This method was used, since the height of the floors changes linearly with the span according to Martinsons (2006). The weight of the floors is also changing linearly with the span and could therefore be calculated in the same manner. Cassette floors were chosen due to their lower height compared to timber-concrete composite floors. In addition timber-concrete composite floors are still uncommon in Sweden and would therefore be hard to advocate. Concrete floors were not chosen, since the aim was to implement as much timber as possible and, concrete has a higher weight, which results in higher loads on columns and beams.

The beams used in the concepts were chosen to be LVL beams or HEA beams. LVL beams were chosen, because they had some lower height than glulam beams according to the component study. HEA beams on the other hand are higher than HEB beams, but the differences are not large and HEA beams are usually less expensive than HEB beams. Therefore HEA beams were chosen. In all tables with dimensions of beams in the following sections, 'balcony beam' refers to the beam in the building part A, see Figure 35b, on the 12th storey. Above this storey the balcony is located and this beam is therefore resisting balcony load. The other beams are referred to as 'office beam', besides the beam on the 14th storey, which is referred to as roof beam.

When designing beams the dimensions were not directly taken from the component study, because the loads and the spans differ in the concepts compared to those investigated. However, the component study was used as a guideline to provide an indication of which dimensions that are suitable. The concepts include both timber and steel beams. Timber beams were designed with the calculation procedure presented in Section 4.3 and Appendix A2a and A2b, which are the same as for the component study. Dimensions for the steel beams were also calculated in the same manner as in the component study, with tables from Tibnor AB (2011). The design considered the fire case as well as ULS and SLS.

The loads acting on a wall were calculated and then the component study was used to find a suitable dimension of the wall. However, the walls needed to be checked again with respect to fire. This was performed according to the calculation principle in Section 4.5.

Dimensions of the columns were taken directly from the component study. The design load for a column was first calculated and then a suitable dimension was found in the tables from the component study. Utilisation ratios for each column in each concept were calculated. For the timber columns Appendix A1 was used, and for steel columns tables from Tibnor AB (2011) were used.

All components were designed for the standard fire resistance R90. For the timber components the dimensions were made larger when needed and for the steel components additional gypsum boards were added on the exposed sides. The extra thickness obtained by these gypsum boards was determined in the same manner as in the component study.

6.3 Assumed conditions and demands for the structural system

The design of the structural systems was based on the following conditions.

- For the load combination in ULS, the imposed office load is the main load. This is the worst case for the building globally. For the roof beam, however, snow load is the main load.
- Characteristic snow load on roof: $s_r = 1.2\ \mathrm{kN/m^2}$
- Characteristic snow load acting on the terrace: $s_b = 1.5\ \mathrm{kN/m^2}$
- The roof of the reference building is covered by sedum, which is assumed to be a demand from the client. Therefore the same roof as for the reference building was assumed in all developed systems. The roof is composed of 320 mm prestressed hollow core elements, 300 mm cellular plastic and sedum.
- Weight of roof: $g_r = 4.56\ \mathrm{kN/m^2}$
- In order to simplify the calculation of the snow load the layout of the roof was been simplified, see Figure 38.

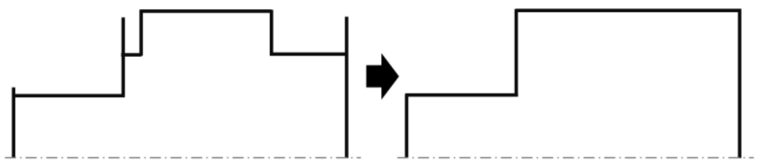


Figure 38 Simplification of the roof, left is roof in the reference building and right is the model used in calculations.

Dimensions for different components were calculated for the second, sixth and eleventh floor of the building. This was in order to optimise the components in the building and to better understand how the size for different components alters throughout the building. In Table 21 loads and coefficients used in the calculations are presented. The calculation principle for the loads used in the design of the different members is presented in Appendix C1 and C2.

Table 21 Characteristic loads acting on the building and used combination factors.

	Loads [kN/m²]	Ψ_0	Ψ_1	Ψ_2
Variable loads				
Snow	1.2	0.7	0.5	0.2
Imposed loads				
Office	2.5	0.7	0.5	-
Partition walls	0.5	0.7	0.5	0.3
Balcony	5	0.7	0.7	-
Permanent loads				
Roof	4.56			
Installations	0.3			

The following demands were considered in the development of the systems.

- The height of the building cannot be increased. A taller building will induce larger bending moments and shear forces in the stabilising core.
- According to Arbetsmiljöverket (2009) the free height in an open office building should not be less than 2.7 metres. The free height in the reference building is around 2.8 metres.
- No load bearing interior walls are accepted due to the demanded open floor plan.
- The number of storeys could not be changed. A fewer amount of storeys would result in less available area to rent and, hence, less income for the owner.

6.4 Concept 1

For Concept 1 all components except the stabilising core, the roof, members in the basement and the entrance floor were composed of timber. The first concept was developed in two iterations.

6.4.1 First iteration – layout from the reference building

In the first iteration the same layout of beam-column lines was used as in the reference building, see Figure 39a. In Table 22 the different members are presented with the materials that were used.

Table 22 Materials for different members used in Concept 1.

Components in the structural system		
Stabilising core	Concrete	Prefabricated
Roof	Concrete	Hollow core slab
Floors	Timber	Cassette floor from Martinsons (2006)
Beams	Timber	Kerto-S
Columns	Timber	Glulam, L40c
Walls	Timber	CLT from Martinsons (2014)

In the first iteration the layout was the same as for the reference building, hence the first iteration has a maximum

- floor span of 10.7 metres
- beam span of 7.8 metres

According to the component study the height of a cassette floor with a span of 10.7 metres would be around 900 mm and the height of the LVL-beams would be approximately 1120 mm. If trying to optimise the floor structure by connecting the floor to the beam with a heel attached to the side of the beam, see Figure 29c, the total height of the floor structures for the whole building would then be approximately 15 metres. In the reference building, HSQ-beams are used, see Figure 37b, and hollow core elements with a height of 270 mm. If assuming that an extra space for installations of 500 mm is necessary then the total height of the floors for the whole building would become 10 metres. This means that approximately five metres are lost, if timber is chosen for the beams and floor elements.

There are three alternatives of how to consider these extra five metres of materials. The first is to increase the height of the building, which according to the demands, is not possible. Secondly the free height of each floor could be decreased with 360 mm. However this results in a free height of approximately 2.4 metres, which is below the limit of 2.7 metres. The third and last alternative is to decrease the amount of storeys, which is not allowed. Therefore this alternative with timber floors, beams and columns in the same layout as the original reference building is not possible. Further development of the system was needed.

6.4.2 Second iteration – modified layout

The layout of the floor plan was modified by adding an extra beam-column line and changing the location of the columns according to Figure 39 and the list below. The change in the layout was made in accordance with the architectural drawings. The columns were placed so that they do not interfere with offices. However, one column was placed in the middle of an open area intended as a social area, see Figure 39. In Appendix C3 a more detailed drawing of Concept 1 is presented.

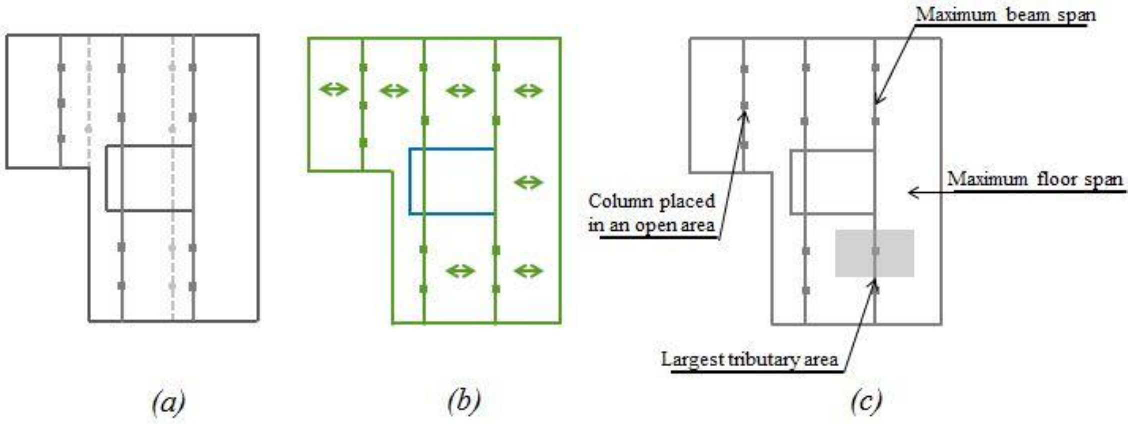


Figure 39 Floor plans of Concept 1 (a) dashed lines are showing the beam-column lines in the reference building while the solid lines are showing the beam-column lines used in Concept 1. (b) Floor plan that shows what materials that were used, green stands for timber, blue for concrete and red for steel. (c) Presents information like maximum span and the column which is placed in the open social area.

Concept 1 has a maximum

- floor span of 8.425 m
- beam span of 5.7 m
- tributary area for columns of 41.4 m^2
- influence width for the walls of 4.21 m

According to Martinsons (2006) the height of the cassette floor including installations and insulations was interpolated to 800 mm. The weight including insulation is 0.75 kN/m^2 and the ceiling and the topping has a weight of 0.25 kN/m^2; hence the total weight of the floor is 1 kN/m^2. The height of the floor excluding the installations is around 400 mm.

Dimensions for the beams are presented in Table 23. Compared to the first iteration these beams have a smaller height; the beam type mostly used have almost half the height of the beams in the first iteration.

Table 23 Needed dimensions of the beams in Concept 1 including fire protection.

Beams concept 1	Dimension [mm]	Span [m]	Influence width [m]	Load ULS [kN/m^2]	Utilisation [%]
Roof beam	800× 225	5.7	8.425	63.6	82.5
Balcony beam	800× 300	4.3	7.675	119.1	87.4
Office beams	650× 225	5.7	8.425	50.21	79.9

The columns needed for this alternative are presented in Table 24.

Table 24 Needed dimensions of the columns in Concept 1 including fire protection.

Storey	Dimension [mm]	Load ULS [MN]	Utilisation [%]
11th – 14th floor	330×270	1.00	64.8
6th – 10th floor	430×405	2.22	72.8
2nd – 5th floor	430×540	3.19	79.6

The wall elements have geometry according to Figure 40. The required dimensions for the walls in Concept 1 can be seen in Table 25.

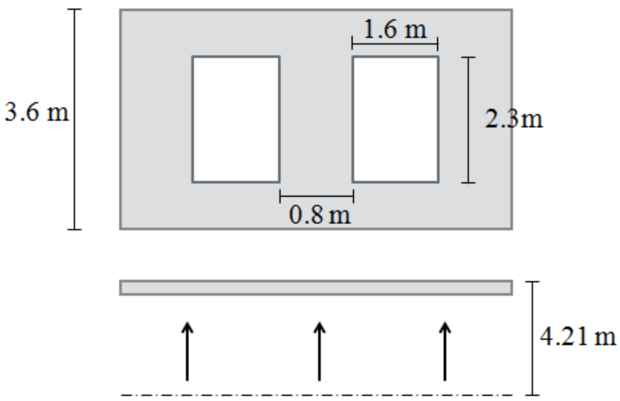


Figure 40 Dimensions of wall elements used in the concept and the influence width from loads applied on the floor.

Table 25 Needed dimensions of the walls used in Concept 1 including fire protection.

Storey	Thickness [mm]	Compressive force ULS [kN]	Horizontal force ULS [kN/m]	Utilisation [%]
2nd – 5th	259	885.7	2.86	56.9
6th – 10th	221	605.9	2.86	68.5
11th – 14th	158	265.3	3.27	58.0

6.5 Concept 2

In Concept 2 steel columns were used instead of timber columns. The rest of the elements and layout of columns and beams are the same as for the Concept 1, see Figure 41. The materials used can be seen in Table 26.

Table 26   Materials for different members used in Concept 2.

Components in the structural system		
Stabilising core	Concrete	Prefabricated
Roof	Concrete	Hollow core slab
Floors	Timber	Cassette floor from Martinsons (2006)
Beams	Timber	Kerto-S
Columns	Steel	VKR
Walls	Timber	CLT from Martinsons (2014)

Since the layout of Concept 2 is the same as in Concept 1 the maximum

- floor span is 8.425 m
- beam span is 5.7 m
- tributary area for columns is 41.4 m^2
- influence width for the walls is 4.21 m

In Appendix C3 a more detailed drawing of Concept 2 is presented.

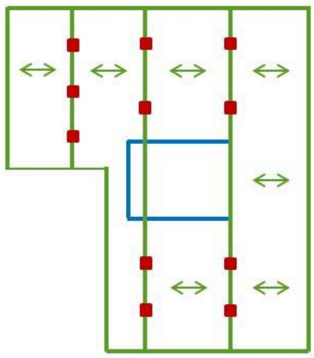


Figure 41   Floor plan that shows what materials that were used, green stands for timber, blue for concrete and red for steel.

The floor height and the dimensions of the timber beams and walls are the same as for Concept 1. Dimensions of the steel columns are presented in Table 27. Each column is protected by two fire gypsum boards on each side.

Table 27   Needed dimensions of the columns used in Concept 2 including fire protection.

Storey	Profile	Dimension [mm]	Load ULS [MN]	Utilisation [%]
11^{th} – 14^{th} floor	VKR150, 6.3	212×212	1.0	99.0
6^{th} – 10^{th} floor	VKR250, 10	312×312	2.24	71.6
2^{nd} – 5^{th} floor	VKR250, 16	312×312	3.24	67.1

6.6 Concept 3

In concept 3 the column in the open social area was removed. This was enabled by using steel beams in the left beam-column line. For the other beams timber was used.

In other words, both steel and timber were used for the beams in this concept, see Figure 42. In Appendix C3 a more detailed drawing of Concept 3 is presented.

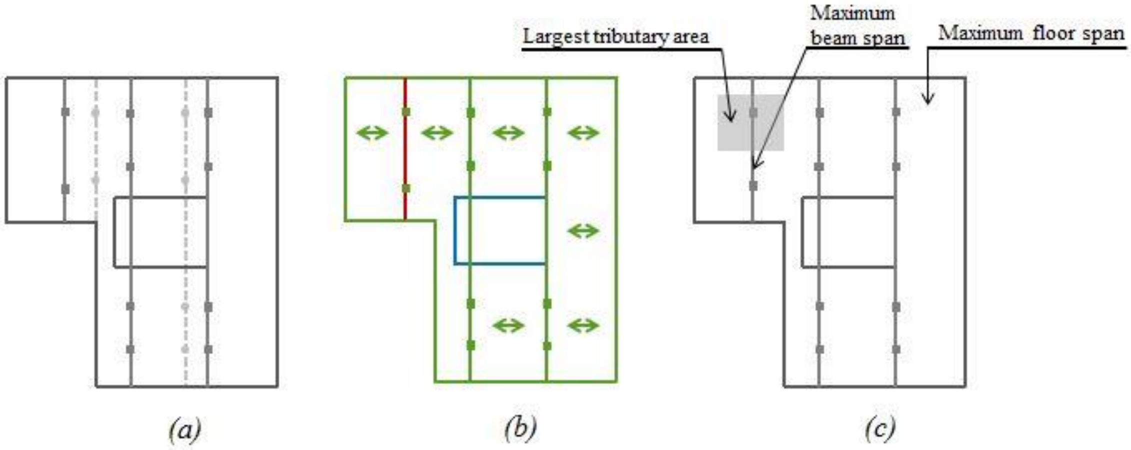


Figure 42 *Floor plans of Concept 3, (a) dashed lines are showing the beam-column lines in the reference building, while the solid lines are showing the beam-column lines used in Concept 3. (b) Floor plan that shows what materials that were used, green stands for timber, blue for concrete and red for steel. (c) Presents information like the maximum span and largest tributary area.*

Since steel is used in the left beam-column line, the balcony beam is also a steel beam. The materials used are presented in Table 28.

Table 28 *Materials used for members in Concept 3.*

Components in the structural system		
Stabilising core	Concrete	Prefabricated
Roof	Concrete	Hollow core slab
Floors	Timber	Cassette floor from Martinsons (2006)
Beams	Timber and Steel	Kerto-S and HEA, see Figure 42b
Columns	Steel	VKR
Walls	Timber	CLT from Martinsons (2014)

Concept 3 has a maximum

- floor span of 8.425 m
- beam span of 8.6 m for the steel beam and 5.7 m for the timber beams
- tributary area for columns of 48.7 m^2
- influence width for the walls of 4.21 m

In Table 29 the dimensions of the beams are presented. Due to the change of material and removal of one column the steel beams in the left beam line needed to be designed. The timber beams are the same as in the first concept.

Table 29 Needed dimensions of the beams in Concept 3 including fire protection.

Beams concept 3	Dimension [mm]	Span [m]	Influence width [m]	Load ULS [kN/m^2]	Utilisation [%]
Roof beam	800× 225 (Kerto-S)	5.7	8.425	63.6	82.5
Balcony beam	655×331 (HEA650)	8.6	7.675	107.1	98.4
Office beams 1	650× 225 (Kerto-S)	5.7	8.425	50.2	79.9
Office beam 2	505×331 (HEA500)	8.6	7.675	44.8	82.7

The removal of the middle column in the left beam line also created an increased tributary area for the columns. New dimensions for the columns were therefore calculated. Table 30 is showing the needed dimensions.

Table 30 Needed dimensions of the columns in Concept 3 including fire protection.

Storey	Dimension [mm]	Load ULS [MN]	Utilisation [%]
11th – 14th floor	330×360	1.18	56.6
6th – 10th floor	430×405	2.73	89.5
2nd – 5th floor	430×540	3.90	97.0

6.7 Concept 4

For concept 4 steel beams were used instead of timber beams in all beam lines. The choice of materials is shown in Table 31.

Table 31 Materials used for members in Concept 4.

Components in the structural system		
Stabilising core	Concrete	Prefabricated
Roof	Concrete	Hollow core slab
Floors	Timber	Cassette floor from Martinsons (2006)
Beams	Steel	HEA
Columns	Timber	Glulam, L40c
Walls	Timber	CLT from Martinsons (2014)

The change of beam material enabled a reduction of columns by one compared to Concept 1 and 2, see Figure 43. In Appendix C3 a more detailed drawing of Concept 4 is presented.

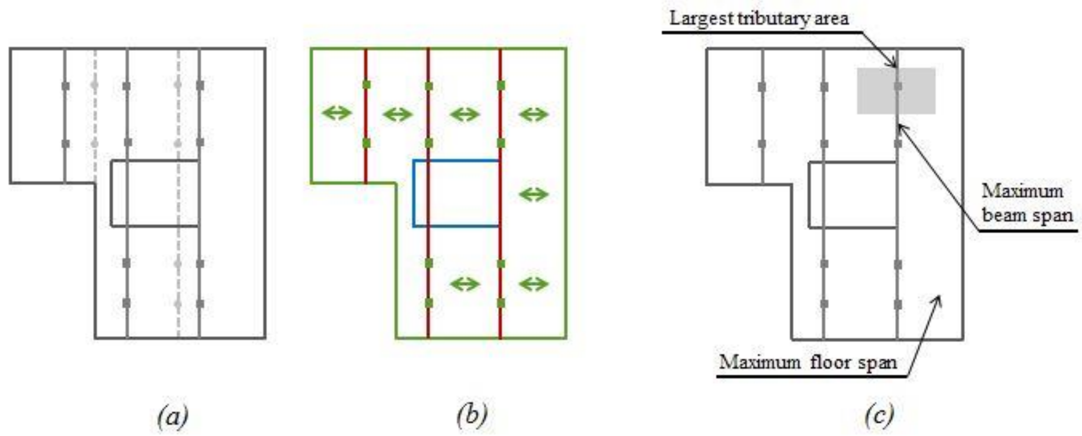


Figure 43 *Floor plans of Concept 4, (a) dashed lines are showing the beam-column lines in the reference building, while the solid lines are showing the beam-column lines used in Concept 4. (b) Floor plan that shows what materials that were used, green stands for timber, blue for concrete and red for steel. (c) Presents information like the maximum span and largest tributary area.*

Concept 4 has a maximum:

- floor span of 8.425 m
- beam span of 7.8 m
- tributary area for columns of 50.1 m^2.
- influence width for the walls of 4.21 m

The needed dimensions of the steel beams are presented in Table 32. Each beam is protected by one fire gypsum board on the exposed sides.

Table 32 *Needed dimensions of the beams in Concept 4 including fire protection.*

Beams Concept 4	Profile	Dimension [mm]	Span [m]	Influence width [m]	Load ULS [kN/m^2]	Utilisation [%]
Roof beam	HEA500	505×331	7.8	8.425	62.5	87.7
Balcony beam	HEA650	655×331	7.8	7.675	117.6	82.3
Office beam	HEA450	455×331	7.8	8.425	49.2	93.9

A larger tributary area was obtained in this concept than for Concept 1 and Concept 2, inducing larger loads on the columns. In Table 33 the needed dimensions of the columns are presented.

Table 33 *Needed dimensions of columns in Concept 4 including fire protection.*

Storey	Dimension [mm]	Load ULS [MN]	Utilisation [%]
11^{th} – 14^{th} floor	330×360	1.22	58.5
6^{th} – 10^{th} floor	430×405	2.68	87.9
2^{nd} – 5^{th} floor	430×540	3.86	96.0

6.8 Concept 5

In Concept 5 both steel beams and steel columns were used. The concept still has three beam lines and the same layout as in Concept 4, with maximum beam spans as in the reference building, see Figure 44. In Appendix C3 a more detailed drawing of Concept 5 is presented. Table 34 summarises the materials for the components in Concept 5.

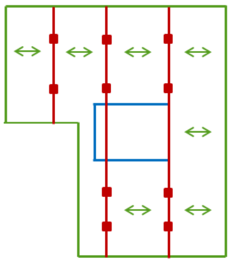


Figure 44 Floor plan that shows what materials that were used, green stands for timber, blue for concrete and red for steel.

Table 34 Materials used for members in Concept 5.

Components in the structural system		
Stabilising core	Concrete	Prefabricated
Roof	Concrete	Hollow core slab
Floors	Timber	Cassette floor from Martinsons (2006)
Beams	Steel	HEA
Columns	Steel	VKR
Walls	Timber	CLT from Martinsons (2014)

Concept 5 has a maximum

- floor span of 8.425 m
- beam span of 7.8 m
- tributary area for columns of 50.1 m^2
- influence width for the walls of 4.21 m

The steel beams are the same as in Concept 4. The needed dimensions of the steel columns are presented in Table 35. Each column is protected by fire gypsum boards on the exposed sides.

Table 35 Needed dimensions of the columns used in Concept 5 including fire protection.

Storey	Profile	Dimension [mm]	Load ULS [MN]	Utilisation [%]
$11^{th} - 14^{th}$ floor	VKR180, 10	242×242	1.22	60.1
$6^{th} - 10^{th}$ floor	VKR250, 10	312×312	2.78	88.8
$2^{nd} - 5^{th}$ floor	VKR250, 16	312×312	4.00	82.8

6.9 Concept 6

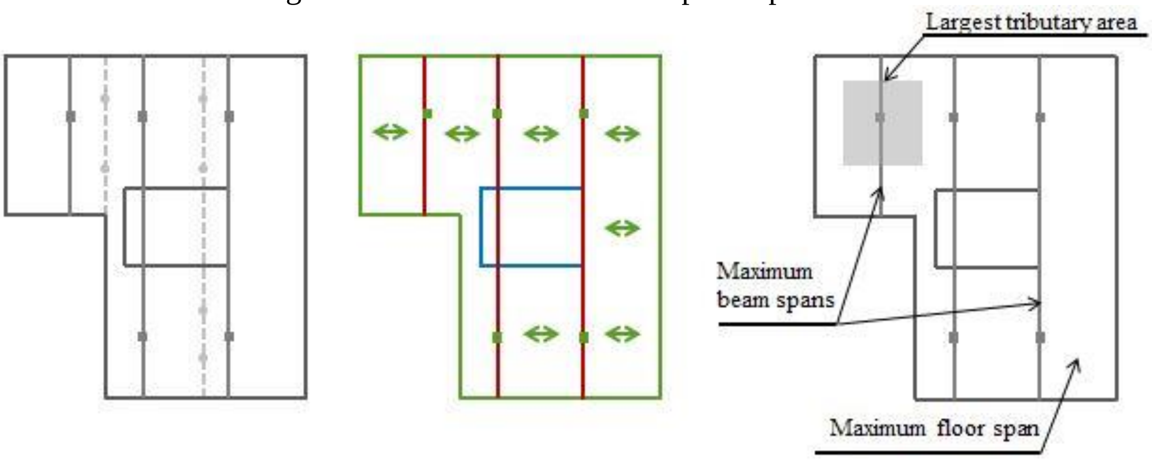
For Concept 6 the aim was to decrease the number of columns by increasing the spans of the beams by using steel instead of timber. Concept 6 was developed in two iterations. The components used in the system are presented in Table 36.

Table 36 Materials used for members in Concept 6.

Components in the structural system		
Stabilising core	Concrete	Prefabricated
Roof	Concrete	Hollow core slab
Floors	Timber	Cassette floor from Martinsons (2006)
Beams	Steel	HEA
Columns	Timber	Glulam, Lc40
Walls	Timber	CLT from Martinsons (2014)

6.9.1 First iteration

In the first iteration the columns were moved more into the middle of each side of the stabilising core in order to reduce the number of columns. This creates long spans but less columns compared to the other concepts, see Figure 45. The amount of columns is one less than in the reference building. However, this concept has one more beam-column line than the reference building, which enables timber floors. In Appendix C3 a more detailed drawing of the first iteration of Concept 6 is presented.



(a) (b) (c)

Figure 45 Floor plans of Concept 6, (a) Dashed lines are showing the beam-column lines in the reference building, while the solid lines are showing the beam-column lines used in the first iteration of Concept 6. (b) Floor plan that shows what materials that were used, green stands for timber, blue for concrete and red for steel. (c) Presents information like the maximum span and largest tributary area.

The first iteration of Concept 6 has a maximum

- floor span of 8.425 m
- beam span of 11 m and 8.025m respectively
- tributary area for columns of 64.5 m^2
- influence width for the walls of 4.21 m

The needed dimensions for the steel beams are presented in Table 37. For the first iteration of Concept 6 two different beams were chosen for the office beams. This is since there are large differences in the spans and it is therefore inefficient to have the same dimension everywhere, see Figure 45c.

Table 37 Needed dimensions of the beams used in the first iteration of Concept 6, including fire protection.

Beams concept 6.1	Profile	Dimensions [mm]	Span [m]	Influence width [m]	Load ULS [kN/m^2]	Utilisation [%]
Roof beam	HEA500	505×331	8.03	8.425	62.5	94.9
Balcony beam	HEA900	905×331	11.0	7.675	117.6	92.3
Office beam 1	HEA500	505×331	8.03	8.425	49.2	74.7
Office beam 2	HEA650	655×331	11.0	7.675	44.8	85.3

The needed dimensions for the timber columns are presented in Table 38.

Table 38 Needed dimensions of the columns used in the first iteration of Concept 6, including fire protection.

Storey	Dimension [mm]	Load ULS [MN]	Utilisation [%]
11^{th} – 14^{th} floor	330×360	1.56	74.8
6^{th} – 10^{th} floor	430×540	3.62	90.0
2^{nd} – 5^{th} floor	645×540	5.19	86.4

6.9.2 Second iteration

Since the columns became relatively large and three different beams were used Concept 6 was developed further. In addition the balcony beam became too high. By adding an extra column the maximum span of 11 metres was reduced, resulting in lower beam heights and a smaller tributary area for the columns. The layout of the concept is illustrated in Figure 46. In Appendix C3 a more detailed drawing of the second iteration of Concept 6 is presented.

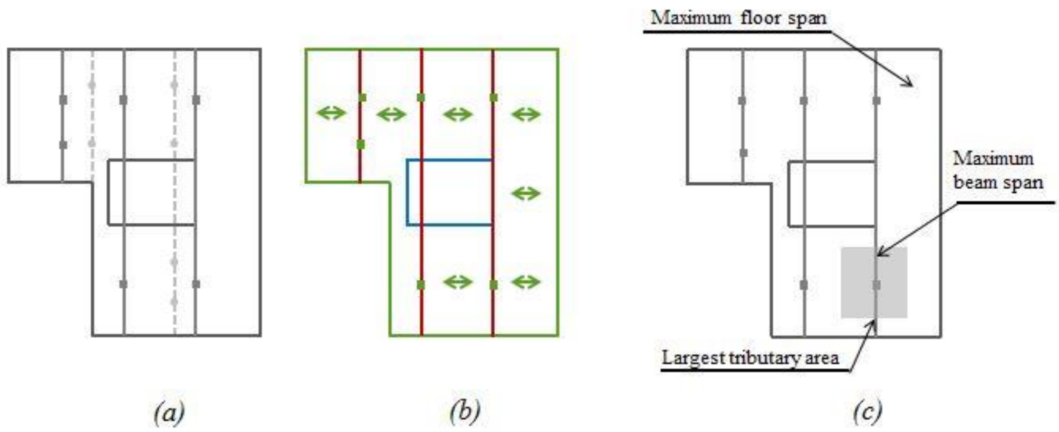


Figure 46 *Floor plans of Concept 6 (a) Dashed lines are showing the beam-column lines in the reference building, while the solid lines are showing the beam-column lines used in the second iteration of Concept 6. (b) Floor plan that shows what materials that were used, green stands for timber, blue for concrete and red for steel. (c) Presents information like the maximum span and largest tributary area*

The second iteration of Concept 6 has a maximum

- floor span of 8.425 m
- beam span of 8.025 m
- tributary area for columns of 58.2 m^2
- influence width for the walls of 4.21 m

The needed dimensions for the steel beams are presented in Table 39.

Table 39 *Needed dimensions of the beams used in the second iteration of Concept 6, including fire protection.*

Beams concept 6.2	Profile	Dimensions [mm]	Span [m]	Influence width [m]	Load ULS [kN/m^2]	Utilisation [%]
Roof beam	HEA500	505×331	8.03	8.425	62.5	94.9
Balcony beam	HEA500	505×331	6.10	7.675	117.6	78.1
Office beam	HEA500	505×331	8.03	8.425	49.2	75.0

The needed dimensions for the timber columns are presented in Table 40.

Table 40 *Needed dimensions of the columns used in the second iteration of Concept 6, including fire protection.*

Storey	Dimension [mm]	Load ULS [MN]	Utilisation [%]
11^{th} – 14^{th} floor	330×360	1.41	67.6
6^{th} – 10^{th} floor	430×540	3.24	80.6
2^{nd} – 5^{th} floor	570×540	4.63	87.1

6.10 Evaluation and choice of promising solutions

In this section the positive and negative aspects of each concept are presented separately. The aim of the project was to implement timber in the structural system as much as possible. Exchanging material to timber should not affect the activity and purpose of the building by having to large dimensions of structural members. Furthermore, columns should not be placed in such way that they interfere with the purpose of the building. Timber should be used where it is best suited. However, the aim was not to obtain smaller dimensions than for the reference building. By using timber to a great extent the disadvantages of having larger dimensions and more components can be outweighed.

6.10.1 Evaluation of concepts

For all concepts a cassette floor measuring 800 mm in height including installations and insulation was used. The total height of the floor structure is approximately the same as for the reference building and thereby the interior height of the building remains constant. However, to enable the usage of timber floors an extra beam-column line had to be added, leading to more columns and beams compared to the reference building.

By adding an extra beam-column line the influence width of the wall was reduced, leading to decreased compressive forces on the walls. The thickness of the walls became larger than for the reference building but still considered as acceptable.

6.10.1.1 Concept 1

In this concept timber is used to a greater extent than in the other concepts which is preferable. The size of the columns became somewhat larger than for the reference building but still acceptable since the difference is small. However, timber beams tend to become high even for relatively short spans. Therefore extra columns were added in order to ensure that the beams remained sufficiently low, lower than the floor elements.

According to the architectural drawings the eight columns to the right in Figure 39b are possible to integrate in the layout of the floor plan. Therefore the consequence of adding extra columns in this part of the building is low. However, the column added in the middle of the left beam-column line interferes with the demanded open area that could be used as a social area.

6.10.1.2 Concept 2

In this concept the timber columns were replaced by steel columns, while the floor plan remained the same as for Concept 1. The dimensions of the steel columns became considerably smaller than the columns in Concept 1 and in the reference building. However, the problem with the poorly placed column in the middle of the left beam line still remains.

6.10.1.3 Concept 3

In this concept the beam in the left beam-column line was replaced by a HEA beam. By doing so, the column in the middle of the open area could be removed without consequences regarding height of the beam. The negative aspects of this concept are that several different beam types are needed and that the column on the 11th floor and above needs a larger dimension due to a larger tributary area. However, the dimensions are approximately the same as in Concept 1 and timber is used to a great extent.

6.10.1.4 Concept 4

In Concept 4 steel beams were used instead of timber beams, which enable longer spans and thereby the unfavourable placed column in the open social area in Concept 1 and 2 could be removed. The timber columns have the same dimensions as the timber columns in Concept 1, so the reduction of the number of columns is not resulting in larger columns.

6.10.1.5 Concept 5

Concept 5 has the same layout as Concept 4 but is consisting of steel beams and steel columns. The dimensions of the beams and columns are small due to small loads compared to the reference building, because of the additional beam-column line. In this concept the columns are smaller than for the reference building and they do not interfere with the activity of the building. However this is the concept where timber is used to the least extent.

6.10.1.6 Concept 6

For this concept longer spans are used resulting in higher steel beams but fewer columns. In the first iteration more beam-column lines were used than in the reference building, but the number of columns was still reduced by one. However, several different beams had to be used and some beams were higher than the floor elements. Therefore, in order to limit the height of the floor structure one more column was added in the second iteration, hence smaller beams could be used.

Concept 6 has few columns compared to the other concepts, but the same amount as the reference building, which is advantageous for this concept. The beam-column lines are the same as for the other concepts enabling timber floors. This creates open spaces and few columns that are taking up space; on the other hand the columns were designed as timber columns and therefore obtained large dimensions. A way to reduce the column size would be to change material to steel, but then the aim of using as much timber as possible would not be fulfilled in the same degree.

6.10.2 Summation of the concepts

In Table 41 the dimensions of the different components in each concept are summarised and in Figure 47 the layout of each concept is illustrated. As can be seen in Table 41 the height of the floor structure is the same for all concepts including the reference building. This value refers to the total structural height including insulations and beams, see Figure 47. However, for the timber floor it might be difficulties with installations that need to be placed perpendicular to the beam direction. Either the total construction height can be increased by lowering the ceiling or holes can be made in the beams. If making holes it might be necessary to increase the dimensions of the beams. This was however not investigated in this project.

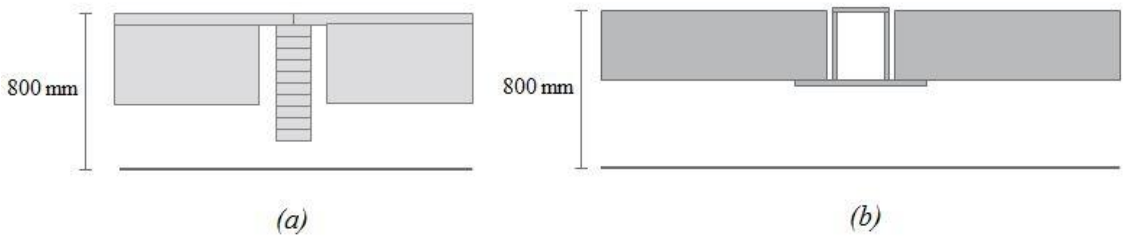


Figure 47 *Floor structures in the concepts and the reference building, (a) the floors in the concepts (b) floors in the reference building.*

It should also be mentioned that all members in Table 41 were designed with regard to fire. Therefore dimensions of additional fire protection is included in the values given in the table. If information about the steel profile or utilisation ratios is of interest, the reader is referred to the tables given in the relevant section for each concept.

Table 41 *Summarising table of all concepts with needed dimensions and materials for different members.*

	Reference Building [mm] Figure 48a	**Concept 1 [mm]** Figure 48b	**Concept 2 [mm]** Figure 48b	**Concept 3 [mm]** Figure 48c	**Concept 4 [mm]** Figure 48d	**Concept 5 [mm]** Figure 48d	**Concept 6 [mm]** Figure 48e
Floor	**Concrete**	**Timber**	**Timber**	**Timber**	**Timber**	**Timber**	**Timber**
Office floor	800	800	800	800	800	800	800
Beams	**HSQ**	**Kerto-S**	**Kerto-S**	**Kerto-S/ HEA**	**HEA**	**HEA**	**HEA**
Roof beam	-	800× 225	800× 225	800× 225 (Kerto-S)	505×331	505×331	505×331
Balcony beam	380×400	800× 300	800× 300	655×331 (HEA)	655×331	655×331	505×331
Office beam	275×400	650× 225	650× 225	650× 225 (Kerto-S)	455×331	455×331	505×331
Office beam 2	-	-	-	505×331 (HEA)	-	-	-
Columns	**VKR**	**Glulam Lc40**	**VKR**	**Glulam Lc40**	**Glulam Lc40**	**VKR**	**Glulam Lc40**
11th – 14th floor	300×300	330×270	212×212	330×360	330×360	242×242	330×360
6th – 10th floor	400×400	430×405	312×312	430×405	430×405	312×312	430×540
2nd – 5th floor	400×400	430×540	312×312	430×540	430×540	312×312	570×540
Walls	**Concrete**	**CLT**	**CLT**	**CLT**	**CLT**	**CLT**	**CLT**
11th – 14th floor	150	158	158	158	158	158	158
6th – 10th floor	150	221	221	221	221	221	221
2nd – 5th floor	150	259	259	259	259	259	259

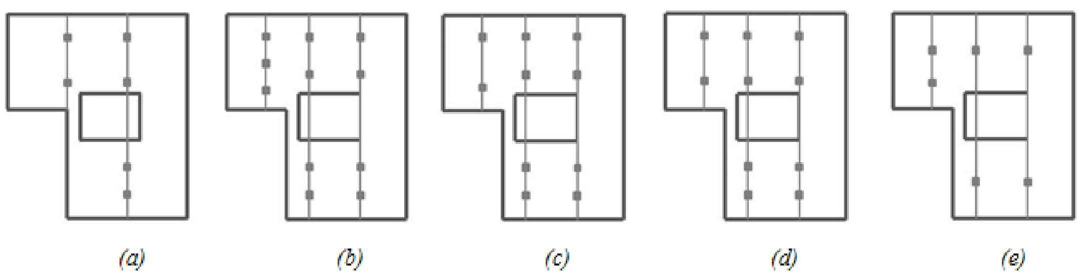


Figure 48 *Layout for the different concepts (a) the reference building, (b) Concepts 1 and 2, (c) Concept 3, (d) Concepts 4 and 5 and (e) Concept 6.*

6.11 Choice of promising concepts

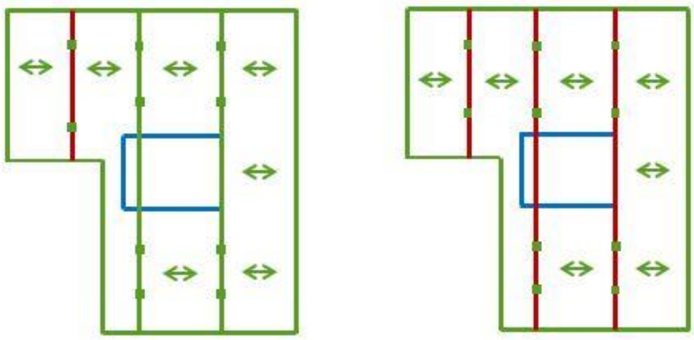
Both Concepts 1 and 2 were disregarded due to the placement of the column in the middle of the open area at the left beam-column line. Concept 5 is good with regard to the size and placement of the members but was disregarded due to the aim of implementing as much timber as possible.

The dimensions of the timber members in Concept 6 were considered to be too large and, even though the concept enables few columns, they will most likely be hard to integrate well in the building. In addition the upper left column in the left beam-column line was moved a little bit towards the middle, making the open space smaller, see Figure 46. Therefore Concept 6 was disregarded.

Two concepts where chosen for further investigation, Concept 3 and Concept 4. For both concepts timber components are utilised to great extent and the placement of the columns are not interfering with the architectural drawings. The dimensions of the components are somewhat larger than for the reference building but still considered as acceptable. Needed dimensions for the two promising concepts are summarised in Table 42 and the layout is illustrated in Figure 49.

Table 42 Needed dimensions and material choice for the promising concepts.

	Concept 3 [mm]	**Concept 4 [mm]**
Floor	**Timber**	**Timber**
Office floor	800	800
Beams	**Kerto-S/ HEA**	**HEA**
Roof beam	800× 225 (Kerto-S)	505×331
Balcony beam	655×331 (HEA)	655×331
Office beam	650× 225 (Kerto-S)	455×331
Office beam 2	505×331 (HEA)	-
Columns	**Glulam Lc40**	**Glulam Lc40**
11^{th} – 14^{th} floor	330×360	330×360
6^{th} – 10^{th} floor	430×405	430×405
2^{nd} – 5^{th} floor	430×540	430×540
Walls	**CLT**	**CLT**
11^{th} – 14^{th} floor	158	158
6^{th} – 10^{th} floor	221	221
2^{nd} – 5^{th} floor	259	259



(a) (b)

Figure 49 Floor plans, for the two promising concepts, that show what materials that have been used, green stands for timber, blue for concrete and red for steel. (a) is showing Concept 3 and (b) is showing Concept 4.

7 Additional analysis of promising concepts

In this chapter the results from the more detailed investigations of the promising concepts are presented.

7.1 Total weight of the buildings

As described in Section 2.3.2 the low self-weight of a timber building is beneficial for the foundation work. The lighter the building is, the less piles are needed. On the other hand, a light structure can be a problem when designing tall buildings and in many cases it is therefore necessary to provide such light structures with extra weight or with anchorage in order to prevent lifting and tilting of the building. Therefore it was of interest to calculate the total weight of the building according to Concept 3, Concept 4 and for the reference building. The results are presented in Table 43. The weight of the two mixed systems is approximately 50 % lighter than the weight of the reference building. Performed calculations are presented in Appendix D5.

Table 43 Weight of the reference building and the buildings according to concepts, expressed both in tonnes and in MN.

	Reference building	Concept 3	Concept 4
Total weight of the building [tonnes]	12740	6205	6252
Total permanent load from the building [MN]	125	60.8	61.3
Average weight of one storey [tonnes]	849	414	417
Average permanent load from one storey [MN]	8.3	4.1	4.1

In Table 44 the weight of the individual members are presented. The weight of the core and the roof is the same for the concepts and the reference building. The weight of the floors and walls results in the largest difference in relation to the total weight of the building. The largest difference in percent is between the beams in the reference building and in Concept 4. However the influence on the total weight is small and therefore not important in comparison to the weight of the floor and wall structures.

Table 44 Weight of the individual members and their differences in relation to the reference building in percent.

	Reference building [ton]	Concept 3 [ton]	Difference relative to reference [%]	Concept 4 [ton]	Difference relative to reference [ton]
Floor	7230	2825	60.9% lighter	2825	60.9% lighter
Roof	385.4	385.4	0 % difference	385.4	0 % difference
Columns	63.0	63.2	0.3% heavier	63.2	0.3 % heavier
Beams	70.6	90.5	28.2% heavier	138.2	95.8 % heavier
Walls	3214	1095	65.9 % lighter	1095	65.9 % lighter
Core	1773	1773	0% difference	1773	0% difference
Total weight	12740	6205	51.3 % lighter	6252	50.9 % lighter

As stated previously, a lighter building demands less foundation work. For a building founded on piles this can mean fewer piles, smaller piles or both. This results in less material used, less energy put into construction of the foundation which makes the foundation, of a lighter building cheaper, simpler and more environmental friendly. A lighter building is especially good to consider in areas of bad soil conditions, like those in Göteborg, where clay is the dominant soil.

7.2 Sectional forces in the core

The sectional forces in the core were calculated in order to investigate whether the core is fully compressed while subjected to horizontal wind load and unintended inclination. In Section 7.2.1 the assumptions made and the calculation procedure are presented and in Section 7.2.2 the results showing the differences between the reference building and the concepts are presented. The calculations are presented in Appendix D9.

7.2.1 Assumptions and calculation procedure

Initially the equivalent load effect due to unintended inclination and wind load, which result in a bending moment needed to be calculated. It was assumed that all the walls of the core are coupled making the core acting as one unit. Navier's formula was used to check if parts of the core, when the building is subjected to wind from north or east, are in tension.

The normal force was determined from a load combination with the weight of the core plus additional permanent load from the floors. Figure 50 shows which areas that were assumed when accounting for the permanent loads from the floors. It should be noted that there is a small difference between area 4 in Concept 3 and area 4 in Concept 4. However, this is not illustrated in Figure 50. The total tributary area in the reference building is smaller than for the concepts due to the change in the layout of beams and columns. However, the weight of the floors in the reference building is heavier than in the concepts.

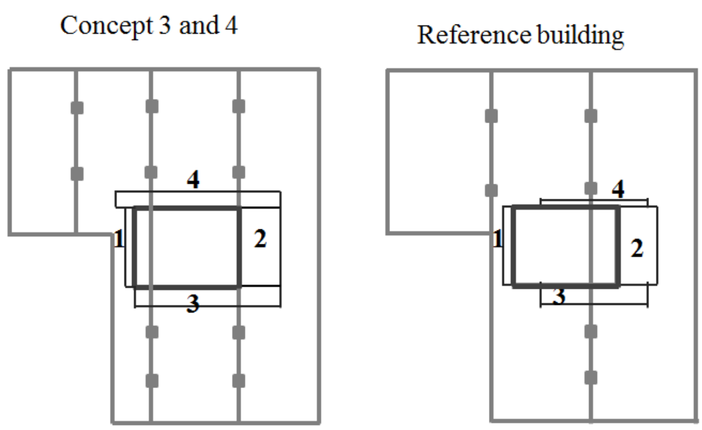


Figure 50 Tributary areas for the core in the concepts and the reference building.

In order to obtain the worst load combination the permanent load needs to be considered as both unfavourable and favourable. If the permanent load is unfavourable the equivalent load effect from unintended inclination is larger, but so is also the vertical normal force, which is reducing the tension in the core. In the opposite case, when permanent load is favourable, the equivalent load effect is smaller, but so is also the normal force. Hence, both cases need to be considered.

7.2.2 Results from the analysis of sectional forces

In Table 45 the stresses in the concrete core for the reference building and the two concepts are presented. The stresses were calculated for two cases, wind on the north façade and wind on the east façade. Calculations proved that the worst load combination is the one were the permanent load is assumed favourable. When the north side is subjected to wind, notable tensile stresses arise in the core. On the other hand, when the east side is subjected to wind, the entire core is compressed. This is because the concrete core has a considerably higher stiffness in this direction.

Table 45 Calculated stresses in the part of the core where tensile stresses may arise. Positive sign is tension and negative sign is compression.

	Wind from north	Wind from east
Reference building	2.22 MPa	-0.23 MPa
Concept 3	2.35 MPa	-0.08 MPa
Concept 4	2.37 MPa	-0.07 MPa

It can be concluded that, even though the influencing area is increased for the concepts compared to the reference building, the core in the concepts is experiencing more tension than the core in the reference building. This is because timber floors have a lower weight than concrete floors.

The concrete strength class assumed for the core is C45/55, with a characteristic 5%-fractile tensile strength of 2.7 MPa and a mean tensile strength of 3.8 MPa. It is therefore argued that the concrete core is not likely to crack. Still minimum reinforcement should be used, which increases the capacity further. Moreover, since the core in the reference building already is designed by the consulting company Integra, it is assumed that it has sufficient capacity to resist the lateral loads. As can be seen in Table 45, the difference between the tensile stresses of the reference building and the concepts is small and therefore it was assumed not to be necessary to check state II.

It can be concluded that it is beneficial to design the building such that the core is resisting more loads from the floors. This would have decreased the tensile stresses in the core and increased the compressive stresses. When using a mixed structure with a timber system stabilised by a core, it can be concluded that it is extra important to enable the core to carry more load. This will also make the concrete core to creep more, which is positive with regard to the vertical displacements.

One way to prevent tilting of a building is to anchor the bracing members to the foundation. This method is for example used in the timber building Limnologen in Växjö. Another solution is to increase the vertical load on the bracing members. In the timber building Treet in Bergen, Norway, this is achieved by having concrete floors on every fifth storey.

7.3 Vertical displacements in mixed structures

One factor that is important to consider in a mixed structural system is the difference between the vertical displacements for systems of different materials. In Engquist et al. (2014) the in-situ measurements of the vertical displacement of the outer load bearing CLT-walls of Limnologen in Växjö are presented. Limnologen is an eight-stories building whereof storey 2-8 are in timber. The measurements resulted in a total annual average vertical displacement of 23 mm after 6.5 years of service life. One of the conclusions in Engquist et al. (2014) is that the main factors affecting the vertical displacements of the CLT-walls was the shrinkage and swelling of the timber due to variation in the climate, see Figure 51. For further description of Limnologen the reader is referred to Section 2.4.2.

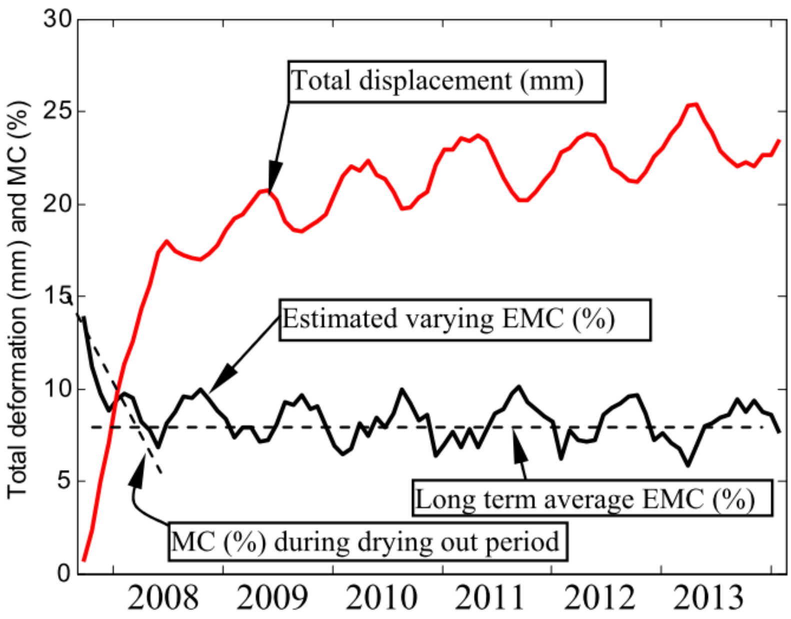


Figure 51 Total vertical displacement and estimated equilibrium moisture content in Limnologen (Engquist et al., 2014) .

In Engquist et al. (2014) the deformations due to initial shrinkage were estimated for the CLT-walls according to Equation (22) and Equation (23). It was assumed that six wall elements with a height of three metres each were shrinking in the longitudinal direction of the grains. In addition the floor board connected between the wall elements was shrinking in the transversal direction, contributing to the overall vertical displacements. The calculated total change in height proved to be in accordance with the measured displacement after nine months. However, when using this method only the initial deformations due to moisture change are considered. The annual variation of the moisture content results in irreversible deformations that also need to be considered.

7.3.1 Method for determining vertical deformations

The vertical displacement was calculated as the sum of the creep deformations due to the load and the shrinkage deformations due to change in the moisture content.

Deformations due to creep

The magnitude of the creep deformations depends both on the magnitude and the duration of the applied load. For many building materials the relation between the creep deformation and the elastic deformation is nearly constant. This relation is called the creep coefficient (Burström, 2007).

For timber the creep coefficient increases with increasing temperature and humidity. When taking creep deformations into account the creep coefficient can be used to reduce the elastic modulus of the material. The final strain-dependent deformations for timber including creep can be calculated according to Hooke's law, by using an effective value of the elastic modulus, see Equation (20).

$$\sigma = E \cdot \varepsilon = E \cdot \frac{u}{L} \rightarrow u = \frac{\sigma \cdot L}{E} \rightarrow u = \frac{Q \cdot (1 + k_{def}) \cdot L}{A \cdot E_{0.mean}} \tag{20}$$

As for timber, the creep strain of concrete is defined by means of a creep coefficient. Hence the total stress-dependent deformation can be calculated according to Equation (21). The creep coefficient is influenced by the concrete age at loading, concrete composition, size of the section and the surrounding relative humidity (Engström, 2014).

$$u = \frac{Q \cdot (1 + \varphi(t, t_0)) \cdot L}{A \cdot E_{cm}} \tag{21}$$

Deformations due to shrinkage and swelling

Moisture induced deformations occur in all porous materials. When the moisture content decreases, the material shrinks and, when the moisture content increases, the material swells.

According to Burström (2007) the shrinkage or swelling of timber can be estimated by assuming a linear relationship between the moisture content and the shrinkage or swelling. The magnitude of the shrinkage or swelling can thereby be calculated by knowing the fibre saturation point, see Equation (22).

$$\Delta\alpha = \frac{u_2 - u_1}{u_f} \cdot \alpha_f \tag{22}$$

$u_2 - u_1$	Difference in moisture content, around 8%
u_f	Fibre saturation point, 30 % for conifers such as spruce and pine
α_f	Maximum shrinkage in a certain direction, 0.3 % for spruce parallel to the grain

The absolute value of the shrinkage or swelling movement related to the original size can then be calculated according to Equation (23).

$$\Delta L = \Delta\alpha \cdot L \tag{23}$$

The shrinkage of concrete starts during hardening and increases with time. Shrinkage strain can be divided into two components; drying shrinkage and autogenous shrinkage. The former depends on the exchange of moisture content between the concrete and the surrounding, and the latter develops during the hardening of the concrete. According to part 1-1 in Eurocode 2, CEN (2008b), the final shrinkage strain can be calculated with Equation (24).

$$\varepsilon_{cs}(\infty) = \varepsilon_{cd}(\infty) + \varepsilon_{ca}(\infty) \tag{24}$$

$\varepsilon_{cd}(\infty)$ Drying shrinkage strain
$\varepsilon_{ca}(\infty)$ Autogenous shrinkage strain

7.3.2 Vertical displacements for Concept 3 and Concept 4

Since it is concluded in Engquist et al. (2014) that vertical displacements may be a problem in tall timber buildings, it was of interest to investigate the displacements of the two concepts as well. In contrast to the results in Engquist et al. (2014), where the main factors affecting the vertical displacements were shrinkage and swelling, the magnitude of the creep and the shrinkage was about the same for the concepts. The displacements due to shrinkage and creep of the first floor and in the basement were neglected, since they are made of concrete.

The load on the columns was calculated by using the maximum tributary area. Therefore the results differ between the concepts. Each column has an individual creep and displacement development. Two different tributary areas were used for each concept, one area from the part of the building that has 14 storeys and one area for the part of the building that has 12 storeys. The concrete core was only assumed to carry its own weight and the weight of the concrete floor inside the core. For more details of the calculations, the reader is referred to Appendices D1 to D4.

Initially the timber beams in Concept 3 were assumed to be placed between the columns, but calculations showed that the compression perpendicular to the grains became too high resulting in large vertical deformations and crushing of the material. Therefore the beams were instead assumed to be placed on corbels to the columns, see Figure 52.

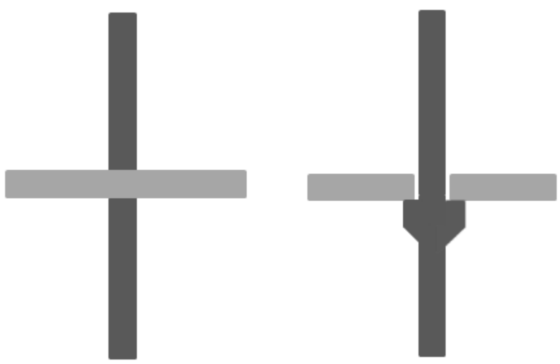


Figure 52 Two different types of beam-column connections.

The total vertical displacement of the concrete core, the walls and the columns are presented in Table 46. For the timber components, a change in moisture content of 8 % is assumed and the creep coefficient is taken as 0.6 for glulam and 0.8 for CLT.

Table 46 *The total vertical displacements for different parts in the concepts and the distribution between shrinkage and creep (moisture content change of 8 %, deformation factor k_{def} of 0.8 for CLT-walls and 0.6 for the glulam columns).*

	Shrinkage [mm]	Total creep [mm]	Total deformation [mm]	Difference from concrete core [mm]
Concrete core	19.4	2.6	22.0	0
CLT walls	38.7	30.9	69.6	47.6
Columns concept 3	38.7	40.4	79.1	57.1
Columns concept 4	38.7	49.6	88.3	66.3

The total deformation accumulates through the building; hence the largest displacements occur on the top floor. In Table 47 the displacements at each floor are presented. For each concept two results are presented, one for the columns in the part of the building having 14 storeys and one for the part with 12 storeys.

Table 47 *Total vertical displacements at each storey.*

Floor	Concrete core [mm]	Walls [mm]	Columns concept 4 (14) [mm]	Columns Concept 4 (12) [mm]	Columns concept 3 (14) [mm]	Columns concept 3 (12) [mm]
14	22.0	69.6	88.3	-	79.1	-
13	20.1	65.1	83.2	-	74.2	-
12	18.5	61.4	78.4	67.4	69.8	69.9
11	16.8	56.8	72.2	62.6	64.2	64.9
10	15.3	52.3	65.9	57.4	58.4	59.6
9	13.7	47.5	60.0	52.5	53.2	54.5
8	12.1	42.2	53.7	47.0	47.4	48.9
7	10.4	36.6	46.7	41.1	41.3	42.7
6	8.8	30.6	39.3	34.6	34.6	36.0
5	7.1	24.3	31.2	27.6	27.6	28.8
4	5.3	18.6	24.0	21.3	21.2	22.2
3	3.6	12.7	16.4	14.6	14.5	15.2
2	1.8	6.5	8.4	7.5	7.4	7.8

As can be seen in both Table 46 and Table 47, there is a significant difference between the vertical displacements of the concrete and the timber systems. For example there is a difference of approximately 6.6 cm between the concrete core and the columns in Concept 4. This difference in vertical displacements might cause problem during the service life of the building. However, when erecting a timber building, one storey is assembled at the time. The timber starts to deform directly when loaded and therefore some of the vertical deformations occur during the construction. In order to compensate for these deformations the columns can be made longer than their final length. Therefore some of the displacements can be handled by appropriate measures in design and production.

In addition it is also important to consider that the values for the vertical displacements are determined theoretically. The magnitude of the real vertical deformations might differ. By controlling some of the parameters the deformations can be decreased. In Engquist et al. (2014) the change in moisture content was taken as 8 % and therefore the same value was assumed for the calculations of the total displacement in this project. However, according to Kliger (2015-03-12) it is possible to reduce the change in moisture content to 2%. This reduced value is based on the assumption that the glulam beams and CLT walls can be allowed to dry out before use and wrapped in plastic during transportation. In addition each floor must be constructed directly after mounting of the columns beneath and the stories must be heated in order to prevent moisture change.

In Table 48 values for the vertical displacements with the assumption that the change in moisture content is 2% are presented. The results show a considerable decrease in the shrinkage deformations. However, the magnitude of the creep deformations remains.

Table 48 The total vertical displacements for different parts in the concepts and the distribution between shrinkage and creep (moisture content change of 2 % , deformation factor k_{def} of 0.8 for CLT-walls and 0.6 for the glulam columns).

	Shrinkage [mm]	Creep [mm]	Total deformation [mm]	Difference from concrete core [mm]
Concrete core	19.4	2.6	22.0	0
CLT walls	9.7	30.9	40.6	18.6
Columns concept 3 (14)	9.7	40.4	50.1	28.1
Columns concept 4 (14)	9.7	49.6	59.3	37.3

Another way to decrease the total vertical displacement is to reduce the creep deformations by choosing parts of the wood that are more mature. The mature wood, close to the bark has a higher elastic modulus than the juvenile wood near the pith and is thereby less prone to creep. According to Kliger (2015-03-12) the value of the deformation coefficient, in other words the creep coefficient, can be decreased significantly, if the product is entirely made of mature wood. However, such products are not available on the market today.

In Table 49, values for the vertical displacements are presented with the assumption that the deformation coefficient is 0.2 for glulam and 0.3 for CLT. The change in moisture content is still assumed to be 2 %.

Table 49 The total vertical deisplacements for different parts in the concepts and the distribution between shrinkage and creep (moisture content change of 8 % , deformation factor k_{def} of 0.3 for CLT-walls and 0.2 for the glulam columns).

	Shrinkage [mm]	Creep [mm]	Total deformation [mm]	Difference from concrete core [mm]
Concrete core	19.4	2.6	22.0	0
CLT walls	9.7	22.3	32.0	10
Columns concept 3 (14)	9.7	30.3	40.0	18
Columns concept 4 (14)	9.7	37.2	46.9	24.9

The results in Table 49 show that by controlling the environment of the timber products and by choosing the material more carefully, the theoretical value of the vertical displacements can be decreased. Nevertheless, there will still be differences between the vertical displacements of the concrete core and the timber systems. The connections between the concrete core and the timber floor must therefore be able to both transfer shear forces horizontally and allow vertical movements.

7.4 Capacity of timber cassette floors

Both Concept 3 and Concept 4 are structures stabilised with regard to lateral loads by a concrete core in the middle of the building. There are no other bracing units in the buildings. However, the floor structure needs to resist the load effects from the lateral loads, acting on the exterior walls. Therefore it is of interest to check that sufficient diaphragm action can be obtained in a floor consisting of cassette floor elements.

7.4.1 Design of timber cassette floors with regard to lateral loads

Floor structures need to be designed for in-plane action with respect to lateral loads such as wind load and effects from unintended inclination. The effects of unintended inclination are described further in Section 3.1.3.

Modelling of floors

According to Kliger (2015-04-29) floors can be considered as high I-beams, where the connected floor elements behave as a web and some additional edge beams behave as flanges, when floors are to be designed for in-plane action . Figure 53 shows two models for two different layouts of bracing units. In the left model, both walls are bracing units. In the right model two walls are bracing units and the third does not contribute to the lateral stability; hence a cantilever beam is obtained resulting in larger moments.

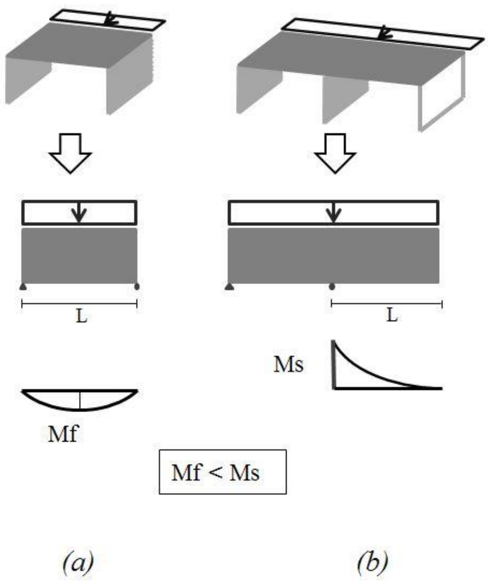


Figure 53 Models of floors for two different arrangements of bracing walls where (a) shows the moment distribution for a simply supported floor diaphragm and (b) shows the moment distribution for a cantilevering part of a floor diaphragm.

Verification of the capacity of floors

The first step in the design of a floor diaphragm is to calculate the moment and shear force distribution in the floor. Figure 53 also shows the moment distribution for each case. When the moment distribution is known, the sectional moment can be converted to a force couple with a tensile and a compressive force, which the floor should be able to resist.

Secondly, the connections between adjacent floor elements need to be designed so that the shear force in the joint can be resisted. The connection can be detailed in different ways; two examples are shown in Figure 54. The last thing that needs to be designed is the connection between connected floor elements and connected stabilising walls.

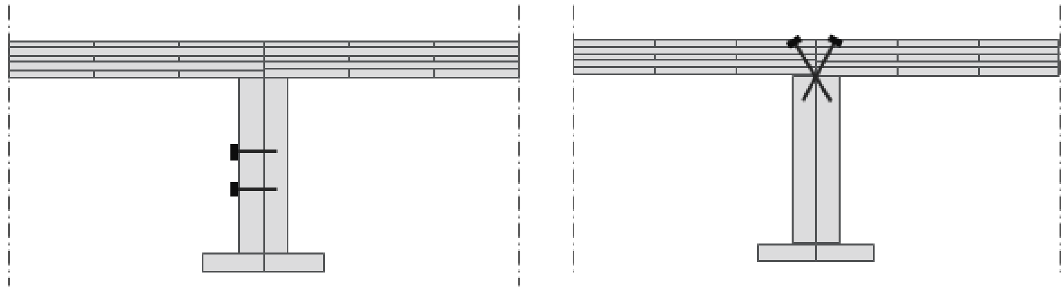


Figure 54 Two different ways of connecting two floor elements for shear resistance in the longitudinal joint.

7.4.2 Load effects in the cassette floor due to lateral loads

The reference building and the two developed concepts are stabilised against horizontal forces by a concrete core in the middle of the building, while the outer walls do not contribute to the global stability. Hence the floor needs to be able to resist horizontal loads by in-plane action. Concepts 3 and 4 have the same type of cassette floor and the calculation procedure and the results are therefore the same.

Model of the cassette floor for Concept 3 and Concept 4

Figure 55 shows how the floor structure in the reference building and the concepts can be modelled as a deep beam supported by the concrete core. The areas marked with grey illustrate the deep beam and how the load is applied. As can be seen in Figure 55 the beam model has two cantilevers, one on each side of the core. Since the direction of the wind load differs, the in-plane resistance of both marked deep beams needs to be checked. The worst load case depends both on the length of the cantilever beam and the applied wind load. The load effect from initial imperfections depends on the layout of walls and columns and does not differ between the floors.

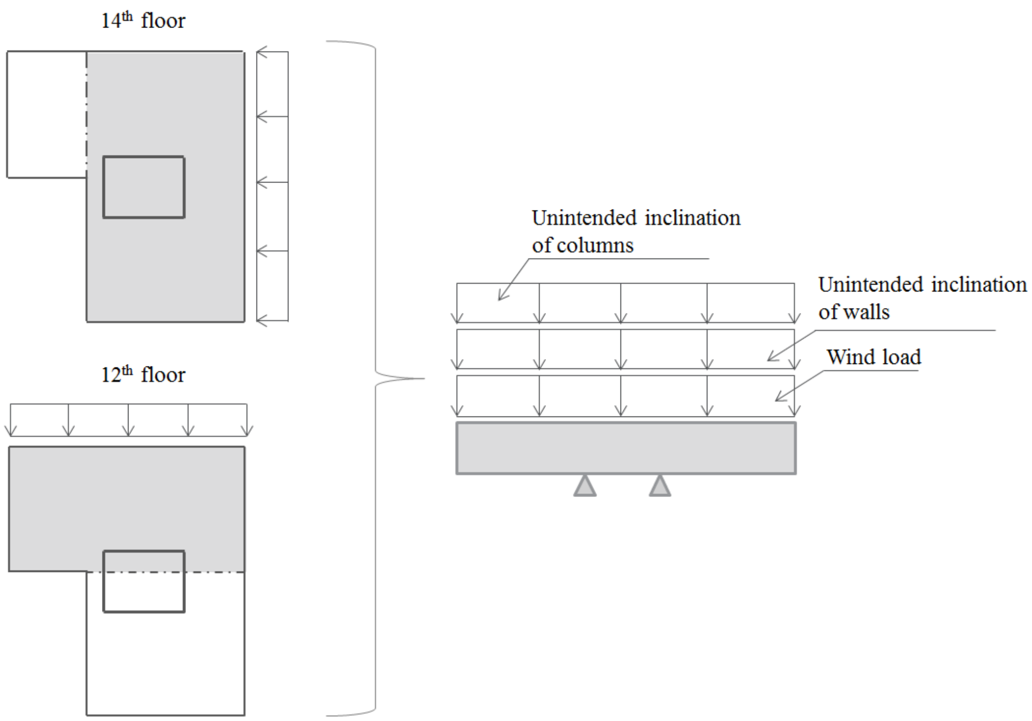


Figure 55 Model of the floors in the concept as a deep beam supported by the concrete core with two supports. Two cantilever parts.

When the east façade (the upper model in Figure 55) is subjected to wind load, the floor on the 14th storey is the worst loaded. This is since the influence area for the floor is largest at this storey, because the influence area is higher than for the other storeies. On the other hand, the 12th storey is the most loaded when the north façade is subjected to wind load. This is because the cantilever that is to the left side of the concrete core is longer than to the right side. For the 13th and 14th storeies part A in Figure 56 does not exist; therefore the slab on the 12th storey is subjected to the highest wind loads.

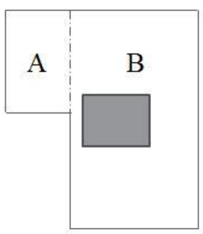


Figure 56 Notation for different parts of the building where part A reaches the 12th storey and part B reaches the 14th storey.

For this specific case the largest moment and shear forces occur at the supporting wall. The bracing wall needs to take all the load acting on the cantilever. In Figure 57 the model of the deep beam is illustrated. The deep beam continues over the support.

The cassette floor is provided with two steel ties and can thereby be modelled as a I-beam, where the connected cassette floor elements correspond to the web and steel edge beams correspond to the flanges, see Figure 57. The flanges resist the moment by enabling a force couple, while the web resist shear forces.

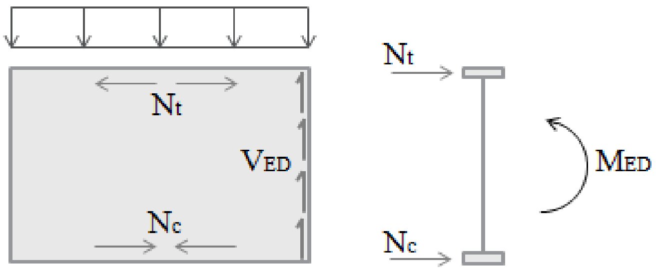


Figure 57 Forces in the deep beam.

The effect from unintended inclination differs between the 14th storey and the 12th storey. This is since part A of the building only exists up to the 12th storey. Therefore the number of interacting vertically loaded members is less for the 14th storey. Less vertically loaded members result in a larger unintended inclination of the building; hence the equivalent load effect becomes larger.

7.4.3 Results for the timber cassette floor

Table 50 shows the results for the steel tie. The dimensions of the steel tie provide such capacity that the utilisation ratio in ULS is below 45 % for the two cases, wind from north and wind from east. The ties were designed with a low utilisation ratio in order to prevent too large strains. Table 50 also shows the elongation of the steel. The elongation is elastic, since the stresses are below the yield limit for the steel tie. Appendix D6 and D7 presents the performed calculations.

Table 50 Dimensions, utilisation ratio and elongation of the steel tie.

	Size [mm]	Utilisation in tension	Elongation
Wind from east	6×65	42.7 %	0.07 %
Wind from north	6×65	38.3 %	0.06 %

When the east façade is subjected to wind load the largest shear force between two floor elements is 5.54 kN/m. It was assumed that the two floor elements are connected by nails in two rows according to the detailing shown to the left in Figure 54. The needed dimensions and spacing of these nails are shown in Table 51.

Table 51 Nails used in the connection between two floor elements.

	Length [mm]	Diameter [mm]	Spacing [mm]	Shear force [kN/m]	Utilisation
Wind from east	75	5	200 (2 rows)	5.54	49.2 %

From Table 50 and Table 51 it can be concluded that the connection between the floor elements has sufficient capacity to resist the shear forces. By optimising the connection a higher shear capacity and hence a lower utilisation ratio can be obtained. Since sufficient capacity of the connections was easy to obtain for the case when the east façade is subjected to wind, the effect from wind from north was not investigated.

Figure 58 shows the connection between the floor elements and the concrete core that was assumed. The timber floor is connected to the concrete core by an L-shaped steel plate. The shape of the holes for the screws is elliptical to allow for vertical movements and to prevent horizontal movements. Both the fastener between the steel plate and the concrete and between the steel plate and the timber floor need to be designed to resist shear forces.

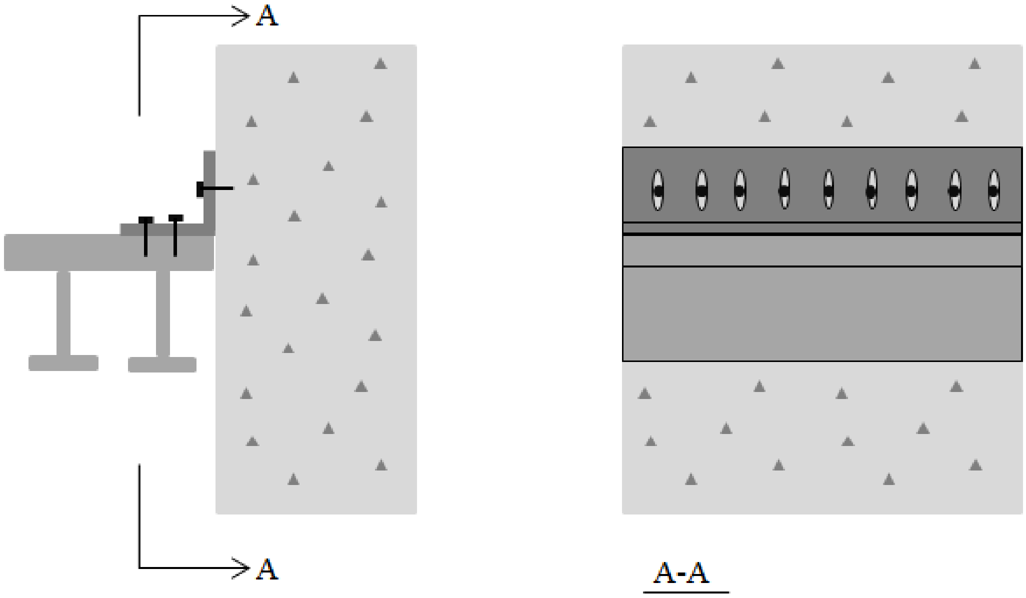


Figure 58 Detail of how the floor could be connected to the concrete core.

The capacity of the FBS screws used between the steel plate and the concrete core was determined from Fischer (2015). For the connection to the timber floor plate the same nail as in Table 51 was assumed. In Table 52 the results from the core when the east façade is subjected to wind are presented and in Table 53 on the other hand the results from the core when the north façade is subjected to wind is presented.

Table 52 Dimensions and utilisation ratios for the fasteners when the east façade is subjected to wind load.

	Length [mm]	Diameter [mm]	Spacing [mm]	Utilisation
Steel-concrete connection	115	12.5	250	24.4 %
Steel-timber connection	75	5	150 (2 rows)	86.4 %

Table 53 Dimensions and utilisation ratios for the fasteners when the north façade is subjected to wind load.

	Length [mm]	Diameter [mm]	Spacing [mm]	Utilisation
Steel-concrete connection	115	12.5	250	26.4 %
Steel-timber connection	75	5	250	93.5 %

From these results it can be concluded that the floor and its connections have sufficient capacity with regard to lateral loads. Therefore the capacity of the floor is not considered to be problematic for the concepts.

7.5 Dynamic behaviour of high-rise structures

During the preliminary design of high-rise buildings it is important to consider whether the structure has a static or dynamic response. It is very expensive to influence the behaviour of the structure, after it has been constructed, and therefore it is of importance to foresee if the structure has a dynamic response or not.

According to the results presented in Section 7.1 there is a significant difference between the total weight of the reference building and the total weight of the buildings in concepts 3 and 4. In Section 7.5.3 it is shown that the natural frequency and thereby the acceleration of a building partly depends on the weight of the building. Hence it was of interest to investigate the differences between the dynamic responses.

Unless another source is specified the information is taken from Stafford Smith & Coull (1991).

7.5.1 Dynamic analysis

The movements of a tall building can be divided into static motions and dynamic motions. Static motions are caused by slowly applied forces such as the long term component of wind load, gravity load or thermal effects, while dynamic movements are caused by short-term impact loads. Examples of dynamic actions are short period wind loads, seismic accelerations and vibrations from machinery. The two latter effects were not considered in this project.

There are several factors that affect the dynamic response of buildings, the most important factors are stated below.

- Applied load on the structure
- Geometry of the structure
- Mass of the structure
- Stiffness of the structure
- Damping of the structure

The criteria of when a dynamic analysis of a building is required differ between different codes. According to Stafford Smith & Coull (1991) the Australian Code states that a dynamic analysis is required if the height to width ratio of the structure is more than 5 or if the natural frequency in the first mode of vibration is less than 1 Hz. Boverket (1997) on the other hand has the same criteria for the height to width ratio but states that a dynamic analysis is required if the eigenfrequency is lower than 3 Hz.

Generally, the stiffer a building is, the higher the first natural frequency becomes. A stiff structure will follow the shifting wind load without an accumulation of the lateral deflections and therefore the design parameter is the static load effect. For a flexible structure on the other hand, the first natural frequencies are often low and thereby closer to the frequency of the wind. In this case the building will tend to follow the shifting wind load with an accumulation of the lateral deflections; hence the building may start to oscillate.

The major effect that has to be considered when designing buildings subjected to dynamic loads is the acceleration. Lateral acceleration of a structure occurs when a building is suddenly loaded in the lateral direction. The effects of lateral acceleration are described further in Section 7.5.4.

7.5.2 Lateral deflection

Both the reference building and Concepts 3 and 4 are stabilised by a concrete core in the middle of the building. The stabilising member is considered to be slender. According to Section 3.1.2 a slender building subjected to lateral load is behaving predominantly in flexure. The lateral deflection of a structure behaving mainly in flexure can be calculated according to Equation (25), which can be derived from (Engström, 2011).

$$u(z_1) = \int_0^{x_1} \frac{M(z_1)}{EI(z_1)}(z_1 - z)dz \tag{25}$$

$u(z_1)$	Deflection in the wind direction at the height z_1
$M(z)$	Bending moment at the height z_1
E	Elastic modulus of the material in the member
$I(z_1)$	Second moment of inertia of the stabilising core at the height z_1
z	Coordinate along the longitudinal axis

Whether the lateral deflection at the top of the building is acceptable or not can be evaluated with a drift index. There is no specific limit for the drift index, but an example of a drift index is shown in Equation (26).

$$\frac{u_{max}}{H} \leq \frac{1}{500} \rightarrow \ u_{max} \leq \frac{H}{500} \tag{26}$$

u_{max}	Maximum deflection at the top of the building
H	Height of the building

In general the limit of the maximum lateral deflection is lower for residential buildings than for office buildings. However, the dynamic comfort criterion is not automatically fulfilled, when the criterion for the lateral deflection is fulfilled.

7.5.3 Natural frequency

An approximate value for the fundamental frequency can be determined according to Equation (27). The equation is based on Rayleigh’s method and takes differences in the applied load and weights of different storeys into account. For the wind load, F_i, the statically equivalent load due to wind is to be used.

$$f_n = \frac{1}{2\pi}\sqrt{\frac{g\sum F_i u_i}{\sum W_i {u_i}^2}} \tag{27}$$

F_i	Equivalent lateral load at the i:th storey
u_i	Lateral deflection at the i:th storey
W_i	Weight of the i:th storey

It can be seen in Equation (25) and Equation (27) that the factors influencing the dynamic response presented in Section 7.5.1 have an influence on the natural frequency of buildings. A structure with a high stiffness has a lower lateral deflection then a structure with low stiffness. A low lateral deflection results in a higher natural frequency. In addition the lower the mass of the structure is, the higher the eigenfrequency becomes.

7.5.4 Acceleration

A high-rise building can experience two types of accelerations, along-wind accelerations and cross-wind accelerations. When performing a check of the dynamic structural response of buildings due to wind load, both types need to be considered. The along-wind induced accelerations can be analysed by the gust factor method which provides an estimation of the peak dynamic response. For the cross-wind acceleration there is no corresponding method available. The most accurate method is to perform wind tunnel tests. Nevertheless, a rough estimation can be made by empirical formulas. Even though the cross-wind accelerations might become larger and more critical only the along-wind accelerations were considered in this project. This is since this method is more accurate and since the aim was to evaluate the consequences on the dynamic response when implementing timber in the structural system.

The equations for the maximum acceleration differ between different parts of Eurocode. For the calculations made in this project part 1-4 in Eurocode 1, CEN (2008c) and The Swedish National Annex, Boverket (2013), were used. The maximum along-wind acceleration can be determined from Equation (28).

$$\ddot{x}_{max}(z) = k_p \sigma_{\ddot{x}}(z) \tag{28}$$

k_p	Peak factor
$\sigma_{\ddot{x}}(z)$	Standard deviation of the acceleration

The standard deviation of the acceleration is calculated according to Equation (29).

$$\sigma_{\ddot{x}}(z) = \frac{3 I_v(z) R q_m(z) b c_f \phi_{1,x}(z)}{m} \tag{29}$$

$I_v(z)$	Turbulence intensity at the height z
R	Factor for the resonance response
$q_m(z)$	Wind velocity pressure at the height z for a return period of 5 years
b	Width of the building perpendicular to wind direction
c_f	Force coefficient factor
$\phi_{1,x}(z)$	Deflected modal shape of the building
m	The equivalent mass of the building per unit area

The peak factor is determined according to Equation (30).

$$k_p = \sqrt{2\ln(vT)} + \frac{0.6}{\sqrt{2\ln(vT)}} \tag{30}$$

v	Up-crossing frequency
T	Averaging time for the mean wind velocity

A higher mass and higher damping result in a lower acceleration. Consequently, a higher natural frequency results in a higher acceleration. Hence, the acceleration of a structure can be limited by ensuring that the structure has a sufficient weight or damping properties. In addition a slight reduction of the acceleration can be achieved by increasing the stiffness of the structure.

If the dynamic movements of a structure are too large, the occupants of the building can experience anxiety and nausea and the building can be considered to have insufficient comfort. When evaluating the human response to vibration of a structure the acceleration is the most important parameter. Today, there are no specific limits for comfort criteria.

In Table 54 approximate values of the human behaviour and motion perception for different ranges of accelerations are described. The values and information in Table 54 are directly taken from Stafford Smith & Coull (1991).

Table 54 Description of the human behaviour and motion perception for different range of accelerations.

Acceleration [m/s^2]	**Human perception**
< 0.05	Humans can not perceive motions
0.05-0.10	Sensitive people can perceive motions. Hanging objects may move slightly.
0.10-0.25	Majority of people will perceive motion. Level of motion may affect desk work. Long-term exposure may produce sickness.
0.25-0.40	Desk work becomes difficult or almost impossible. Ambulation still possible.
0.40-0.50	People strongly perceive motion. Difficult to walk naturally. Standing people might lose balance.
0.50-0.60	Most people cannot tolerate motion and are unable to walk naturally.
0.60-0.70	People cannot walk or tolerate motion.
>0.85	Objects begin to fall and people may be injured.

7.5.5 Results from the dynamic analysis

As can be seen in Table 55 the natural frequencies of both the reference building and the buildings in Concepts 3 and 4 are below 3 Hz; hence a dynamic analysis is required according to Boverket (1997). In this project the dynamic analysis was limited to a check of the along wind induced acceleration. For more details about the calculations the reader is referred to Appendix D8. It should be noted that only Concept 3 was evaluated. This is since the concepts are similar, but Concept 3 is somewhat lighter.

Table 55 Lateral top deflection, natural frequency and along-wind accelerations for the reference building and Concept 3, where the acceleration is calculated for a wind with a return period of 5 years.

	Lateral top deflection		Natural frequency		Along wind induced acceleration, 5 year-wind	
	Wind from north	**Wind from east**	**Wind from north**	**Wind from east**	**Wind from north**	**Wind from east**
Reference building	38.50 mm	10.71 mm	0.67 Hz	1.38 Hz	0.054 m/s^2	0.027 m/s^2
Concept 3	36.90 mm	9.54 mm	1.13 Hz	2.41 Hz	0.070 m/s^2	0.032 m/s^2

As can be seen in Table 55 the lateral deflections differ between the reference building and the concepts even though the concrete cores have the same stiffness. The difference occurs, since the equivalent horizontal load effects from unintended inclination in the reference building are larger than for the concepts. However, the higher self-weight of the reference building results in lower natural frequencies compared to the concepts. The lateral deflections fulfil the criterion given in Equation (26), which is the height of the building divided by 500, approximately a limit of 105 mm.

The most important effect is however the acceleration of the building. When the buildings are subjected to wind, with a return period of 5 years from east, the reference building and the buildings in Concepts 3 and 4 obtain almost the same acceleration, which is in the order that humans cannot perceive. On the other hand, when the buildings are subjected to wind from north, there is some difference in the acceleration between the reference building and the concepts. The reference building is close to the limit where humans can not perceive the motion; only the very sensitive humans can perceive the motion. For the concepts sensitive humans can perceive the motions. It is therefore important to consider the acceleration in timber buildings, because the acceleration will increase in comparison to similar but heavier concrete buildings.

In Bjertnæs & Malo (2014) the accelerations for the timber building Treet, in Bergen, are presented. Bjertnæs and Malo have calculated the accelerations from a wind with a return period of 1 year. The Swedish National Annex, Boverket (2013), states that a wind with a 5 year return period should be used for acceleration calculations. However, from the data presented in Bjertnæs & Malo (2014) the factor used to calculate the wind with a 1 year return period could be determined. The accelerations for the reference building and Concept 3 using a wind with 1 year return period could be calculated and compared to the accelerations obtained in Treet, see Table 56.

Table 56 Top accelerations for the reference building, Concept 3 and Treet in Bergen, Norway, for a wind with a return period of 1 year.

	Along wind induced acceleration, 1 year-wind	
	North	**East**
Reference building	0.034 m/s^2	0.016 m/s^2
Concept 3	0.043 m/s^2	0.019 m/s^2
Treet, in Bergen	0.051 m/s^2	0.048 m/s^2

As can been seen in Table 56 the accelerations obtained for the reference building and Concept 3 are lower than the accelerations for Treet. However the accelerations are still in the same magnitude.

8 Analysis of promising timber concepts

The two promising concepts were evaluated with regard to the objectives of the project. The analysis concerns the consequences regarding sectional forces, global equilibrium, needed size of the load bearing elements, the layout of the floor plan, differences in vertical displacements and the dynamic response of the structures. In the discussion there are also general remarks about the consequences and possibilities of choosing timber instead of concrete and steel.

8.1 Size of individual members and layout of floor plan

From the component study and the development of structural systems it can be concluded that timber elements can be used for all structural components when regarded as isolated elements. All components investigated were designed with regard to the ultimate limit state and the serviceability limit state. However, a consequence of using timber is that the dimensions become larger than for concrete and steel. Hence, the spans need to be limited in order to obtain acceptable dimensions. Nevertheless, it is important to have in mind that the global performance of the structural systems needs to be considered as well. The discussion concerning consequences of the global behaviour is presented in the following sections.

According to Section 5.4 it was concluded that timber floors are suitable to use provided that the floor span is limited to approximately 8 metres. In cases where there are requirements of large spans, which often are the case in office buildings, more beam-column lines might be needed as an outcome of the limited floor span. More beam-column lines result in additional columns, which make the layout less flexible. However, it is important to be aware of that more columns do not necessarily mean an interference with the floor plan. When developing the structural systems it was seen that the columns could be arranged differently compared to the reference building, creating shorter span and still not interfering with the activity in the building. Still, more beams and columns are used in the developed concepts compared to the reference building. A close collaboration with the architect and the client can support a good design that enables an open floor plan, even though more beam-column lines are used.

The requirements of a building differ depending on the intended applications of the building. For example office buildings usually demands larger floor spans compared to residential buildings. The imposed load is the same for a residential building and an office building. Altogether this can make residential buildings more suitable for timber floors. This is since the spans for timber floors need to be limited in order to fulfil requirements regarding the serviceability limit state and in order to reduce the height. On the other hand, a residential building has higher requirements concerning vibrations and sound insulation and thereby sufficient performance can be harder to obtain.

To use timber floors instead of concrete floors is almost a prerequisite, if timber beams and timber columns are to be used in a structural system. This is since the weight of a concrete floor is more than twice the weight of a timber floor. If choosing a concrete floor the dimensions of the beams and columns would need to increase or shorter spans and more columns would be needed in order to limit the dimensions

Timber beams proved to need considerably higher sections than steel beams and therefore steel is considered to be the best option when considering the beams. It is important to have in mind that the aim is to develop a structural system that is efficient. The use of steel beams instead of timber beams enables larger spans without a consequence of high sections.

On the other hand the consequence regarding dimensions when using CLT-walls proved to be relatively small. They became somewhat larger than the corresponding concrete walls but an increase of the thickness of a wall does not necessarily mean a reduction of the floor area.

8.2 Vertical displacements

The results for the vertical displacements in Section 7.3.2 show that it may be problematic to combine a timber system with a stabilising concrete core. For both concepts the concrete core has relatively small vertical displacements in comparison to the vertical displacements of the timber systems. However, the results obtained are theoretical and could probably be decreased by controlling the environment of the timber products and choosing the material more carefully. Still it should be noticed that there is a risk of large differences in vertical displacements and thus measures should be taken to reduce negative consequences. The promising concepts need to be provided with connections between the concrete core and the timber system that allow different vertical movements and still resist horizontal forces. This could compose a difficulty when choosing a mixed structural system with a concrete core and a timber system. Another solution is to avoid a concrete core and brace the system with a timber framework or using timber shear walls instead. The vertical displacement would still be of the same magnitude, but the difference between the bracing system and the vertically load resisting system would be smaller.

In the component study dimensions for diagonal and chevron bracings are presented. The bracings were designed with regard to wind load acting on a fictive 15 storey building. It could be seen that the dimensions were similar to the ones in Treet, even though the conditions differed. This indicates the possibility of using timber in a bracing frame structure to obtain global equilibrium without undesirable differences in the vertical displacements. However, it is important to consider that such solution becomes lighter than a system with a concrete core, which may result in consequences such as need of anchorage and higher accelerations. The consequences of having a light structure are discussed further in Section 8.4.

8.3 Capacity of timber cassette floor elements

Generally a concrete floor is considered to have sufficient stiffness to transfer lateral loads to the bracing members. Timber cassette floors on the other hand do not have the same stiffness and therefore it was of interest to check the in-plane response of timber cassette floor elements. The results from the analysis of the cassette floor proved that it has sufficient capacity to resist the effect from lateral loads. This provided that the connections between floor elements and between the floor structure and the core have sufficient capacity. However, it is important to have in mind that the results only are valid for the specific floor investigated.

Nevertheless, it can be concluded that using timber cassette floors may not be a problem as long as the engineer makes a careful design of the connections. The connections used in the project could easily be more efficient by using other fasteners, decreasing the spacing and/or by using a different solution.

8.4 Weight of the structure and sectional forces

The two promising concepts became as expected lighter than the reference building. From the results it can be seen that the reduction of the weight was in the order of 50 % compared to the reference building.

A positive aspect of having a lighter building is that the foundation work can become less extensive, especially if the building is located on soil with poor load bearing capacity such as clay. In such areas the amount of piles could be reduced and/or smaller piles could be used. A less extensive foundation work is positive with regard to the construction time, economy and environmental impact.

On the contrary, a negative aspect of having a lighter building is that the global equilibrium and an acceptable dynamic behaviour might be harder to obtain. Regarding the global equilibrium, for a lighter building there may be a higher risk for tensile stresses in the bracing members or tilting of the building is not anchored properly. However, for the buildings investigated in this project the effect of having a lighter building was not large. The results show that the concrete core in all the concepts would experience somewhat larger tensile stresses than the concrete core in the reference building. The increase of the tensile stresses is small, since the tributary area of the floor that is resisted by the core is small; hence the concrete core itself is designed to fulfil global equilibrium. If choosing a timber system stabilised by a concrete core, the aim should be to load the core as much as possible with vertical load in order to decrease the tensile stresses in the core.

Nevertheless, in cases where a building is not stabilised by a heavy concrete core there might be problems with the global equilibrium, if no other measures are taken. The risk of tilting can be prevented by anchoring the bracing members to the foundation and/or by increasing the vertical load on the bracing members.

8.5 Dynamic response

Since the human perception of motion is individual, there are no exact limits for the maximum allowed acceleration in a building. It should also be noted that acceptable limits of acceleration differ depending on the application of the building. A higher acceleration is generally accepted in an office building compared to a residential building.

The accelerations obtained in this project can be considered to be acceptable with respect to the approximate values of human perception of motion given in Table 54. In addition the values obtained in the project coincide with the values of the accelerations in the building Treet in Norway. However, it should be noted that it is possible that the cross-wind acceleration are more severe. Nevertheless, it could be concluded that, since the mass of the buildings in the promising concepts is significantly lower than for the reference building, the acceleration increases. Therefore it is important that engineers are aware of that a building that utilises a mixed structural system might have more problems with the dynamic response than a concrete building of the same height, especially if timber is used to a great extent. For this project the effect of the lateral loads is resisted by a concrete core with a high stiffness. If the building is braced with timber members, the stiffness might be decreased leading to increasing accelerations.

Furthermore, the reader should be aware of that the values obtained in the dynamic analysis are approximate. In order to obtain more accurate results a FEM model or wind tunnel tests are required. However, that was beyond the scope of this project.

9 Conclusions and recommendations

The aim of the project was to develop possible solutions for mixed structural systems for medium high-rise office buildings and evaluate the consequences of implementing timber in the structural system. In this chapter conclusions and recommendations for design of such buildings are presented with regard to the objectives of the project.

9.1 Consequences regarding needed size of load bearing elements

- From the component analysis it could be concluded that the needed size of timber members always becomes larger than for corresponding steel and concrete members. However, it was shown that it was possible to use timber members for all components in a 14- storey office building with regard to the required resistance and performance in the ultimate limit state and the serviceability limit state.
- The effective span of timber floors should be limited to 8-9 metres in order not to limit the acceptable structural heights of floors. As a consequence of reducing the effective floor span more interior beam-column lines are needed.
- Using timber floors instead of concrete floors is almost a prerequisite, if timber beams and timber columns are to be used in a mixed structural system. Otherwise the dimensions of the beams and columns become too large.
- For office buildings where long spans are demanded, it may be better to use steel beams instead of timber beams. This is since the cross-section of timber beams becomes very high. Timber beams are better suited in buildings where the spans can be limited, for example residential buildings.
- The results indicate that CLT-walls may be a possible alternative to concrete walls. This provided that timber floors are used in order to limit the permanent loads on the walls.

9.2 Consequences regarding sectional forces and global equilibrium

- If a lighter material such as timber is to be used for the bracing system, it is important to ensure that lifting of the structure is prevented.
- It is beneficial that the bracing walls resist as much vertical load as possible in order to prevent the need for tensile capacity and anchorage due to the effect from lateral loads.

9.3 Consequences regarding vertical displacements

- The major challenge with a system composed of a concrete core and timber systems are the different vertical displacements. Either the connections need to allow for different vertical displacements or different displacements need to be avoided.

- Compressive stresses perpendicular to the grain should be avoided in order to limit the vertical displacements.

9.4 Consequences regarding the dynamic response

- Since the weight of a timber building is smaller compared to a corresponding concrete building, the acceleration becomes higher. It is therefore important that the engineer is aware of that unacceptable accelerations may arise in taller mixed structures with timber, even though a similar structure in concrete and steel usually poses no problem.
- For the dynamic response it is important to provide the building with sufficiently high mass and ensure that the stiffness of the bracing members is high.

9.5 General conclusions and recommendations

- It proved to be hard to advocate timber in a mixed structural system by only considering its structural properties. If timber is to be used to a great extent, it may be necessary to highlight other aspects such as a less extensive foundation work, architectural expression, traditions and/or environmental and social benefits.
- A prerequisite for timber to be advocated is that the use of timber does not contribute to any major problems and additional costs compared to a conventional solution in steel and concrete. It is therefore important to ensure that timber is used where it is best suited and that there are simple and elaborated solutions available.
- However, the investigations made in this project indicate that using timber in a mixed structural system for medium high-rise buildings is highly possible. Still it is important that the engineers are aware of the problems that might arise, when a structure becomes lighter, less stiff and when combining different materials. It is also important with a close collaboration between clients, architects and engineers so that any problems can be foreseen and handled in an early stage.

9.6 Recommendations for further investigations

In the list below recommendations for further investigations concerning the consequences of implementing timber in medium high-rise buildings are presented.

- For this project a preliminary design was performed for a structural system with a beam-column system in timber and steel and a bracing system in concrete. If timber is to be used more in the future, it would be of interest to investigate the consequences of other arrangements of structural systems. Examples of systems that could be investigated further are frame structures in timber such as the one used in the timber building Treet or systems braced with timber shear walls.
- The analysis of the vertical displacements showed that differences in vertical displacements may be a challenge in mixed structural systems. For future work it would be of interest to analyse the consequences of vertical displacements more thoroughly. In addition it would be of interest to investigate various measures that can be performed in order to prevent large differences in vertical displacements and connections that can allow for different vertical movements still transferring horizontal forces.
- For the type of structural system investigated in this project it was not a problem with lifting due to low tensile stresses in the bracing members. However, this might be a problem, if a heavy concrete core is not used. Therefore it is of interest to investigate the consequences regarding global equilibrium for other types of mixed structures. For example mixed structures with timber shear walls or mixed structures braced by timber frames.
- A positive aspect of using timber to a great extent is that the building becomes considerably lighter and thereby the foundation work can become less extensive. Therefore it would be of interest to investigate the savings that can be achieved with regard to economy and environment if using timber in a mixed structural system. By highlighting the economic and environmental benefits of a less extensive foundation work, as a result of using timber in the structural system, it can be easier to advocate such systems for a client.
- Since it could be seen in the dynamic analysis that a lighter building influenced the accelerations negatively, it may be of interest to perform a deeper analysis of this. Especially for a building stabilised with timber instead of a heavy concrete core, since the dynamic response also depends on the stiffness.
- In this project only the structural consequences of implementing timber have been evaluated. Even though it is possible to implement timber into medium high-rise buildings it proved to be hard to advocate a structural system where timber is used to a great extent if only considering its structural properties. Thus, it would be of interest to also investigate the environmental aspects and cost efficiency, both during construction and during the service life, as an outcome of implementing timber into structural systems.

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10.4 Interviews

Kliger R. (2015-02-06): Professor in timber engineering at Chalmers University of Technology, Interview, Göteborg.

Kliger R. (2015-03-12): Professor in timber engineering at Chalmers University of Technology, Interview, Göteborg.

Kliger R. (2015-04-29): Professor in timber engineering at Chalmers University of Technology, Interview, Göteborg.

Appendix A1: Columns

$kNm := kN \cdot m$

In this appendix the procedure of how columns have been calculated is presented. The results are presented in Section 5.2.

A1.1 Vertical load

The columns are designed to withstand some loads. These loads are aprroximately loads that can arise in columns for a building with 15-storeys, if using timber floors.

$ii := 0..7$

8 different ULS loads for the columns $P := \begin{pmatrix} 0.5 \\ 1 \\ 2 \\ 3 \\ 4 \\ 5 \\ 6 \\ 7 \end{pmatrix} MN$

A1.1.1 Fire load case

Load relation between fire load case and the ULS load case $r := 0.533$

Load fire case, 8 different loads $P_{fire_{ii}} := r \cdot P_{ii}$

$$P_{fire} = \begin{pmatrix} 0.267 \\ 0.533 \\ 1.066 \\ 1.599 \\ 2.132 \\ 2.665 \\ 3.198 \\ 3.731 \end{pmatrix} \cdot MN$$

Charring depth for 90 minutes $d_{char.0} := 0.65 \frac{mm}{min} \cdot 90min = 58.5 \cdot mm$

A1.1.2 Height of the column

Buckling length (Assumed to be the same as one storey) $h_{column} := 3.6m$

A1.2 Timber columns

Calcualtions have been performed according to **SS-EN 1995-1-1:2004**. All references made refers back to this eurocode

A1.2.1 Dimensions

A1.2.1.1 Glulam columns

$i := 0..7$

The dimensions were changed during the project to those that was of interest. Here the values for the presented in Section 5.2 are shown but the same document was used when performing calculations that are presented in Chapter 6.

Width of the beam

$$w_{glulam} := \begin{pmatrix} 2\cdot 140 \\ 2\cdot 165 \\ 2\cdot 165 \\ 2\cdot 215 \\ 2\cdot 215 \\ 3\cdot 190 \\ 3\cdot 215 \\ 3\cdot 215 \end{pmatrix} mm$$

Width in case of fire, after 90 minutes

$$w_{fire_i} := w_{glulam_i} - 2\cdot d_{char.0} \qquad w_{fire} = \begin{pmatrix} 0.163 \\ 0.213 \\ 0.213 \\ 0.313 \\ 0.313 \\ 0.453 \\ 0.528 \\ 0.528 \end{pmatrix} m$$

Height of the beam

$$h_{glulam} := \begin{pmatrix} 6\cdot 45 \\ 6\cdot 45 \\ 8\cdot 45 \\ 9\cdot 45 \\ 12\cdot 45 \\ 12\cdot 45 \\ 12\cdot 45 \\ 14\cdot 45 \end{pmatrix} mm$$

Height in case of fire, after 90 minutes

$$h_{fire_i} := h_{glulam_i} - 2\cdot d_{char.0} \qquad h_{fire} = \begin{pmatrix} 0.153 \\ 0.153 \\ 0.243 \\ 0.288 \\ 0.423 \\ 0.423 \\ 0.423 \\ 0.513 \end{pmatrix} m$$

Area of the beam section $A_{glulam_i} := w_{glulam_i} \cdot h_{glulam_i}$ $A_{glulam} = \begin{pmatrix} 0.076 \\ 0.089 \\ 0.119 \\ 0.174 \\ 0.232 \\ 0.308 \\ 0.348 \\ 0.406 \end{pmatrix} m^2$

A1.2.1.2 Solid wood columns

$n := 0..6$

Here, common dimensions for solid wood columns have been used. Only the greatest dimension have enough capacity to resist 0.5 MN. Se further down.

Dimension of one side of the beam $a_{solid} := \begin{pmatrix} 95mm \\ 120mm \\ 145mm \\ 170mm \\ 195mm \\ 220mm \\ 225mm \end{pmatrix}$

Area of the beam section $A_{solid_n} := \left(a_{solid_n}\right)^2$ $A_{solid} = \begin{pmatrix} 9.025 \times 10^{-3} \\ 0.014 \\ 0.021 \\ 0.029 \\ 0.038 \\ 0.048 \\ 0.051 \end{pmatrix} m^2$

A1.2.2 Material data for timber

A1.2.2.1 Characteristic strenght value

Compression parallell to grain $f_{c.0.k.glulam} := 30.8MPa$ (Assume strenght class L40c)

$f_{c.0.k.solid} := 26MPa$ (Assume strength class C40)

Elastic modulus $E_{0.05.glulam} := 13000MPa$

$E_{0.05.solid} := 9400MPa$

Partial factors: $\gamma_{M.glulam} := 1.25$

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

Assuming long term load and service class 2

Strength modification factors $k_{mod.glulam} := 0.7$

$k_{mod.solid} := 0.7$

Deformation modification factors

$$k_{def} := 0.8$$

Effect of member size

eq. 3.2 in section 3.3

$$k_{h.glulam.y_i} := \begin{cases} \min\left[\left(\frac{600mm}{h_{glulam_i}}\right)^{0.1}, 1.1\right] & \text{if } h_{glulam_i} \le 600mm \\ 1 & \text{otherwise} \end{cases}$$

$$k_{h.glulam.z_i} := \begin{cases} \min\left[\left(\frac{600mm}{w_{glulam_i}}\right)^{0.1}, 1.1\right] & \text{if } w_{glulam_i} \le 600mm \\ 1 & \text{otherwise} \end{cases}$$

eq. 3.1 in section 3.2

$$k_{h.solid_n} := \begin{cases} \min\left[\left(\frac{150mm}{a_{solid_n}}\right)^{0.2}, 1.3\right] & \text{if } a_{solid_n} \le 150mm \\ 1 & \text{otherwise} \end{cases}$$

A1.2.2.2 Design strenght values for ULS

Compression parallell to grain

$$f_{c.0.d.glulam.y_i} := k_{mod.glulam} \cdot k_{h.glulam.y_i} \cdot \frac{f_{c.0.k.glulam}}{\gamma_{M.glulam}}$$

$$f_{c.0.d.glulam.z_i} := k_{mod.glulam} \cdot k_{h.glulam.z_i} \cdot \frac{f_{c.0.k.glulam}}{\gamma_{M.glulam}}$$

$$f_{c.0.d.solid_n} := k_{mod.solid} \cdot k_{h.solid_n} \cdot \frac{f_{c.0.k.solid}}{\gamma_{M.solid}}$$

A1.2.2.3 Design strenght values for the fire load case

Conversion factor for timber

$$k_{mod.fi} := 1.0$$

Modification factor for glulam

$$k_{fi.glulam} := 1.15$$

Partial factor for fire in timber

$$\gamma_{M.fi} := 1$$

Design load for fire load case

$$f_{d.fi} := k_{mod.fi} \cdot k_{fi.glulam} \cdot \frac{f_{c.0.k.glulam}}{\gamma_{M.fi}} = 35.42 \cdot MPa$$

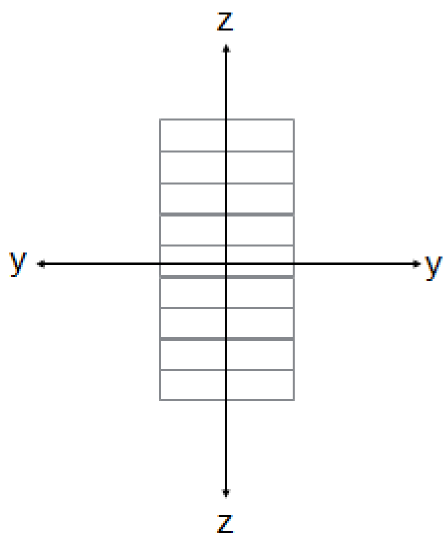
A1.2.3 Capacities to be checked

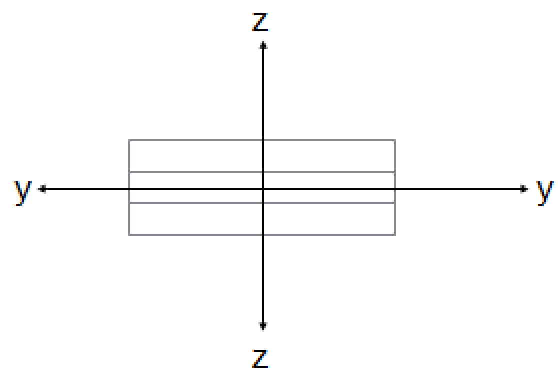
Modelling the diagonal bracing as a column subjected to compression, they should therefore fulfill the following expression:

$$\boxed{\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} \le 1}$$ eq. 6.23 in section 6.3

Critical axial load: $\boxed{N_{cr} := k_c \cdot f_{c.0.d} \cdot A}$

A1.2.3.1 Glulam column





Second moment of inertia and slenderness with respect to both directions.

$$I_{glulam.yy_i} := \frac{w_{glulam_i} \cdot \left(h_{glulam_i}\right)^3}{12}$$

$$I_{glulam.zz_i} := \frac{\left(w_{glulam_i}\right)^3 \cdot h_{glulam_i}}{12}$$

$$i_{glulam.y_i} := \sqrt{\frac{I_{glulam.yy_i}}{A_{glulam_i}}}$$

$$i_{glulam.z_i} := \sqrt{\frac{I_{glulam.zz_i}}{A_{glulam_i}}}$$

$$\lambda_{glulam.y_i} := \frac{h_{column}}{i_{glulam.y_i}}$$

$$\lambda_{glulam.z_i} := \frac{h_{column}}{i_{glulam.z_i}}$$

$$\lambda_{rel.glulam.y_i} := \frac{\lambda_{glulam.y_i}}{\pi} \cdot \sqrt{\frac{f_{c.0.k.glulam}}{E_{0.05.glulam}}} \qquad \lambda_{rel.glulam.z_i} := \frac{\lambda_{glulam.z_i}}{\pi} \cdot \sqrt{\frac{f_{c.0.k.glulam}}{E_{0.05.glulam}}}$$

$$\lambda_{rel.glulam.y} = \begin{pmatrix} 0.716 \\ 0.716 \\ 0.537 \\ 0.477 \\ 0.358 \\ 0.358 \\ 0.358 \\ 0.307 \end{pmatrix} \qquad \lambda_{rel.glulam.z} = \begin{pmatrix} 0.69 \\ 0.586 \\ 0.586 \\ 0.449 \\ 0.449 \\ 0.339 \\ 0.3 \\ 0.3 \end{pmatrix}$$

Reduction factor of the strength for both direction, kc.

$\beta_{c.glulam} := 0.1$

$$k_{glulam.y_i} := 0.5 \cdot \left[1 + \beta_{c.glulam} \cdot \left(\lambda_{rel.glulam.y_i} - 0.3\right) + \left(\lambda_{rel.glulam.y_i}\right)^2\right]$$ eq. 6.27 in section 6.3

$$k_{glulam.z_i} := 0.5 \cdot \left[1 + \beta_{c.glulam} \cdot \left(\lambda_{rel.glulam.z_i} - 0.3\right) + \left(\lambda_{rel.glulam.z_i}\right)^2\right]$$

$$k_{c.glulam.y_i} := \frac{1}{k_{glulam.y_i} + \sqrt{\left(k_{glulam.y_i}\right)^2 - \left(\lambda_{rel.glulam.y_i}\right)^2}}$$ eq. 6.25 in section 6.3

$$k_{c.glulam.z_i} := \frac{1}{k_{glulam.z_i} + \sqrt{\left(k_{glulam.z_i}\right)^2 - \left(\lambda_{rel.glulam.z_i}\right)^2}}$$

Critical axial load and the capacity fo the column in ULS

By the condition given for columns, $\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} \le 1$ the maximum compression stress can be calculated as:

$$\boxed{\sigma_{c.0.d} := k_c \cdot f_{c.0.d}}$$

Maximum stress with regard to y-direction $\quad \sigma_{c.0.d.glulam.y_i} := k_{c.glulam.y_i} \cdot f_{c.0.d.glulam.y_i}$

Maximum stress with regard to z-direction $\quad \sigma_{c.0.d.glulam.z_i} := k_{c.glulam.z_i} \cdot f_{c.0.d.glulam.z_i}$

Maximum axial load with regard to y-direction

$$N_{cr.glulam.y_i} := \sigma_{c.0.d.glulam.y_i} \cdot A_{glulam_i}$$

Maximum axial load with regard to z-direction

$$N_{cr.glulam.z_i} := \sigma_{c.0.d.glulam.z_i} \cdot A_{glulam_i}$$

Maximum axial load

$$N_{cr.glulam_i} := \min\left(N_{cr.glulam.y_i}, N_{cr.glulam.z_i}\right)$$

Critical axial load and the capacity fo the column in case of Fire

Maximum stresses with regard to y- and z-direction

$$\sigma_{c.0.d.fire.y_i} := k_{c.glulam.y_i} \cdot f_{d.fi}$$

$$\sigma_{c.0.d.fire.z_i} := k_{c.glulam.z_i} \cdot f_{d.fi}$$

Maximum axial load with regard to y- and z-direction

$$N_{y_i} := \sigma_{c.0.d.fire.y_i} \cdot A_{glulam_i}$$

$$N_{z_i} := \sigma_{c.0.d.fire.z_i} \cdot A_{glulam_i}$$

Maximum axial load

$$N_{cr.fire_i} := \min\left(N_{z_i}, N_{y_i}\right)$$

Check of the ULS load case

Capacity, Nrd

$$N_{cr.glulam} = \begin{pmatrix} 1.309 \\ 1.543 \\ 2.087 \\ 3.049 \\ 4.021 \\ 5.313 \\ 6.008 \\ 7.004 \end{pmatrix} \cdot MN$$

Applied load, Ned

$$P = \begin{pmatrix} 0.5 \\ 1 \\ 2 \\ 3 \\ 4 \\ 5 \\ 6 \\ 7 \end{pmatrix} \cdot MN$$

Utilisation ratio

$$\frac{P}{N_{cr.glulam}} = \begin{pmatrix} 0.382 \\ 0.648 \\ 0.959 \\ 0.984 \\ 0.995 \\ 0.941 \\ 0.999 \\ 0.999 \end{pmatrix}$$

Check of the Fire load case

$$N_{cr.fire} = \begin{pmatrix} 2.481 \\ 2.925 \\ 4.036 \\ 6.031 \\ 8.074 \\ 10.831 \\ 12.256 \\ 14.382 \end{pmatrix} \cdot MN$$

$$P_{fire} = \begin{pmatrix} 0.267 \\ 0.533 \\ 1.066 \\ 1.599 \\ 2.132 \\ 2.665 \\ 3.198 \\ 3.731 \end{pmatrix} \cdot MN$$

$$\frac{P_{fire}}{N_{cr.fire}} = \begin{pmatrix} 0.107 \\ 0.182 \\ 0.264 \\ 0.265 \\ 0.264 \\ 0.246 \\ 0.261 \\ 0.259 \end{pmatrix}$$

A1.2.3.2 Solid wood column

Second moment of inertia $I_{solid_n} := \dfrac{a_{solid_n} \cdot \left(a_{solid_n}\right)^3}{12}$

$$i_{solid_n} := \sqrt{\frac{I_{solid_n}}{A_{solid_n}}}$$

Slenderness $\lambda_{solid_n} := \dfrac{h_{column}}{i_{solid_n}}$

Relative slenderness $\lambda_{rel.solid_n} := \dfrac{\lambda_{solid_n}}{\pi} \cdot \sqrt{\dfrac{f_{c.0.k.solid}}{E_{0.05.solid}}}$ $\quad \lambda_{rel.solid} = \begin{pmatrix} 2.198 \\ 1.74 \\ 1.44 \\ 1.228 \\ 1.071 \\ 0.949 \\ 0.928 \end{pmatrix}$

Reduction factor for the strength, kc

$\beta_{c.solid} := 0.2$

$k_{solid_n} := 0.5 \cdot \left[1 + \beta_{c.solid} \cdot \left(\lambda_{rel.solid_n} - 0.3\right) + \left(\lambda_{rel.solid_n}\right)^2\right]$ <u>eq. 6.27 in section 6.3</u>

$k_{c.solid_n} := \dfrac{1}{k_{solid_n} + \sqrt{\left(k_{solid_n}\right)^2 - \left(\lambda_{rel.solid_n}\right)^2}}$ <u>eq. 6.25 in section 6.3</u>

Critical axial load and the capacity fo the column

By the condition given for columns, $\dfrac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} \leq 1$ the maximum compression stress can be calculated as:

$$\boxed{\sigma_{c.0.d} := k_c \cdot f_{c.0.d}}$$

Maximum stress $\boxed{\sigma_{c.0.d.solid_n} := k_{c.solid_n} \cdot f_{c.0.d.solid_n}}$

Maximum axial load $N_{cr.solid_n} := \sigma_{c.0.d.solid_n} \cdot A_{solid_n}$

Capacity, Nrd	Applied load, Ned
$N_{cr.solid} = \begin{pmatrix} 0.026 \\ 0.061 \\ 0.121 \\ 0.213 \\ 0.339 \\ 0.493 \\ 0.526 \end{pmatrix} \cdot MN$	$P = \begin{pmatrix} 0.5 \\ 1 \\ 2 \\ 3 \\ 4 \\ 5 \\ 6 \\ 7 \end{pmatrix} \cdot MN$

A1.3 Concrete columns

Columns have been designed according to Eurocode **SS-EN 1992-1-1:2005**.

A1.3.1 Dimensions

Height of the column $h_{column} = 3.6\ m$

Cross-section area of the column $A_c := 0.04m^2, 0.041m^2 .. 0.3m^2$

Sides lengths of the column, assuming quadratic cross section $b(A_c) := \sqrt{A_c}$ $\quad h(A_c) := \sqrt{A_c}$

Distances from top of cross-section to the reinforcement, concrete cover 50 mm. $d(A_c) := \sqrt{A_c} - 0.05m$

$d' := 0.05m$

A1.3.2 Material data for reinforced concrete

Partial factors $\gamma_c := 1.5$ (Concrete)

$\gamma_s := 1.15$ (Reinforcing steel)

A1.3.2.1 Concrete N 30/37

Concrete strength $f_{ck} := 30MPa$ $\quad f_{cd} := \dfrac{f_{ck}}{\gamma_c} = 20 \cdot MPa$

Mean strength $f_{cm} := 38MPa$

Elastic modulus $E_{cm} := 33GPa$

Design value for elastic modulus $\gamma_{cE} := 1.2$ $\quad E_{cd} := \dfrac{E_{cm}}{\gamma_{cE}} = 27.5 \cdot GPa$

A1.3.2.2 Reinforcement B500B

Reinforcement strength $f_{yk} := 500MPa$ $\quad f_{yd} := \frac{f_{yk}}{\gamma_s} = 434.783 \cdot MPa$

Elastic modulus $E_s := 200GPa$

Strain limit for reinforcement $\varepsilon_{yd} := \frac{f_{yd}}{E_s} = 2.174 \times 10^{-3}$

Assume 16 mm reinforcement bars $\phi := 16mm$

Area of one bar $A_{si} := \pi \cdot \left(\frac{\phi}{2}\right)^2 = 201.062 \cdot mm^2$

A1.3.2.3 Creep coefficient

Final creep coefficient $\boxed{\varphi_{ef} := \varphi_{RH} \cdot \beta(f_{cm}) \cdot \beta(t_0)}$ eq. B.1 in Appendix B

Indoor environment, assumed $RH := 50\%$

Notional size
(assuming quadratic cross-section)

$$h_0(A_c) := \frac{2 \cdot A_c}{2 \cdot 2 \cdot \sqrt{A_c}}$$

Factor to allow for the effect of relative humidity $f_{cm} > 35MPa$

$$\varphi_{RH}(A_c) := \left[1 + \frac{1 - RH}{0.1 \sqrt[3]{h_0(A_c) \cdot \frac{1000}{m}}} \cdot \left(\frac{35MPa}{f_{cm}}\right)^{0.7}\right] \left(\frac{35MPa}{f_{cm}}\right)^{0.2}$$

eq. B3b in Appendix B

Factor to allow for the effect of concrete strength $\beta_{fcm} := 2.73$

Factor to allow for the effect of concrete age at loading, (assuming loading after 28 days)

$$\beta_{t0} := \frac{1}{0.1 + 28^{0.2}} = 0.488$$

eq. B.5 in Appendix B

Creep coefficient $\varphi_{ef}(A_c) := \varphi_{RH}(A_c) \cdot \beta_{fcm} \cdot \beta_{t0}$

A1.3.3 First order moment, with regard to unintended imperfections

A1.3.3.1 Unintended imperfections

Imperfections is represented as an inclination according to section 5.2 in EC2.

Basic value $\theta_0 := \frac{1}{200} = 5 \times 10^{-3}$

$$\alpha_{hh} := \frac{2}{\sqrt{\frac{h_{column}}{m}}} = 1.054$$

Reduction factor for height

$$\alpha_h := \begin{cases} \frac{2}{3} & \text{if } \alpha_{hh} < \frac{2}{3} \\ \alpha_{hh} & \text{if } \frac{2}{3} \le \alpha_{hh} \le 1 \\ 1 & \text{otherwise} \end{cases}$$

$\alpha_h = 1$

$m_c := 1$ Load effect is calcualted for one column

Reduction factor for number of members $\alpha_m := \sqrt{0.5 \cdot \left(1 + \frac{1}{m_c}\right)} = 1$

Unintended inclination $\boxed{\theta_i := \theta_0 \cdot \alpha_h \cdot \alpha_m = 5 \times 10^{-3}}$ <u>eq. 5.1 in section 5.2</u>

Additional first order eccentricity $e_i := \theta_i \cdot \frac{h_{column}}{2} = 9 \times 10^{-3}\ m$ <u>eq. 5.2 in section 5.2</u>

No horisontal forces, hence $e_0 := 0$

First order moment $\boxed{M_{0.Ed_{ii}} := P_{ii}(e_0 + e_i)}$

A1.3.4 Second order moment

If the column is slender the second order moment must be considered.

A1.3.4.1 Slenderness

Second moment of inertia $$I_c(A_c) := \frac{b(A_c)\cdot h(A_c)^3}{12}$$

$$i_c(A_c) := \sqrt{\frac{I_c(A_c)}{A_c}}$$

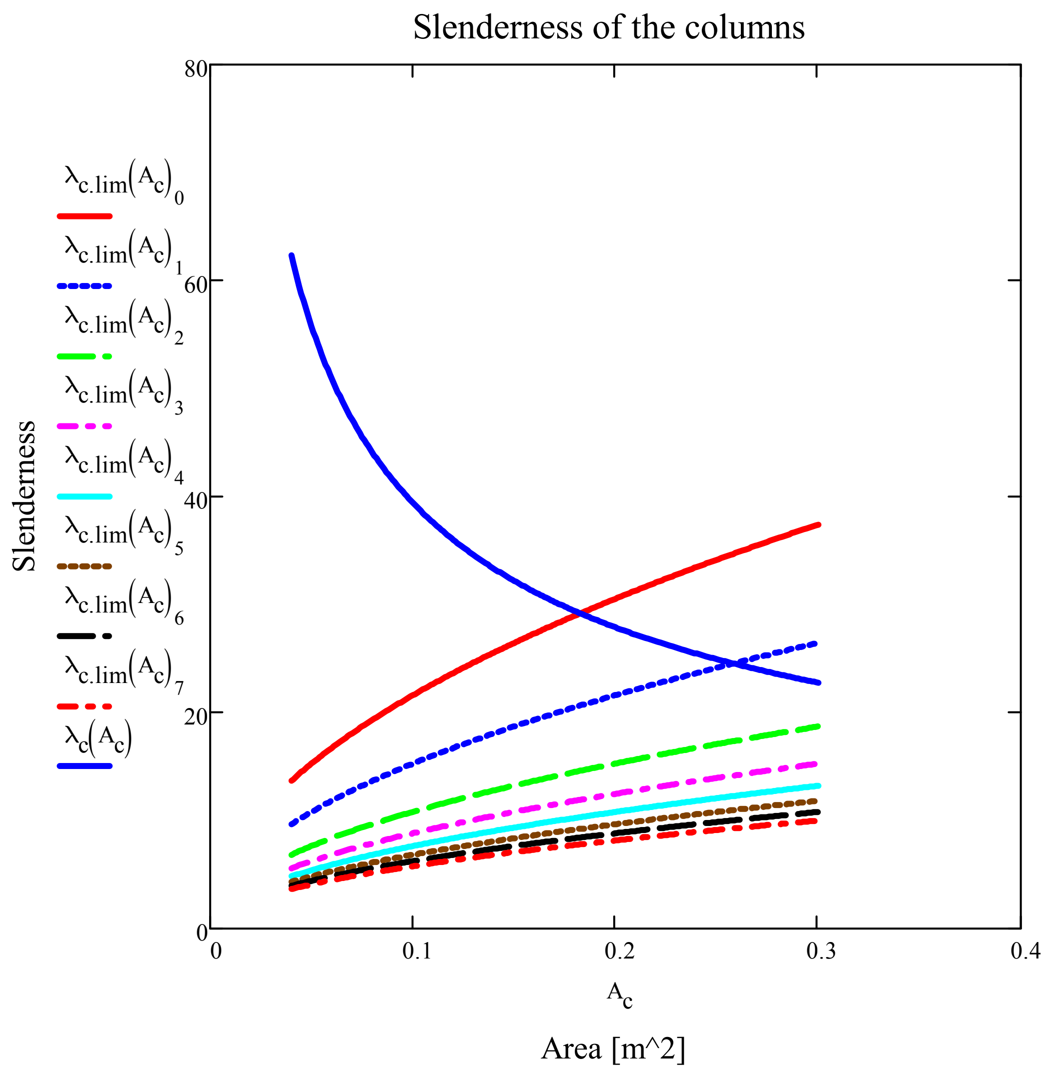
Slenderness $$\lambda_c(A_c) := \frac{h_{column}}{i_c(A_c)}$$

Relative normal force $$n_c(A_c) := \frac{P}{f_{cd}\cdot A_c}$$ eq. 5.13N in section 5.8

Slenderness limit $$\lambda_{c.lim}(A_c) := \frac{10.8}{\sqrt{n_c(A_c)}}$$ eq. 5.13N in section 5.8

(Values given in EC2 has been used when calculatin 10.8, it is on the safe side to use 10.8. If more accurate a higher value will be obtained.)

Condition when column is regarded as slender $\boxed{\lambda_c > \lambda_{c.lim}}$



The column is slender for almost all the loads and areas, therefore taking into account the second order effect for all the cases. The declining line is the limit for when a column is slender or not. If the lines that are increasing are below the decreasing line, the correpsonding column is slender.

A1.3.4.2 Second order moment

Design moment, including second order effects

$$M_{Ed} := \left(1 + \frac{\beta_m}{\frac{N_B}{N_{Ed}} - 1}\right) \cdot M_{0.Ed}$$

eq. 8.28 in section 5.8

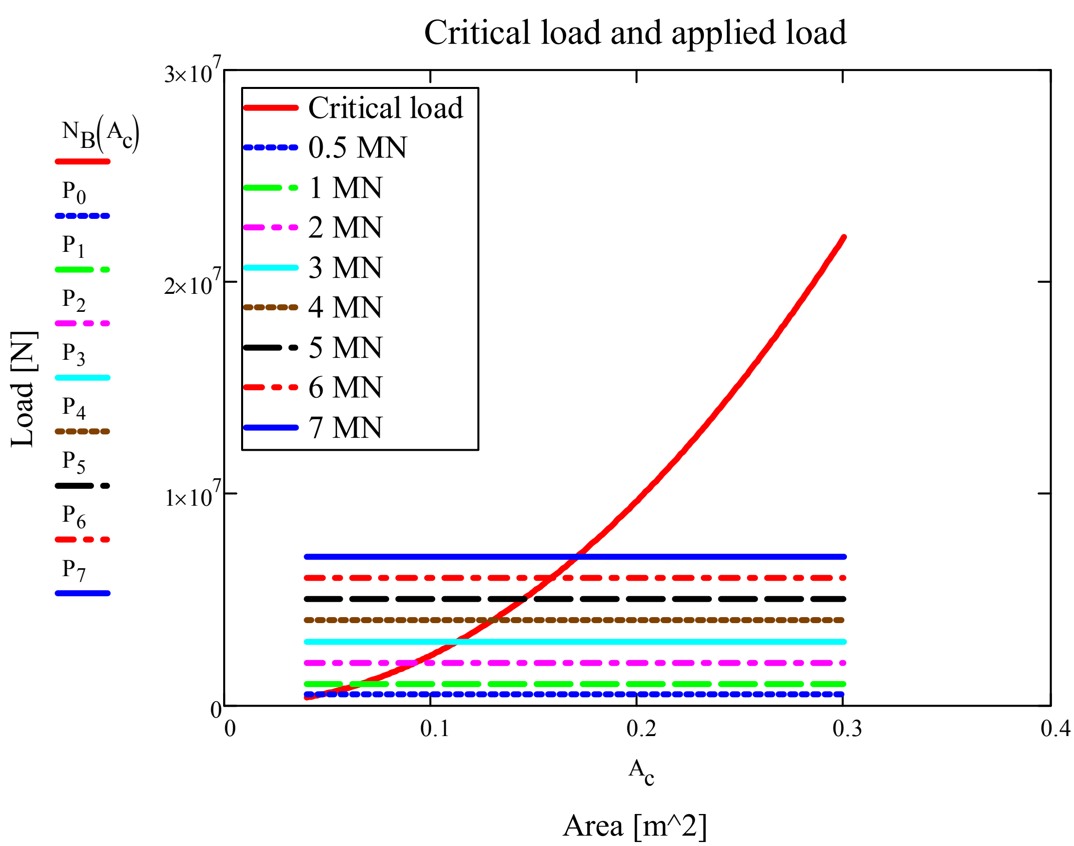
Approximate value of nominal stiffness

$$EI(A_c) := \frac{0.3}{1 + 0.5 \cdot \varphi_{ef}(A_c)} \cdot E_{cd} \cdot I_c(A_c)$$

(Stiffness from the reinforcement is neglected, safe side)

Critical load

$$N_B(A_c) := \frac{\pi^2 EI(A_c)}{h_{column}^{\;2}}$$



Factor depending on 1st and 2nd order distribtuion

$$\beta_m := 1$$

Design moment, first and second order moment. 0.5 MN axial load.

$$M_{Ed.0}(A_c) := \left(1 + \frac{\beta_m}{\frac{N_B(A_c)}{P_0} - 1}\right) \cdot M_{0.Ed_0}$$

1 MN axial load

$$M_{Ed.1}(A_c) := \left(1 + \frac{\beta_m}{\frac{N_B(A_c)}{P_1} - 1}\right) \cdot M_{0.Ed_1}$$

2 MN axial load

$$M_{Ed.2}(A_c) := \left(1 + \frac{\beta_m}{\frac{N_B(A_c)}{P_2} - 1}\right) \cdot M_{0.Ed_2}$$

3 MN axial load

$$M_{Ed.3}(A_c) := \left(1 + \frac{\beta_m}{\frac{N_B(A_c)}{P_3} - 1}\right) \cdot M_{0.Ed_3}$$

4 MN axial load

$$M_{Ed.4}(A_c) := \left(1 + \frac{\beta_m}{\frac{N_B(A_c)}{P_4} - 1}\right) \cdot M_{0.Ed_4}$$

5 MN axial load

$$M_{Ed.5}(A_c) := \left(1 + \frac{\beta_m}{\frac{N_B(A_c)}{P_5} - 1}\right) \cdot M_{0.Ed_5}$$

6 MN axial load

$$M_{Ed.6}(A_c) := \left(1 + \frac{\beta_m}{\frac{N_B(A_c)}{P_6} - 1}\right) \cdot M_{0.Ed_6}$$

7 MN axial load

$$M_{Ed.7}(A_c) := \left(1 + \frac{\beta_m}{\frac{N_B(A_c)}{P_7} - 1}\right) \cdot M_{0.Ed_7}$$

A1.3.5 Sectional analysis in ULS

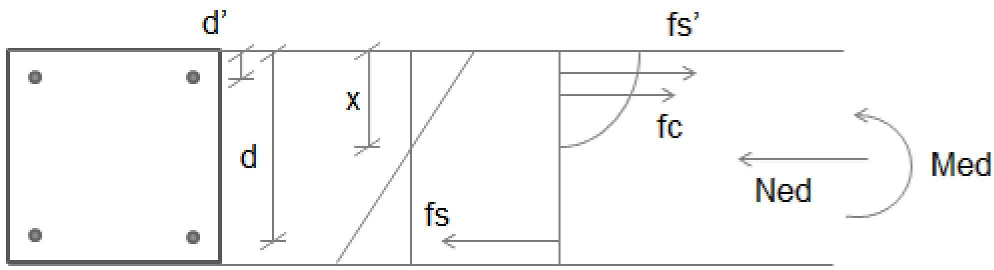
Check the moment capacity in state III assuming a rectangular compression zone

$\alpha := 0.810$

$\beta := 0.416$

$\varepsilon_{cu} := 3.5 \cdot 10^{-3}$

$\varepsilon_{yd} = 2.174 \times 10^{-3}$



A1.3.5.1 P = 0.5 MN

Assume that all the reinforcement is yielding and using 4 bars in total.

$$\boxed{\alpha \cdot f_{cd} \cdot b \cdot x = \blacksquare \cdot N_{Ed}}$$

The first area that has higher capacity than the critical axial load, 0.05m2

$b_0 := 0.224\text{m}$

$$x_0 := \frac{P_0}{\alpha \cdot f_{cd} \cdot b_0} = 0.138\,\text{m}$$

$d_0 := b_0 - 0.05\text{m} = 0.174\,\text{m}$

$$\varepsilon'_{s.0} := \frac{x_0 - d'}{x_0} \cdot \varepsilon_{cu} = 2.23 \times 10^{-3}$$ OK

$$\varepsilon_{s.0} := \frac{d_0 - x_0}{x_0} \cdot \varepsilon_{cu} = 9.199 \times 10^{-4}$$ Not OK

Assume A´s is yielding and As not

$x_0 := 0.1\text{m}$

$$x_0 := \text{root}\left(\alpha \cdot f_{cd} \cdot b_0 \cdot x_0 - E_s \cdot \frac{d_0 - x_0}{x_0} \cdot \varepsilon_{cu} \cdot 2 \cdot A_{si} + f_{yd} \cdot 2 \cdot A_{si} - P_0, x_0\right)$$

$x_0 = 0.122\,\mathrm{m}$

$$\varepsilon'_{s.0} := \frac{x_0 - d'}{x_0}\cdot\varepsilon_{cu} = 2.07\times 10^{-3}$$ Not OK!

$$\varepsilon_{s.0} := \frac{d_0 - x_0}{x_0}\cdot\varepsilon_{cu} = 1.477\times 10^{-3}$$ OK! (not yielding)

Assuming both compression and tension reinforcement not yielding

$x_0 := 0.1\mathrm{m}$

$$x_0 := \mathrm{root}\left(\alpha\cdot f_{cd}\cdot b_0\cdot x_0 + E_s\cdot\frac{x_0 - d'}{x_0}\cdot\varepsilon_{cu}\cdot 2\cdot A_{si} - E_s\cdot\frac{d_0 - x_0}{x_0}\cdot\varepsilon_{cu}\cdot 2\cdot A_{si} - P_0, x_0\right)$$

$x_0 = 0.123\,\mathrm{m}$

$$\varepsilon'_{s.0} := \frac{x_0 - d'}{x_0}\cdot\varepsilon_{cu} = 2.082\times 10^{-3}$$

No one is yielding, OK!

$$\varepsilon_{s.0} := \frac{d_0 - x_0}{x_0}\cdot\varepsilon_{cu} = 1.434\times 10^{-3}$$

Capacity of the column

$$M_{Rd.0} := \alpha\cdot f_{cd}\cdot b_0\cdot x_0\cdot\left(d_0 - \beta\cdot x_0\right) + E_s\cdot\varepsilon'_{s.0}\cdot 2\cdot A_{si}\cdot\left(d_0 - d'\right) - P_0\cdot\left(d_0 - \frac{b_0}{2}\right) = 44.7\cdot\mathrm{kNm}$$

A1.3.5.2 P = 1 MN

Assume that all the reinforcement is yielding and using 4 bars in total.

$$\alpha\cdot f_{cd}\cdot b\cdot x = \blacksquare\cdot N_{Ed}$$

The first area that has higher capacity than the critical axial load, 0.071m2

$b_1 := 0.266\mathrm{m}$

$$x_1 := \frac{P_1}{\alpha\cdot f_{cd}\cdot b_1} = 0.232\,\mathrm{m}$$

$d_1 := b_1 - 0.05\mathrm{m} = 0.216\,\mathrm{m}$

$$\varepsilon'_{s.1} := \frac{x_1 - d'}{x_1}\cdot\varepsilon_{cu} = 2.746\times 10^{-3}$$ OK

$$\varepsilon_{s.1} := \frac{x_1 - d_1}{x_1}\cdot\varepsilon_{cu} = 2.422\times 10^{-4}$$ Not OK

Assume A´s is yielding and As not

$$x_1 := 0.1\text{m}$$

$$x_1 := \text{root}\left(\alpha \cdot f_{cd} \cdot b_1 \cdot x_1 - E_s \cdot \frac{d_1 - x_1}{x_1} \cdot \varepsilon_{cu} \cdot 2 \cdot A_{si} + f_{yd} \cdot 2 \cdot A_{si} - P_1, x_1\right)$$

$$x_1 = 0.198\,\text{m}$$

$$\varepsilon'_{s.1} := \frac{x_1 - d'}{x_1} \cdot \varepsilon_{cu} = 2.614 \times 10^{-3}$$ OK!

$$\varepsilon_{s.1} := \frac{d_1 - x_1}{x_1} \cdot \varepsilon_{cu} = 3.263 \times 10^{-4}$$ OK!

Capacity of the column

$$M_{Rd.1} := \alpha \cdot f_{cd} \cdot b_1 \cdot x_1 \cdot \left(d_1 - \beta \cdot x_1\right) + E_s \cdot \varepsilon'_{s.1} \cdot 2 \cdot A_{si} \cdot \left(d_1 - d'\right) - P_1 \cdot \left(d_1 - \frac{b_1}{2}\right) = 65.826 \cdot \text{kNm}$$

A1.3.5.3 P = 2 MN

Assume that all the reinforcement is yielding and using 6 bars, 3 bars on the compressive side and 3 bars on the tensile side.

$$\alpha \cdot f_{cd} \cdot b \cdot x = \blacksquare \cdot N_{Ed}$$

The first area that has higher capacity than the critical axial load, 0.105m2

$$b_2 := 0.324\text{m}$$

$$x_2 := \frac{P_2}{\alpha \cdot f_{cd} \cdot b_2} = 0.381\,\text{m}$$

$$d_2 := b_2 - 0.05\text{m} = 0.274\,\text{m}$$

$$\varepsilon'_{s.2} := \frac{x_2 - d'}{x_2} \cdot \varepsilon_{cu} = 3.041 \times 10^{-3}$$ OK

$$\varepsilon_{s.2} := \frac{x_2 - d_2}{x_2} \cdot \varepsilon_{cu} = 9.832 \times 10^{-4}$$ Not OK

Assume A´s is yielding and As not

$$x_2 := 0.2\text{m}$$

$$x_2 := \mathrm{root}\left(\alpha \cdot f_{cd} \cdot b_2 \cdot x_2 - E_s \cdot \frac{d_2 - x_2}{x_2} \cdot \varepsilon_{cu} \cdot 3 \cdot A_{si} + f_{yd} \cdot 3 \cdot A_{si} - P_2, x_2\right)$$

$$x_2 = 0.32\,\mathrm{m}$$

$$\varepsilon'_{s.2} := \frac{x_2 - d'}{x_2} \cdot \varepsilon_{cu} = 2.952 \times 10^{-3}$$ OK!

$$\varepsilon_{s.2} := \frac{x_2 - d_2}{x_2} \cdot \varepsilon_{cu} = 4.994 \times 10^{-4}$$ OK!

Capacity of the column

$$M_{Rd.2} := \alpha \cdot f_{cd} \cdot b_2 \cdot x_2 \cdot (d_2 - \beta \cdot x_2) + E_s \cdot \varepsilon'_{s.2} \cdot 3 \cdot A_{si} \cdot (d_2 - d') - P_2 \cdot \left(d_2 - \frac{b_2}{2}\right) = 92.39 \cdot \mathrm{kNm}$$

A1.3.5.4 P = 3 MN

Assume that all the reinforcement is yielding, using 8 bars.

$$\alpha \cdot f_{cd} \cdot b \cdot x = \blacksquare \cdot N_{Ed}$$

The first area that has higher capacity than the critical axial load, 0.14m2

$$b_3 := 0.374\mathrm{m}$$

$$x_3 := \frac{P_3}{\alpha \cdot f_{cd} \cdot b_3} = 0.495\,\mathrm{m}$$

$$d_3 := b_3 - 0.05\mathrm{m} = 0.324\,\mathrm{m}$$

$$\varepsilon'_{s.3} := \frac{x_3 - d'}{x_3} \cdot \varepsilon_{cu} = 3.147 \times 10^{-3}$$ OK

$$\varepsilon_{s.3} := \frac{x_3 - d_3}{x_3} \cdot \varepsilon_{cu} = 1.21 \times 10^{-3}$$ Not OK

Assume A´s is yielding and As not

$$x_3 := 0.2\mathrm{m}$$

$$x_3 := \mathrm{root}\left(\alpha \cdot f_{cd} \cdot b_3 \cdot x_3 - E_s \cdot \frac{d_3 - x_3}{x_3} \cdot \varepsilon_{cu} \cdot 4 \cdot A_{si} + f_{yd} \cdot 4 \cdot A_{si} - P_3, x_3\right)$$

$$x_3 = 0.417\,\mathrm{m}$$

$$\varepsilon'_{s.3} := \frac{x_3 - d'}{x_3} \cdot \varepsilon_{cu} = 3.08 \times 10^{-3}$$ OK!

$$\varepsilon_{s.3} := \frac{x_3 - d_3}{x_3} \cdot \varepsilon_{cu} = 7.79 \times 10^{-4}$$ OK!

Capacity of the column

$$M_{Rd.3} := \alpha \cdot f_{cd} \cdot b_3 \cdot x_3 \cdot \left(d_3 - \beta \cdot x_3\right) + E_s \cdot \varepsilon'_{s.3} \cdot 4 \cdot A_{si} \cdot \left(d_3 - d'\right) - P_3 \cdot \left(d_3 - \frac{b_3}{2}\right) = 105.094 \cdot kNm$$

A1.3.5.5 P = 4 MN

Assume that all the reinforcement is yielding, using 10 bars.

$$\alpha \cdot f_{cd} \cdot b \cdot x = \blacksquare \cdot N_{Ed}$$

The first area that has higher capacity than the critical axial load, 0.17m2

$$b_4 := 0.412m$$

$$x_4 := \frac{P_4}{\alpha \cdot f_{cd} \cdot b_4} = 0.599\,m$$

$$d_4 := b_4 - 0.05m = 0.362\,m$$

$$\varepsilon'_{s.4} := \frac{x_4 - d'}{x_4} \cdot \varepsilon_{cu} = 3.208 \times 10^{-3}$$ OK

$$\varepsilon_{s.4} := \frac{x_4 - d_4}{x_4} \cdot \varepsilon_{cu} = 1.386 \times 10^{-3}$$ Not OK

Assume A´s is yielding and As not

$$x_4 := 0.2m$$

$$x_4 := \mathrm{root}\left(\alpha \cdot f_{cd} \cdot b_4 \cdot x_4 - E_s \cdot \frac{d_4 - x_4}{x_4} \cdot \varepsilon_{cu} \cdot 5 \cdot A_{si} + f_{yd} \cdot 5 \cdot A_{si} - P_4, x_4\right)$$

$$x_4 = 0.504\,m$$

$$\varepsilon'_{s.4} := \frac{x_4 - d'}{x_4} \cdot \varepsilon_{cu} = 3.153 \times 10^{-3}$$ OK!

$$\varepsilon_{s.4} := \frac{x_4 - d_4}{x_4} \cdot \varepsilon_{cu} = 9.866 \times 10^{-4}$$ OK!

Capacity of the column

$$M_{Rd.4} := \alpha \cdot f_{cd} \cdot b_4 \cdot x_4 \cdot (d_4 - \beta \cdot x_4) + E_s \cdot \varepsilon'_{s.4} \cdot 5 \cdot A_{si} \cdot (d_4 - d') - P_4 \cdot \left(d_4 - \frac{b_4}{2}\right) = 86.188 \cdot kNm$$

A1.3.5.6 P = 5 MN

Assume that all the reinforcement is yielding using 12 bars.

$$\alpha \cdot f_{cd} \cdot b \cdot x = \blacksquare \cdot N_{Ed}$$

The first area that has higher capacity than the critical axial load, 0.21m2

$$b_5 := 0.458m$$

$$x_5 := \frac{P_5}{\alpha \cdot f_{cd} \cdot b_5} = 0.674\,m$$

$$d_5 := b_5 - 0.05m = 0.408\,m$$

$$\varepsilon'_{s.5} := \frac{x_5 - d'}{x_5} \cdot \varepsilon_{cu} = 3.24 \times 10^{-3}$$ OK

$$\varepsilon_{s.5} := \frac{x_5 - d_5}{x_5} \cdot \varepsilon_{cu} = 1.381 \times 10^{-3}$$ Not OK

Assume A´s is yielding and As not

$$x_5 := 0.2m$$

$$x_5 := \text{root}\left(\alpha \cdot f_{cd} \cdot b_5 \cdot x_5 - E_s \cdot \frac{d_5 - x_5}{x_5} \cdot \varepsilon_{cu} \cdot 6 \cdot A_{si} + f_{yd} \cdot 6 \cdot A_{si} - P_5, x_5\right)$$

$$x_5 = 0.571\,m$$

$$\varepsilon'_{s.5} := \frac{x_5 - d'}{x_5} \cdot \varepsilon_{cu} = 3.193 \times 10^{-3}$$ OK!

$$\varepsilon_{s.5} := \frac{x_5 - d_5}{x_5} \cdot \varepsilon_{cu} = 9.98 \times 10^{-4}$$ OK!

Capacity of the column

$$M_{Rd.5} := \alpha \cdot f_{cd} \cdot b_5 \cdot x_5 \cdot (d_5 - \beta \cdot x_5) + E_s \cdot \varepsilon'_{s.5} \cdot 6 \cdot A_{si} \cdot (d_5 - d') - P_5 \cdot \left(d_5 - \frac{b_5}{2}\right) = 103.146 \cdot kNm$$

A1.3.5.7 P = 6 MN

Assume that all the reinforcement is yielding using 14 bars.

$$\alpha \cdot f_{cd} \cdot b \cdot x = \blacksquare \cdot N_{Ed}$$

The first area that has higher capacity than the critical axial load, 0.245m2

$$b_6 := 0.495\text{m}$$

$$x_6 := \frac{P_6}{\alpha \cdot f_{cd} \cdot b_6} = 0.748\,\text{m}$$

$$d_6 := b_6 - 0.05\text{m} = 0.445\,\text{m}$$

$$\varepsilon'_{s.6} := \frac{x_6 - d'}{x_6} \cdot \varepsilon_{cu} = 3.266 \times 10^{-3}$$ OK

$$\varepsilon_{s.6} := \frac{x_6 - d_6}{x_6} \cdot \varepsilon_{cu} = 1.418 \times 10^{-3}$$ Not OK

Assume A´s is yielding and As not

$$x_6 := 0.2\text{m}$$

$$x_6 := \text{root}\left(\alpha \cdot f_{cd} \cdot b_6 \cdot x_6 - E_s \cdot \frac{d_6 - x_6}{x_6} \cdot \varepsilon_{cu} \cdot 7 \cdot A_{si} + f_{yd} \cdot 7 \cdot A_{si} - P_6, x_6\right)$$

$$x_6 = 0.635\,\text{m}$$

$$\varepsilon'_{s.6} := \frac{x_6 - d'}{x_6} \cdot \varepsilon_{cu} = 3.224 \times 10^{-3}$$ OK!

$$\varepsilon_{s.6} := \frac{x_6 - d_6}{x_6} \cdot \varepsilon_{cu} = 1.048 \times 10^{-3}$$ OK!

<u>Capacity of the column</u>

$$M_{Rd.6} := \alpha \cdot f_{cd} \cdot b_6 \cdot x_6 \cdot \left(d_6 - \beta \cdot x_6\right) + E_s \cdot \varepsilon'_{s.6} \cdot 7 \cdot A_{si} \cdot \left(d_6 - d'\right) - P_6 \cdot \left(d_6 - \frac{b_6}{2}\right) = 94.279 \cdot \text{kNm}$$

A1.3.5.8 P = 7 MN

Assume that all the reinforcement is yielding using 16 bars.

$$\alpha \cdot f_{cd} \cdot b \cdot x = \blacksquare \cdot N_{Ed}$$

The first area that has higher capacity than the critical axial load, 0.285m2

$$b_7 := 0.534\text{m}$$

$$x_7 := \frac{P_7}{\alpha \cdot f_{cd} \cdot b_7} = 0.809\,\text{m}$$

$$d_7 := b_7 - 0.05m = 0.484\,m$$

$$\varepsilon'_{s.7} := \frac{x_7 - d'}{x_7}\cdot\varepsilon_{cu} = 3.284\times 10^{-3}$$ OK

$$\varepsilon_{s.7} := \frac{x_7 - d_7}{x_7}\cdot\varepsilon_{cu} = 1.407\times 10^{-3}$$ Not OK

Assume A´s is yielding and As not

$$x_7 := 0.2m$$

$$x_7 := \mathrm{root}\left(\alpha\cdot f_{cd}\cdot b_7\cdot x_7 - E_s\cdot\frac{d_7 - x_7}{x_7}\cdot\varepsilon_{cu}\cdot 8\cdot A_{si} + f_{yd}\cdot 8\cdot A_{si} - P_7, x_7\right)$$

$$x_7 = 0.69\,m$$

$$\varepsilon'_{s.7} := \frac{x_7 - d'}{x_7}\cdot\varepsilon_{cu} = 3.246\times 10^{-3}$$ OK!

$$\varepsilon_{s.7} := \frac{x_7 - d_7}{x_7}\cdot\varepsilon_{cu} = 1.043\times 10^{-3}$$ OK!

Capacity of the column

$$M_{Rd.7} := \alpha\cdot f_{cd}\cdot b_7\cdot x_7\cdot\left(d_7 - \beta\cdot x_7\right) + E_s\cdot\varepsilon'_{s.7}\cdot 8\cdot A_{si}\cdot\left(d_7 - d'\right) - P_7\cdot\left(d_7 - \frac{b_7}{2}\right) = 110.253\cdot kNm$$

Appendix A2a: Beams

$kNm := kN \cdot m$

Calculations for obtaining dimensions for beams, dimensions are presented in Section 5.3. Dimensions for steel and concrete beams are obtained from tables and diagram from Tibnor and Svensk Betong. Therefore these calculations regards only timber beams.

Calcualtions have been performed according to **SS-EN 1995-1-1:2004**. All references made refers back to this eurocode.

A2.1 Loads

Imposed load, for a office building including the loads from partition walls $q_{office} := 3 \frac{kN}{m^2}$

Density for glulam, L40c (mean value) $\rho_{glulam} := 4300 \frac{N}{m^3}$

Density for LVL, Kerto-S (mean value) $\rho_{lvl} := 5100 \frac{N}{m^3}$

Self-weight of floor structure, including installations (assuming a heavy timber floor) $g_{floor} := 2.5 \frac{kN}{m^2}$

Assumed tributary lenght (this length was changed depending on which tributary length that was of interest) $l := 6m$

A2.2 Glulam beams

$i := 0..4$

A2.2.1 Material data

Bending parallell to grain, glulam (L40c) $f_{m.g.k} := 30.8MPa$

Shear strength, glulam (L40c) $f_{v.g.k} := 3.5MPa$

Elastic modulus (capacity analysis) $E_{0.g.05} := 10500MPa$

Elastic modulus (deformation calculations) $E_{0.g.mean} := 13000MPa$

Partial factor $\gamma_{M.glulam} := 1.25$

A2.2.2 Dimensions

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

Height of beam, (changed to obtain correct utilisation ratio)

$$h_{glulam} := \begin{pmatrix} 15 \cdot h_{lamell} \\ 17 \cdot h_{lamell} \\ 19 \cdot h_{lamell} \\ 23 \cdot h_{lamell} \\ 26 \cdot h_{lamell} \end{pmatrix} \qquad h_{glulam} = \begin{pmatrix} 0.675 \\ 0.765 \\ 0.855 \\ 1.035 \\ 1.17 \end{pmatrix} m$$

Width of beam

$$w_{glulam} := \begin{pmatrix} 165 \\ 190 \\ 215 \\ 2 \cdot 115 \\ 2 \cdot 140 \end{pmatrix} mm \qquad w_{glulam} = \begin{pmatrix} 0.165 \\ 0.19 \\ 0.215 \\ 0.23 \\ 0.28 \end{pmatrix} m$$

Area of beam section

$$A_{glulam_i} := h_{glulam_i} \cdot w_{glulam_i}$$

A2.2.3 Load combinations and load effects in ULS

Assuming simply supported beams to obtain worst case.

$$Q_{glulam.a_i} := 1.35 \cdot 0.89 \cdot \left(g_{floor} \cdot 1 + \rho_{glulam} \cdot A_{glulam_i}\right) + 1.5 q_{office} \cdot 1$$ <u>eq. 6.10b in section 6.4 in SS-EN1990</u>

$$Q_{glulam.b_i} := 1.35 \cdot \left(g_{floor} \cdot 1 + \rho_{glulam} \cdot A_{glulam_i}\right) + 1.5 \cdot 0.7 q_{office} \cdot 1$$ <u>eq. 6.10a in section 6.4 in SS-EN1990</u>

$$Q_{glulam_i} := \max\left(Q_{glulam.a_i}, Q_{glulam.b_i}\right) \qquad Q_{glulam} = \begin{pmatrix} 45.598 \\ 45.773 \\ 45.972 \\ 46.252 \\ 46.715 \end{pmatrix} \cdot \frac{kN}{m}$$

5 different spans that are used as design conditions

$$l_{span} := \begin{pmatrix} 4 \\ 6 \\ 8 \\ 10 \\ 12 \end{pmatrix} m$$

A2.2.3.1 Bending moment for a simply supported beam

Largest bending moment is to be found in the middle of the span.

Moment in a simply supported beam

$$\boxed{M_{Ed} := \frac{Q \cdot l^2}{8}}$$

4 metre span

$$M_{Ed.4} := \frac{Q_{glulam_0} \cdot \left(l_{span_0}\right)^2}{8} = 91.196 \cdot kNm$$

6 metre span

$$M_{Ed.6} := \frac{Q_{glulam_1} \cdot \left(l_{span_1}\right)^2}{8} = 205.98 \cdot kNm$$

8 metre span

$$M_{Ed.8} := \frac{Q_{glulam_2} \cdot \left(l_{span_2}\right)^2}{8} = 367.778 \cdot kNm$$

10 metre span

$$M_{Ed.10} := \frac{Q_{glulam_3} \cdot \left(l_{span_3}\right)^2}{8} = 578.155 \cdot kNm$$

12 metre span

$$M_{Ed.12} := \frac{Q_{glulam_4} \cdot \left(l_{span_4}\right)^2}{8} = 840.871 \cdot kNm$$

A2.2.3.2 Shear force for a simply suppoerted beam

Largest shear forces is to be found in the ends of the beam.

Shear force in a simply supported beam $\boxed{V_{Ed} := \frac{Q \cdot l}{2}}$

4 metre span

$$V_{Ed.4} := \frac{Q_{glulam_0} \cdot l_{span_0}}{2} = 91.196 \cdot kN$$

6 metre span

$$V_{Ed.6} := \frac{Q_{glulam_1} \cdot l_{span_1}}{2} = 137.32 \cdot kN$$

8 metre span

$$V_{Ed.8} := \frac{Q_{glulam_2} \cdot l_{span_2}}{2} = 183.889 \cdot kN$$

10 metre span

$$V_{Ed.10} := \frac{Q_{glulam_3} \cdot l_{span_3}}{2} = 231.262 \cdot kN$$

12 metre span

$$V_{Ed.12} := \frac{Q_{glulam_4} \cdot l_{span_4}}{2} = 280.29 \cdot kN$$

A2.2.4 Moment capacity

Assuming medium term load and service class 2

Strength modification factor $k_{mod.glulam} := 0.8$ Section 3.2

Deformation modification factor $k_{def} := 0.6$ Section 3.1

Effect of member size

$$k_{h.glulam_i} := \begin{cases} \min\left[\left(\frac{600mm}{h_{glulam_i}}\right)^{0.1}, 1.1\right] & \text{if } h_{glulam_i} \le 600mm \\ 1 & \text{otherwise} \end{cases}$$

eq. 3.2 in section 3.3

Section modulus

$$W_{glulam_i} := \frac{w_{glulam_i} \cdot \left(h_{glulam_i}\right)^2}{6}$$

Second moment of inertia

$$I_{glulam_i} := \frac{w_{glulam_i} \cdot \left(h_{glulam_i}\right)^3}{12}$$

Design value for bending parallel to grain

$$f_{m.g.d_i} := k_{mod.glulam} \cdot k_{h.glulam_i} \cdot \frac{f_{m.g.k}}{\gamma_{M.glulam}}$$

eq. 2.17 in section 2.4

Design value for shear

$$f_{v.g.d} := k_{mod.glulam} \cdot \frac{f_{v.g.k}}{\gamma_{M.glulam}} = 2.24 \cdot MPa$$

Moment capacity

$$M_{Rd.glulam_i} := f_{m.g.d_i} \cdot W_{glulam_i}$$

Moment capacity

$$M_{Rd.glulam} = \begin{pmatrix} 246.985 \\ 365.305 \\ 516.357 \\ 809.446 \\ 1.259 \times 10^3 \end{pmatrix} \cdot kN \cdot m$$

Applied moment

$M_{Ed.4} = 91.196 \cdot kNm$

$M_{Ed.6} = 205.98 \cdot kNm$

$M_{Ed.8} = 367.778 \cdot kNm$

$M_{Ed.10} = 578.155 \cdot kNm$

$M_{Ed.12} = 840.871 \cdot kNm$

A2.2.5 Shear capacity

A beam should fulfill the following condition regarding shear:

$\boxed{\tau_d < f_{vd}}$ eq. 6.13 in Section 6.1

Applied shear $\boxed{\tau_d := \frac{S \cdot V}{I \cdot b_{eff}}}$

First moment of inertia $\boxed{S := Ay}$

$$S_i := w_{glulam_i} \cdot \frac{1}{2} \cdot h_{glulam_i} \cdot \frac{1}{4} \cdot h_{glulam_i}$$

Effective width $b_{eff} := k_{cr} \cdot b$ eq. 6.13a in section 6.1 in SS-EN 1995-1-1:2008

$$k_{cr} := 0.67$$

$$b_{eff_i} := k_{cr} \cdot w_{glulam_i}$$

4 metre span $\tau_{d.4} := \frac{S_0 \cdot V_{Ed.4}}{I_{glulam_0} \cdot b_{eff_0}} = 1.833 \cdot MPa$ $\tau_d < f_{vd}$ OK

6 metre span $\tau_{d.6} := \frac{S_1 \cdot V_{Ed.6}}{I_{glulam_1} \cdot b_{eff_1}} = 2.115 \cdot MPa$ $\tau_d < f_{vd}$ OK

8 metre span $\tau_{d.8} := \frac{S_2 \cdot V_{Ed.8}}{I_{glulam_2} \cdot b_{eff_2}} = 2.24 \cdot MPa$ $\tau_d < f_{vd}$ OK

10 metre span $\tau_{d.10} := \frac{S_3 \cdot V_{Ed.10}}{I_{glulam_3} \cdot b_{eff_3}} = 2.175 \cdot MPa$ $\tau_d < f_{vd}$ OK

12 metre span $\tau_{d.12} := \frac{S_4 \cdot V_{Ed.12}}{I_{glulam_4} \cdot b_{eff_4}} = 1.915 \cdot MPa$ $\tau_d < f_{vd}$ OK

A2.2.6 Deflection

Quasi-permanent load combination has been used together with a limit of span length/400.

$w_{fin} := w_{fin.G} + w_{fin.Q}$ in section 2.2.3

$$w_{fin.G} := w_{inst.G}(1 + k_{def})$$

$$w_{fin.Q} := w_{inst.Q}(1 + \psi_2 \cdot k_{def})$$

$\psi_2 := 0.3$

Instant deflections for a simply supported beam calculated as

$$\boxed{w := \frac{(g + q)\cdot 5\cdot l^4}{384EI}}$$

4 metre span

$$w_{fin.4.G} := \frac{\left(g_{floor}\cdot 1 + \rho_{glulam}\cdot A_{glulam_0}\right)\cdot 5\cdot \left(l_{span_0}\right)^4}{384\cdot E_{0.g.mean}\cdot I_{glulam_0}}\cdot\left(1 + k_{def}\right)$$

$$w_{fin.4.Q} := \frac{q_{office}\cdot 1\cdot 5\cdot \left(l_{span_0}\right)^4}{384\cdot E_{0.g.mean}\cdot I_{glulam_0}}\cdot\left(1 + \psi_2\, k_{def}\right)$$

$$w_{fin.4} := w_{fin.4.G} + w_{fin.4.Q}$$

$w_{fin.4} = 2.79\cdot mm$ $\qquad$ $w_{limit.4} := \frac{4m}{400} = 10\cdot mm$

6 metre span

$$w_{fin.6.G} := \frac{\left(g_{floor}\cdot 1 + \rho_{glulam}\cdot A_{glulam_1}\right)\cdot 5\cdot \left(l_{span_1}\right)^4}{384\cdot E_{0.g.mean}\cdot I_{glulam_1}}\cdot\left(1 + k_{def}\right)$$

$$w_{fin.6.Q} := \frac{q_{office}\cdot 1\cdot 5\cdot \left(l_{span_1}\right)^4}{384\cdot E_{0.g.mean}\cdot I_{glulam_1}}\cdot\left(1 + \psi_2\, k_{def}\right)$$

$$w_{fin.6} := w_{fin.6.G} + w_{fin.6.Q}$$

$w_{fin.6} = 8.468\cdot mm$ $\qquad$ $w_{limit.6} := \frac{6m}{400} = 15\cdot mm$

8 metre span

$$w_{fin.8.G} := \frac{\left(g_{floor}\cdot 1 + \rho_{glulam}\cdot A_{glulam_2}\right)\cdot 5\cdot \left(l_{span_2}\right)^4}{384\cdot E_{0.g.mean}\cdot I_{glulam_2}}\cdot\left(1 + k_{def}\right)$$

$$w_{fin.8.Q} := \frac{q_{office}\cdot 1\cdot 5\cdot \left(l_{span_2}\right)^4}{384\cdot E_{0.g.mean}\cdot I_{glulam_2}}\cdot\left(1 + \psi_2\, k_{def}\right)$$

$$w_{fin.8} := w_{fin.8.G} + w_{fin.8.Q}$$

$$w_{fin.8} = 17.037 \cdot mm \qquad w_{limit.8} := \frac{8m}{400} = 20 \cdot mm$$

10 metre span

$$w_{fin.10.G} := \frac{\left(g_{floor} \cdot 1 + \rho_{glulam} \cdot A_{glulam_3}\right) \cdot 5 \cdot \left(l_{span_3}\right)^4}{384 \cdot E_{0.g.mean} \cdot I_{glulam_3}} \cdot \left(1 + k_{def}\right)$$

$$w_{fin.10.Q} := \frac{q_{office} \cdot 1 \cdot 5 \cdot \left(l_{span_3}\right)^4}{384 \cdot E_{0.g.mean} \cdot I_{glulam_3}} \cdot \left(1 + \psi_2 \, k_{def}\right)$$

$$w_{fin.10} := w_{fin.10.G} + w_{fin.10.Q}$$

$$w_{fin.10} = 22.095 \cdot mm \qquad w_{limit.10} := \frac{10m}{400} = 25 \cdot mm$$

12 metre span

$$w_{fin.12.G} := \frac{\left(g_{floor} \cdot 1 + \rho_{glulam} \cdot A_{glulam_4}\right) \cdot 5 \cdot \left(l_{span_4}\right)^4}{384 \cdot E_{0.g.mean} \cdot I_{glulam_4}} \cdot \left(1 + k_{def}\right)$$

$$w_{fin.12.Q} := \frac{q_{office} \cdot 1 \cdot 5 \cdot \left(l_{span_4}\right)^4}{384 \cdot E_{0.g.mean} \cdot I_{glulam_4}} \cdot \left(1 + \psi_2 \, k_{def}\right)$$

$$w_{fin.12} := w_{fin.12.G} + w_{fin.12.Q}$$

$$w_{fin.12} = 26.395 \cdot mm \qquad w_{limit.12} := \frac{12m}{400} = 30 \cdot mm$$

A2.2.7 Utilisation ratios

Utilisation ratios for all beams in the different spans are presented below, ratios for bending, shear and deflections.

4 metre span

Moment utilisation	Shear utilisation	Deflection utilisation
$u_{M.glulam.4} := \dfrac{M_{Ed.4}}{M_{Rd.glulam_0}} = 0.369$	$u_{V.glulam.4} := \dfrac{\tau_{d.4}}{f_{v.g.d}} = 0.818$	$u_{defl.4} := \dfrac{w_{fin.4}}{w_{limit.4}} = 0.279$

6 metre span

$$u_{M.glulam.6} := \frac{M_{Ed.6}}{M_{Rd.glulam_1}} = 0.564 \quad u_{V.glulam.6} := \frac{\tau_{d.6}}{f_{v.g.d}} = 0.944 \quad u_{defl.6} := \frac{w_{fin.6}}{w_{limit.6}} = 0.565$$

8 metre span

$$u_{M.glulam.8} := \frac{M_{Ed.8}}{M_{Rd.glulam_2}} = 0.712 \quad u_{V.glulam.8} := \frac{\tau_{d.8}}{f_{v.g.d}} = 1 \quad u_{defl.8} := \frac{w_{fin.8}}{w_{limit.8}} = 0.852$$

10 metre span

$$u_{M.glulam.10} := \frac{M_{Ed.10}}{M_{Rd.glulam_3}} = 0.714 \quad u_{V.glulam.10} := \frac{\tau_{d.10}}{f_{v.g.d}} = 0.971 \quad u_{defl.10} := \frac{w_{fin.10}}{w_{limit.10}} = 0.884$$

12 metre span

$$u_{M.glulam.12} := \frac{M_{Ed.12}}{M_{Rd.glulam_4}} = 0.668 \quad u_{V.glulam.12} := \frac{\tau_{d.12}}{f_{v.g.d}} = 0.855 \quad u_{defl.12} := \frac{w_{fin.12}}{w_{limit.12}} = 0.88$$

A2.3 LVL-beams

$k := 0..4$

A2.3.1 Material data

Bending parallell to grain, LVL (Kerto-S) $f_{m.lvl.k} := 44MPa$

Shear strength, LVL (Kerto-S) $f_{v.lvl.k} := 4.1MPa$

Elastic modulus (capacity analysis) $E_{0.lvl.05} := 11600MPa$

Elastic modulus (deformation calculations)) $E_{0.lvl.mean} := 13800MPa$

Partial factor $\gamma_{M.lvl} := 1.2$

A2.3.2 Dimensions

Width of one veneer: $w_{veneer} := 75mm$

Height of the beam

$$h_{lvl} := \begin{pmatrix} 500 \\ 750 \\ 800 \\ 1000 \\ 1200 \end{pmatrix} mm$$

Width of the beam

$$w_{lvl} := \begin{pmatrix} 3w_{veneer} \\ 3w_{veneer} \\ 3w_{veneer} \\ 3w_{veneer} \\ 3w_{veneer} \end{pmatrix}$$

Area of the beam section $A_{lvl_k} := h_{lvl_k} \cdot w_{lvl_k}$

A2.3.3 Load effect

Assuming simply supported beams

$$Q_{lvl.a_k} := 1.35 \cdot 0.89 \left(g_{floor} \cdot 1 + \rho_{lvl} \cdot A_{lvl_k} \right) + 1.5 q_{office} \cdot 1$$

eq. 6.10b in section 6.4 in SS-EN1990

$$Q_{lvl.b_k} := 1.35 \cdot \left(g_{floor} \cdot 1 + \rho_{lvl} \cdot A_{lvl_k} \right) + 1.5 \times 0.7 q_{office} \cdot 1$$

eq. 6.10a in section 6.4 in SS-EN1990

$$Q_{lvl_k} := \max\left(Q_{lvl.a_k}, Q_{lvl.b_k} \right)$$

A2.3.3.1 Bending moment

For a simply supported beam

4 metre span $$M_{Ed.lvl.4} := \frac{Q_{lvl_0} \cdot \left(l_{span_0}\right)^2}{8} = 91.424 \cdot kNm$$

6 metre span $$M_{Ed.lvl.6} := \frac{Q_{lvl_1} \cdot \left(l_{span_1}\right)^2}{8} = 207.254 \cdot kNm$$

8 metre span $$M_{Ed.lvl.8} := \frac{Q_{lvl_2} \cdot \left(l_{span_2}\right)^2}{8} = 369.004 \cdot kNm$$

10 metre span $$M_{Ed.lvl.10} := \frac{Q_{lvl_3} \cdot \left(l_{span_3}\right)^2}{8} = 580.015 \cdot kNm$$

12 metre span $$M_{Ed.lvl.12} := \frac{Q_{lvl_4} \cdot \left(l_{span_4}\right)^2}{8} = 840.185 \cdot kNm$$

A2.3.3.2 Shear force

For a simply supported beam

4 metre span $$V_{Ed.lvl.4} := \frac{Q_{lvl_0} \cdot l_{span_0}}{2} = 91.424 \cdot kN$$

6 metre span $$V_{Ed.lvl.6} := \frac{Q_{lvl_1} \cdot l_{span_1}}{2} = 138.17 \cdot kN$$

8 metre span $$V_{Ed.lvl.8} := \frac{Q_{lvl_2} \cdot l_{span_2}}{2} = 184.502 \cdot kN$$

10 metre span $$V_{Ed.lvl.10} := \frac{Q_{lvl_3} \cdot l_{span_3}}{2} = 232.006 \cdot kN$$

12 metre span $$V_{Ed.lvl.12} := \frac{Q_{lvl_4} \cdot l_{span_4}}{2} = 280.062 \cdot kN$$

A2.3.4 Bending capacity

Assuming medium term load and service class 2

Strength modification factor $k_{mod.lvl} := 0.8$

Deformation modification factor $k_{def.lvl} := 0.6$

Effect of member size

$$k_{h.lvl_k} := \begin{cases} \min\left[\left(\dfrac{300mm}{h_{lvl_k}}\right)^{0.15}, 1.2\right] & \text{if } h_{lvl_k} \le 300mm \\ 1 & \text{otherwise} \end{cases}$$

Section modulus

$$W_{lvl_k} := \frac{w_{lvl_k} \cdot \left(h_{lvl_k}\right)^2}{6}$$

Second moment of inertia

$$I_{lvl_k} := \frac{w_{lvl_k} \cdot \left(h_{lvl_k}\right)^3}{12}$$

Design value for bending parallel to grain

$$f_{m.lvl.d_k} := k_{mod.lvl} \cdot k_{h.lvl_k} \cdot \frac{f_{m.lvl.k}}{\gamma_{M.lvl}}$$

Moment capacity

$$M_{Rd.lvl_k} := f_{m.lvl.d_k} \cdot W_{lvl_k}$$

Moment capacity

$$M_{Rd.lvl} = \begin{pmatrix} 275 \\ 618.75 \\ 704 \\ 1.1 \times 10^3 \\ 1.584 \times 10^3 \end{pmatrix} \cdot kN \cdot m$$

Applied moment

$M_{Ed.lvl.4} = 91.424 \cdot kNm$

$M_{Ed.lvl.6} = 207.254 \cdot kNm$

$M_{Ed.lvl.8} = 369.004 \cdot kNm$

$M_{Ed.lvl.10} = 580.015 \cdot kNm$

$M_{Ed.lvl.12} = 840.185 \cdot kNm$

A2.3.5 Shear Capacity

A beam should fulfill the following condition regarding shear:

$\boxed{\tau_d < f_{vd}}$ eq. 6.13 in Section 6.1

Applied shear $\boxed{\tau_d := \frac{S \cdot V}{I \cdot b_{eff}}}$

First moment of inertia $\boxed{S := Ay}$

$$S_{lvl_k} := w_{lvl_k} \cdot \frac{1}{2} \cdot h_{lvl_k} \cdot \frac{1}{4} \cdot h_{lvl_k}$$

Design value for shear resistance

$$f_{v.lvl.d} := k_{mod.lvl} \cdot \frac{f_{v.lvl.k}}{\gamma_{M.lvl}} = 2.733 \cdot MPa$$

Effective width

$$b_{eff} := k_{cr} \cdot b$$

eq. 6.13a in section 6.1 in SS-EN 1995-1-1:2008

$$k_{cr.lvl} := 1.0$$

$$b_{eff.lvl_k} := k_{cr} \cdot w_{lvl_k}$$

4 metre span

$$\tau_{d.lvl.4} := \frac{S_{lvl_0} \cdot V_{Ed.lvl.4}}{I_{lvl_0} \cdot b_{eff.lvl_0}} = 1.819 \cdot MPa$$

6 metre span

$$\tau_{d.lvl.6} := \frac{S_{lvl_1} \cdot V_{Ed.lvl.6}}{I_{lvl_1} \cdot b_{eff.lvl_1}} = 1.833 \cdot MPa$$

8 metre span

$$\tau_{d.lvl.8} := \frac{S_{lvl_2} \cdot V_{Ed.lvl.8}}{I_{lvl_2} \cdot b_{eff.lvl_2}} = 2.295 \cdot MPa$$

10 metre span

$$\tau_{d.lvl.10} := \frac{S_{lvl_3} \cdot V_{Ed.lvl.10}}{I_{lvl_3} \cdot b_{eff.lvl_3}} = 2.309 \cdot MPa$$

12 metre span

$$\tau_{d.lvl.12} := \frac{S_{lvl_4} \cdot V_{Ed.lvl.12}}{I_{lvl_4} \cdot b_{eff.lvl_4}} = 2.322 \cdot MPa$$

A2.3.6 Deflection

Quasi-permanent load combination has been used with a deflection limit of span length/400.

$$w_{fin} := w_{fin.G} + w_{fin.Q}$$

$$w_{fin.G} := w_{inst.G}\left(1 + k_{def}\right)$$

$$w_{fin.Q} := w_{inst.Q}\left(1 + \psi_2 \cdot k_{def}\right)$$

$$\psi_2 := 0.3$$

4 metre span

$$w_{fin.4.lvl.G} := \frac{\left(g_{floor} \cdot 1 + \rho_{lvl} \cdot A_{lvl_0}\right) \cdot 5 \cdot \left(l_{span_0}\right)^4}{384 \cdot E_{0.lvl.mean} \cdot I_{lvl_0}} \cdot \left(1 + k_{def.lvl}\right)$$

$$w_{fin.4.lvl.Q} := \frac{q_{office} \cdot 1 \cdot 5 \cdot \left(l_{span_0}\right)^4}{384 \cdot E_{0.lvl.mean} \cdot I_{lvl_0}} \cdot \left(1 + \psi_2 \, k_{def.lvl}\right)$$

$$w_{fin.4.lvl} := w_{fin.4.lvl.G} + w_{fin.4.lvl.Q}$$

$$w_{fin.4.lvl} = 4.757 \cdot mm \qquad w_{limit.lvl.4} := \frac{4m}{400} = 10 \cdot mm$$

6 metre span

$$w_{fin.6.lvl.G} := \frac{\left(g_{floor} \cdot 1 + \rho_{lvl} \cdot A_{lvl_1}\right) \cdot 5 \cdot \left(l_{span_1}\right)^4}{384 \cdot E_{0.lvl.mean} \cdot I_{lvl_1}} \cdot \left(1 + k_{def.lvl}\right)$$

$$w_{fin.6.lvl.Q} := \frac{q_{office} \cdot 1 \cdot 5 \cdot \left(l_{span_1}\right)^4}{384 \cdot E_{0.lvl.mean} \cdot I_{lvl_1}} \cdot \left(1 + \psi_2 \, k_{def.lvl}\right)$$

$$w_{fin.6.lvl} := w_{fin.6.lvl.G} + w_{fin.6.lvl.Q}$$

$$w_{fin.6.lvl} = 7.206 \cdot mm \qquad w_{limit.lvl.6} := \frac{6m}{400} = 15 \cdot mm$$

8 metre span

$$w_{fin.8.lvl.G} := \frac{\left(g_{floor} \cdot 1 + \rho_{lvl} \cdot A_{lvl_2}\right) \cdot 5 \cdot \left(l_{span_2}\right)^4}{384 \cdot E_{0.lvl.mean} \cdot I_{lvl_2}} \cdot \left(1 + k_{def.lvl}\right)$$

$$w_{fin.8.lvl.Q} := \frac{q_{office} \cdot 1 \cdot 5 \cdot \left(l_{span_2}\right)^4}{384 \cdot E_{0.lvl.mean} \cdot I_{lvl_2}} \cdot \left(1 + \psi_2 \, k_{def.lvl}\right)$$

$$w_{fin.8.lvl} := w_{fin.8.lvl.G} + w_{fin.8.lvl.Q}$$

$$w_{fin.8.lvl} = 18.804 \cdot mm \qquad w_{limit.lvl.8} := \frac{8m}{400} = 20 \cdot mm$$

10 metre span

$$w_{fin.10.lvl.G} := \frac{\left(g_{floor} \cdot 1 + \rho_{lvl} \cdot A_{lvl_3}\right) \cdot 5 \cdot \left(l_{span_3}\right)^4}{384 \cdot E_{0.lvl.mean} \cdot I_{lvl_3}} \cdot \left(1 + k_{def.lvl}\right)$$

$$w_{fin.10.lvl.Q} := \frac{q_{office} \cdot 1 \cdot 5 \cdot \left(l_{span_3}\right)^4}{384 \cdot E_{0.lvl.mean} \cdot I_{lvl_3}} \cdot \left(1 + \psi_2 \, k_{def.lvl}\right)$$

$$w_{fin.10.lvl} := w_{fin.10.lvl.G} + w_{fin.10.lvl.Q}$$

$$w_{fin.10.lvl} = 23.69 \cdot mm \qquad w_{limit.lvl.10} := \frac{10 \cdot m}{400} = 25 \cdot mm$$

12 metre span

$$w_{fin.12.lvl.G} := \frac{\left(g_{floor} \cdot 1 + \rho_{lvl} \cdot A_{lvl_4}\right) \cdot 5 \cdot \left(l_{span_4}\right)^4}{384 \cdot E_{0.lvl.mean} \cdot I_{lvl_4}} \cdot \left(1 + k_{def.lvl}\right)$$

$$w_{fin.12.lvl.Q} := \frac{q_{office} \cdot 1 \cdot 5 \cdot \left(l_{span_4}\right)^4}{384 \cdot E_{0.lvl.mean} \cdot I_{lvl_4}} \cdot \left(1 + \psi_2 \, k_{def.lvl}\right)$$

$$w_{fin.12.lvl} := w_{fin.12.lvl.G} + w_{fin.12.lvl.Q}$$

$$w_{fin.12.lvl} = 28.649 \cdot mm \qquad w_{limit.lvl.12} := \frac{12 \cdot m}{400} = 30 \cdot mm$$

A2.3.7 Utilisation ratios

Utilisation ratios for all beams in the different spans are presented below, ratios for bending, shear and deflections.

4 metre span

Moment utilisation	Shear utilisation	Deflection utilisation
$u_{M.lvl.4} := \dfrac{M_{Ed.lvl.4}}{M_{Rd.lvl_0}} = 0.332$	$u_{V.lvl.4} := \dfrac{\tau_{d.lvl.4}}{f_{v.lvl.d}} = 0.666$	$u_{lvl.4} := \dfrac{w_{fin.4.lvl}}{w_{limit.lvl.4}} = 0.476$

6 metre span

$$u_{M.lvl.6} := \frac{M_{Ed.lvl.6}}{M_{Rd.lvl_1}} = 0.335 \qquad u_{V.lvl.6} := \frac{\tau_{d.lvl.6}}{f_{v.lvl.d}} = 0.671 \qquad u_{lvl.6} := \frac{w_{fin.6.lvl}}{w_{limit.lvl.6}} = 0.48$$

8 metre span

$$u_{M.lvl.8} := \frac{M_{Ed.lvl.8}}{M_{Rd.lvl_2}} = 0.524 \qquad u_{V.lvl.8} := \frac{\tau_{d.lvl.8}}{f_{v.lvl.d}} = 0.84 \qquad u_{lvl.8} := \frac{w_{fin.8.lvl}}{w_{limit.lvl.8}} = 0.94$$

10 metre span

$$u_{M.lvl.10} := \frac{M_{Ed.lvl.10}}{M_{Rd.lvl_3}} = 0.527 \qquad u_{V.lvl.10} := \frac{\tau_{d.lvl.10}}{f_{v.lvl.d}} = 0.845 \qquad u_{lvl.10} := \frac{w_{fin.10.lvl}}{w_{limit.lvl.10}} = 0.948$$

12 metre span

$$u_{M.lvl.12} := \frac{M_{Ed.lvl.12}}{M_{Rd.lvl_4}} = 0.53 \qquad u_{V.lvl.12} := \frac{\tau_{d.lvl.12}}{f_{v.lvl.d}} = 0.85 \qquad u_{lvl.12} := \frac{w_{fin.12.lvl}}{w_{limit.lvl.12}} = 0.955$$

$kNm := kN \cdot m$

Appendix A2b: Beams - fire load case

This Appendix is a shorter version of Appendix A2a, where the load case of fire is handled. The obtained beam dimension are checked against the fire load case.

A2.1 Loads (the same as in the ULS load case)

Office load + partition walls and self-weight + installations $q_{office} := 3 \frac{kN}{m^2}$ $g_{floor} := 2.5 \frac{kN}{m^2}$

Density for glulam and LVL $\rho_{glulam} := 4300 \frac{N}{m^3}$ $\rho_{lvl} := 5100 \frac{N}{m^3}$

Assumed tributary lenght $l := 6m$

A2.2 Glulam beams

$i := 0..4$

A2.2.1 Material data (the same as in ULS load case)

Bending parallell to grain and shear strength $f_{m.g.k} := 30.8MPa$ $f_{v.g.k} := 3.5MPa$

Elastic modulus $E_{0.g.05} := 10500MPa$ $E_{0.g.mean} := 13000MPa$

Partial factor $\gamma_{M.glulam} := 1.25$

A2.2.2 Dimensions

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

Charing depth (Fire velocity in the wood $d_{char.0} := 0.65 \frac{mm}{min} \cdot 90min = 58.5 \cdot mm$

Height and width of beam

$$h_{glulam} := \begin{pmatrix} 15 \cdot h_{lamell} - d_{char.0} \\ 17 \cdot h_{lamell} - d_{char.0} \\ 19 \cdot h_{lamell} - d_{char.0} \\ 23 \cdot h_{lamell} - d_{char.0} \\ 26 \cdot h_{lamell} - d_{char.0} \end{pmatrix} = \begin{pmatrix} 0.616 \\ 0.707 \\ 0.796 \\ 0.976 \\ 1.111 \end{pmatrix} m \qquad w_{glulam} := \begin{pmatrix} 165mm - 2 \cdot d_{char.0} \\ 190mm - 2 \cdot d_{char.0} \\ 215mm - 2 \cdot d_{char.0} \\ 2 \cdot 115mm - 2 \cdot d_{char.0} \\ 2 \cdot 140mm - 2 \cdot d_{char.0} \end{pmatrix} = \begin{pmatrix} 0.048 \\ 0.073 \\ 0.098 \\ 0.113 \\ 0.163 \end{pmatrix} m$$

Area of beam section $A_{glulam_i} := h_{glulam_i} \cdot w_{glulam_i}$

A2.2.3 Load combinations and load effects in case of fire

Assuming simply supported beams to obtain worst case.

$Q_{glulam_i} := g_{floor} \cdot l + \rho_{glulam} \cdot A_{glulam_i} + 0.5 \cdot q_{office} \cdot l$

$$Q_{glulam} = \begin{pmatrix} 24.127 \\ 24.222 \\ 24.336 \\ 24.474 \\ 24.779 \end{pmatrix} \cdot \frac{kN}{m}$$

Span lengths

$$l_{span} := \begin{pmatrix} 4 \\ 6 \\ 8 \\ 10 \\ 12 \end{pmatrix} m$$

A2.2.3.1 Bending moment for a simply supported beam

$$M_{Ed.4} := \frac{Q_{glulam_0} \cdot \left(l_{span_0}\right)^2}{8} = 48.254 \cdot kNm \qquad M_{Ed.6} := \frac{Q_{glulam_1} \cdot \left(l_{span_1}\right)^2}{8} = 108.998 \cdot kNm$$

$$M_{Ed.8} := \frac{Q_{glulam_2} \cdot \left(l_{span_2}\right)^2}{8} = 194.685 \cdot kNm \qquad M_{Ed.10} := \frac{Q_{glulam_3} \cdot \left(l_{span_3}\right)^2}{8} = 305.931 \cdot kNm$$

$$M_{Ed.12} := \frac{Q_{glulam_4} \cdot \left(l_{span_4}\right)^2}{8} = 446.023 \cdot kNm$$

A2.2.3.2 Shear force for a simply suppoerted beam

$$V_{Ed.4} := \frac{Q_{glulam_0} \cdot l_{span_0}}{2} = 48.254 \cdot kN \qquad V_{Ed.6} := \frac{Q_{glulam_1} \cdot l_{span_1}}{2} = 72.665 \cdot kN$$

$$V_{Ed.8} := \frac{Q_{glulam_2} \cdot l_{span_2}}{2} = 97.343 \cdot kN \qquad V_{Ed.10} := \frac{Q_{glulam_3} \cdot l_{span_3}}{2} = 122.372 \cdot kN$$

$$V_{Ed.12} := \frac{Q_{glulam_4} \cdot l_{span_4}}{2} = 148.674 \cdot kN$$

A2.2.4 Moment capacity

Strength modification factor $\quad k_{mod.fi} := 1$

$k_{fi.glulam} := 1.15$

$\gamma_{M.fi} := 1$

Section modulus $\quad W_{glulam_i} := \dfrac{w_{glulam_i} \cdot \left(h_{glulam_i}\right)^2}{6}$

Second moment of inertia $\quad I_{glulam_i} := \dfrac{w_{glulam_i} \cdot \left(h_{glulam_i}\right)^3}{12}$

Design value for shear $f_{v.g.fi} := k_{mod.fi} \cdot k_{fi.glulam} \cdot \frac{f_{v.g.k}}{\gamma_{M.fi}} = 4.025 \cdot MPa$ $f_{v.g.d} := f_{v.g.fi}$

Design value for bending parallel to grain $f_{m.g.fi} := k_{mod.fi} \cdot k_{fi.glulam} \cdot \frac{f_{m.g.k}}{\gamma_{M.fi}} = 35.42 \cdot MPa$

Moment capacity $M_{Rd.glulam_i} := f_{m.g.fi} \cdot W_{glulam_i}$

Moment capacity	Applied moment
$M_{Rd.glulam} = \begin{pmatrix} 107.697 \\ 215.102 \\ 367.024 \\ 636.092 \\ 1.189 \times 10^3 \end{pmatrix} \cdot kN \cdot m$	$M_{Ed.4} = 48.254 \cdot kNm$ $M_{Ed.6} = 108.998 \cdot kNm$ $M_{Ed.8} = 194.685 \cdot kNm$ $M_{Ed.10} = 305.931 \cdot kNm$ $M_{Ed.12} = 446.023 \cdot kNm$

A2.2.5 Shear capacity

First moment of inertia $S_i := w_{glulam_i} \cdot \frac{1}{2} \cdot h_{glulam_i} \cdot \frac{1}{4} \cdot h_{glulam_i}$

Effective width $k_{cr} := 0.67$ <u>eq. 6.13a in section 6.1 in SS-EN 1995-1-1:2008</u>

$b_{eff_i} := k_{cr} \cdot w_{glulam_i}$

4 metre span $\tau_{d.4} := \frac{S_0 \cdot V_{Ed.4}}{I_{glulam_0} \cdot b_{eff_0}} = 3.651 \cdot MPa$ $\tau_d < f_{vd}$ OK

6 metre span $\tau_{d.6} := \frac{S_1 \cdot V_{Ed.6}}{I_{glulam_1} \cdot b_{eff_1}} = 3.154 \cdot MPa$ $\tau_d < f_{vd}$ OK

8 metre span $\tau_{d.8} := \frac{S_2 \cdot V_{Ed.8}}{I_{glulam_2} \cdot b_{eff_2}} = 2.792 \cdot MPa$ $\tau_d < f_{vd}$ OK

10 metre span $\tau_{d.10} := \frac{S_3 \cdot V_{Ed.10}}{I_{glulam_3} \cdot b_{eff_3}} = 2.483 \cdot MPa$ $\tau_d < f_{vd}$ OK

12 metre span $\tau_{d.12} := \frac{S_4 \cdot V_{Ed.12}}{I_{glulam_4} \cdot b_{eff_4}} = 1.837 \cdot MPa$ $\tau_d < f_{vd}$ OK

A.2.2.6 Utilisation ratios in load case fire

	Moment utilisation	Deflection utilisation
4 metre span	$u_{M.glulam.4} := \frac{M_{Ed.4}}{M_{Rd.glulam_0}} = 0.448$	$u_{V.glulam.4} := \frac{\tau_{d.4}}{f_{v.g.d}} = 0.907$
6 metre span	$u_{M.glulam.6} := \frac{M_{Ed.6}}{M_{Rd.glulam_1}} = 0.507$	$u_{V.glulam.6} := \frac{\tau_{d.6}}{f_{v.g.d}} = 0.784$
8 metre span	$u_{M.glulam.8} := \frac{M_{Ed.8}}{M_{Rd.glulam_2}} = 0.53$	$u_{V.glulam.8} := \frac{\tau_{d.8}}{f_{v.g.d}} = 0.694$
10 metre span	$u_{M.glulam.10} := \frac{M_{Ed.10}}{M_{Rd.glulam_3}} = 0.481$	$u_{V.glulam.10} := \frac{\tau_{d.10}}{f_{v.g.d}} = 0.617$
12 metre span	$u_{M.glulam.12} := \frac{M_{Ed.12}}{M_{Rd.glulam_4}} = 0.375$	$u_{V.glulam.12} := \frac{\tau_{d.12}}{f_{v.g.d}} = 0.456$

A2.3 LVL-beams

$k := 0..4$

A2.3.1 Material data

Bending parallell to grain and shear strength $f_{m.lvl.k} := 44MPa$ $\quad f_{v.lvl.k} := 4.1MPa$

Elastic modulus $E_{0.lvl.05} := 11600MPa$ $\quad E_{0.lvl.mean} := 13800MPa$

Partial factor $\gamma_{M.lvl} := 1.2$

A2.3.2 Dimensions

Width of one veneer: $w_{veneer} := 75mm$

Height and width of the beam

$$h_{lvl} := \begin{pmatrix} 500mm - d_{char.0} \\ 750mm - d_{char.0} \\ 800mm - d_{char.0} \\ 1000mm - d_{char.0} \\ 1200mm - d_{char.0} \end{pmatrix} = \begin{pmatrix} 0.442 \\ 0.692 \\ 0.742 \\ 0.941 \\ 1.141 \end{pmatrix} m \qquad w_{lvl} := \begin{pmatrix} 3w_{veneer} - 2\cdot d_{char.0} \\ 3w_{veneer} - 2\cdot d_{char.0} \\ 3w_{veneer} - 2\cdot d_{char.0} \\ 3w_{veneer} - 2\cdot d_{char.0} \\ 3w_{veneer} - 2\cdot d_{char.0} \end{pmatrix} = \begin{pmatrix} 0.108 \\ 0.108 \\ 0.108 \\ 0.108 \\ 0.108 \end{pmatrix} m$$

Area of the beam section $A_{lvl_k} := h_{lvl_k}\cdot w_{lvl_k}$

A2.3.3 Load effect

$$Q_{lvl_k} := g_{floor}\cdot l + \rho_{lvl}\cdot A_{lvl_k} + 0.5\cdot q_{office}\cdot l \qquad Q_{lvl} = \begin{pmatrix} 24.243 \\ 24.381 \\ 24.408 \\ 24.519 \\ 24.629 \end{pmatrix}\cdot\frac{kN}{m}$$

A2.3.3.1 Bending moment

$$M_{Ed.lvl.4} := \frac{Q_{lvl_0}\cdot\left(l_{span_0}\right)^2}{8} = 48.486\cdot kNm \qquad M_{Ed.lvl.6} := \frac{Q_{lvl_1}\cdot\left(l_{span_1}\right)^2}{8} = 109.714\cdot kNm$$

$$M_{Ed.lvl.8} := \frac{Q_{lvl_2}\cdot\left(l_{span_2}\right)^2}{8} = 195.267\cdot kNm \qquad M_{Ed.lvl.10} := \frac{Q_{lvl_3}\cdot\left(l_{span_3}\right)^2}{8} = 306.482\cdot kNm$$

$$M_{Ed.lvl.12} := \frac{Q_{lvl_4}\cdot\left(l_{span_4}\right)^2}{8} = 443.317\cdot kNm$$

A2.3.3.2 Shear force

$$V_{Ed.lvl.4} := \frac{Q_{lvl_0}\cdot l_{span_0}}{2} = 48.486\cdot kN \qquad V_{Ed.lvl.6} := \frac{Q_{lvl_1}\cdot l_{span_1}}{2} = 73.143\cdot kN$$

$$V_{Ed.lvl.8} := \frac{Q_{lvl_2}\cdot l_{span_2}}{2} = 97.634\cdot kN \qquad V_{Ed.lvl.10} := \frac{Q_{lvl_3}\cdot l_{span_3}}{2} = 122.593\cdot kN$$

$$V_{Ed.lvl.12} := \frac{Q_{lvl_4}\cdot l_{span_4}}{2} = 147.772\cdot kN$$

A2.3.4 Bending capacity

Strength modification factor for LVL $k_{fi.lvl} := 1.1$

Section modulus $$W_{lvl_k} := \frac{w_{lvl_k} \cdot \left(h_{lvl_k}\right)^2}{6}$$

Second moment of inertia $$I_{lvl_k} := \frac{w_{lvl_k} \cdot \left(h_{lvl_k}\right)^3}{12}$$

Design value for bending parallel to grain $$f_{m.lvl.fi_k} := k_{mod.fi} \cdot k_{fi.lvl} \cdot \frac{f_{m.lvl.k}}{\gamma_{M.fi}}$$

Moment capacity $$M_{Rd.lvl_k} := f_{m.lvl.fi_k} \cdot W_{lvl_k}$$

Moment capacity

$$M_{Rd.lvl} = \begin{pmatrix} 169.816 \\ 416.584 \\ 479.005 \\ 772.251 \\ 1.135 \times 10^3 \end{pmatrix} \cdot kN \cdot m$$

Applied moment

$M_{Ed.lvl.4} = 48.486 \cdot kNm$

$M_{Ed.lvl.6} = 109.714 \cdot kNm$

$M_{Ed.lvl.8} = 195.267 \cdot kNm$

$M_{Ed.lvl.10} = 306.482 \cdot kNm$

$M_{Ed.lvl.12} = 443.317 \cdot kNm$

A2.3.5 Shear Capacity

First moment of inertia $$S_{lvl_k} := w_{lvl_k} \cdot \frac{1}{2} \cdot h_{lvl_k} \cdot \frac{1}{4} \cdot h_{lvl_k}$$

Design value for shear resistance $$f_{v.lvl.fi} := k_{mod.fi} \cdot k_{fi.lvl} \cdot \frac{f_{v.lvl.k}}{\gamma_{M.fi}} = 4.51 \cdot MPa \qquad f_{v.lvl.d} := f_{v.lvl.fi}$$

Effective width $k_{cr.lvl} := 1.0$

eq. 6.13a in section 6.1 in SS-EN 1995-1-1:2008

$$b_{eff.lvl_k} := k_{cr} \cdot w_{lvl_k}$$

Design value for shear

4 metre span $$\tau_{d.lvl.4} := \frac{S_{lvl_0} \cdot V_{Ed.lvl.4}}{I_{lvl_0} \cdot b_{eff.lvl_0}} = 2.277 \cdot MPa$$

6 metre span $$\tau_{d.lvl.6} := \frac{S_{lvl_1} \cdot V_{Ed.lvl.6}}{I_{lvl_1} \cdot b_{eff.lvl_1}} = 2.193 \cdot MPa$$

8 metre span

$$\tau_{d.lvl.8} := \frac{S_{lvl_2} \cdot V_{Ed.lvl.8}}{I_{lvl_2} \cdot b_{eff.lvl_2}} = 2.729 \cdot MPa$$

10 metre span

$$\tau_{d.lvl.10} := \frac{S_{lvl_3} \cdot V_{Ed.lvl.10}}{I_{lvl_3} \cdot b_{eff.lvl_3}} = 2.699 \cdot MPa$$

12 metre span

$$\tau_{d.lvl.12} := \frac{S_{lvl_4} \cdot V_{Ed.lvl.12}}{I_{lvl_4} \cdot b_{eff.lvl_4}} = 2.684 \cdot MPa$$

A2.3.6 Utilisation ratios for fire load case

	Moment utilisation	Deflection utilisation
4 metre span	$u_{M.lvl.4} := \frac{M_{Ed.lvl.4}}{M_{Rd.lvl_0}} = 0.286$	$u_{V.lvl.4} := \frac{\tau_{d.lvl.4}}{f_{v.lvl.d}} = 0.505$
6 metre span	$u_{M.lvl.6} := \frac{M_{Ed.lvl.6}}{M_{Rd.lvl_1}} = 0.263$	$u_{V.lvl.6} := \frac{\tau_{d.lvl.6}}{f_{v.lvl.d}} = 0.486$
8 metre span	$u_{M.lvl.8} := \frac{M_{Ed.lvl.8}}{M_{Rd.lvl_2}} = 0.408$	$u_{V.lvl.8} := \frac{\tau_{d.lvl.8}}{f_{v.lvl.d}} = 0.605$
10 metre span	$u_{M.lvl.10} := \frac{M_{Ed.lvl.10}}{M_{Rd.lvl_3}} = 0.397$	$u_{V.lvl.10} := \frac{\tau_{d.lvl.10}}{f_{v.lvl.d}} = 0.598$
12 metre span	$u_{M.lvl.12} := \frac{M_{Ed.lvl.12}}{M_{Rd.lvl_4}} = 0.391$	$u_{V.lvl.12} := \frac{\tau_{d.lvl.12}}{f_{v.lvl.d}} = 0.595$

Appendix A3: Composite floor structure with timber and concrete

$kNm := kN \cdot m$

This Appendix shows the calculations performed for the composite floor, results are presented in Section 5.4. Load combinations have been performed in accordance with **EN 1995-1-1**. All references made reffers to this Eurocode. Equations in Linden (1999) have been us for the calculations

A3.1 Material and geometric data

The floor structure consists of a web in timber and a flange in concrete. Reinforcement is neglected since the concrete is compressed and thereby uncracked. A centrum distance of 600 mm is assumed for the webs.

Floor span $l_{floor} := 12m$

Width of the floor $b_{floor} := l_{floor}$

Spacing between webs $s_{web} := 0.5m$

Height of the web $h_{web} := 655mm$

Width of the web $w_{web} := 220mm$

Height of the flange $h_{flange} := 70mm$

Distance between the webs $b := \frac{s_{web}}{2} - \frac{w_{web}}{2} = 0.14\,m$

Effecftive width of the flange

$$b_{eff.1a} := \begin{cases} (0.2 \cdot b + 0.1 \cdot l_{floor}) & \text{if } 0.2 \cdot b + 0.1 \cdot l_{floor} \le 0.2 \cdot l_{floor} \\ (0.2 \cdot l_{floor}) & \text{otherwise} \end{cases}$$

$$b_{eff.1} := \begin{cases} (b_{eff.1a}) & \text{if } b_{eff.1a} \le b \\ (b) & \text{otherwise} \end{cases}$$

$$b_{eff} := 2 \cdot b_{eff.1} + w_{web} = 0.5\,m$$

$$w_{flange} := b_{eff} = 0.5\,m$$

Area of the web and the flange $A_{web} := h_{web} \cdot w_{web}$ $\quad A_{flange} := h_{flange} \cdot w_{flange}$

A3.1.1 Concrete C30/35

Characteristic compression strength $f_{ck} := 30MPa$

$f_{cm} := 38MPa$

Characteristic tension strength $f_{ctk} := 2.0MPa$

Partial factor $\gamma_M := 1.5$

Design strengths $f_{c.d} := \dfrac{f_{ck}}{\gamma_M} = 20 \cdot MPa$ $\qquad f_{ctd} := \dfrac{f_{ctk}}{\gamma_M} = 1.333 \cdot MPa$

Self-weight $\rho_{concrete} := 25 \dfrac{kN}{m^3}$

A3.1.1.1 Value for creep coefficient

Creep coefficient $\boxed{\varphi := \varphi_{RH} \cdot \beta(f_{cm}) \cdot \beta(t_0)}$

Indoor environment, assumed $RH := 50\%$

Notional size $h_0 := \dfrac{2 \cdot A_{flange}}{2 \cdot (h_{flange} + w_{flange})} = 0.061\,m$

Factor to allow for the effect of relative humidity $f_{cm} > 35MPa$

$$\varphi_{RH} := \left[1 + \frac{1 - RH}{0.1 \sqrt[3]{h_0 \cdot \frac{1000}{m}}} \cdot \left(\frac{35MPa}{f_{cm}}\right)^{0.7}\right] \cdot \left(\frac{35MPa}{f_{cm}}\right)^{0.2} = 2.161$$

eq. B3b in Appendix B

Factor to allow for the effect of concrete strength $\beta_{fcm} := 2.73$

Factor to allow for the effect of concrete age at loading (Assuming loading after 28 days) $\beta_{t0} := \dfrac{1}{0.1 + 28^{0.2}} = 0.488$ eq. B.5 in Appendix B

Creep coefficient $\varphi := \varphi_{RH} \cdot \beta_{fcm} \cdot \beta_{t0} = 2.881$

A3.1.1.2 Effective elastic modulus

Mean value of the elastic modulus $E_{cm} := 33GPa$

Effective elastic modulus $E_{c.ef} := \frac{E_{cm}}{\varphi + 1} = 8.503 \cdot GPa$

A3.1.2 Solid wood C27

Characteristic tension strength (parallel to grain) $f_{tk} := 16MPa$

Characteristic bending strength $f_{tmk} := 27MPa$

Characteristic shear strength $f_{vk} := 4MPa$

Strength modification factor (solid, service class 1, medium term action) $k_{mod} := 0.8$

Factor regarding size effects

$$k_h := \begin{cases} \min\left[\left(\frac{150mm}{w_{web}}\right)^{0.2}, 1.3\right] & \text{if } w_{web} \le 150mm \\ 1 & \text{otherwise} \end{cases}$$

Partial factor $\gamma_{M.solid} := 1.3$

Design strengths

$$f_{td} := k_{mod} \cdot k_h \cdot \frac{f_{tk}}{\gamma_{M.solid}} = 9.846 \cdot MPa$$

$$f_{tmd} := k_{mod} \cdot \frac{f_{tmk}}{\gamma_{M.solid}} = 16.615 \cdot MPa$$

$$f_{vd} := k_{mod} \cdot \frac{f_{vk}}{\gamma_{M.solid}} = 2.462 \cdot MPa$$

Self-weight $\rho_{solid} := 4.2 \frac{kN}{m^3}$

Elastic modulus

$E_{0.05} := 7700MPa$ (Capacity analysis)

$E_{0.mean} := 11500MPa$ (Deformation calculations)

$k_{def} := 0.6$

$$E_{mean.fin} := \frac{E_{0.mean}}{1 + k_{def}} = 7.188 \cdot \text{GPa}$$

The final modulus of elasticity is the same for both ultimate limit state and serviceability state since the permanent load is the leading action according to EN1995-1-1 section 2.3.2.2 (2).

A3.2 Transformed cross-section

The concrete part is transformed to timber by the factor α.ef.

$$\alpha_{ef} := \frac{E_{c.ef}}{E_{mean.fin}} = 1.183$$

Effective area

$$A_{ef} := A_{web} + \alpha_{ef} \cdot A_{flange} = 0.186\,\text{m}^2$$

Centre of ravity

$$x_{ef} := \frac{A_{web} \cdot \left(h_{flange} + \frac{h_{web}}{2}\right) + \alpha_{ef} \cdot A_{flange} \cdot \frac{h_{flange}}{2}}{A_{ef}} = 0.317\,\text{m}$$

Second moment of inertia

$$I_{tot} := \frac{w_{web} \cdot h_{web}^{3}}{12} + \alpha_{ef} \cdot \frac{w_{flange} \cdot h_{flange}^{3}}{12} = 5.169 \times 10^{-3}\,\text{m}^4$$

Equivalent stiffness for the section

$$\boxed{EI_{ef} := E_{mean.fin} \cdot \left[I_{tot} + \gamma \cdot \left(\alpha_{ef} \cdot A_{flange} \cdot e_{flange}^{2} + A_{web} \cdot e_{web}^{2}\right)\right]}$$

Assuming spacing of 250 mm

$$s_{connector} := 0.20\text{m}$$

Slip modulus (assumed after studying (van der Linden 1999))

$$K_i := 90 \frac{\text{kN}}{\text{mm}}$$

Smeared slip modulus

$$k_i := \frac{K_i}{s_{connector}} = 0.45 \cdot \text{GPa}$$

$$p := E_{c.ef} \cdot \left(\frac{\pi}{l_{floor}}\right)^2 \cdot \frac{1}{k_i} \cdot \frac{A_{web} \cdot A_{flange}}{A_{web} + \alpha_{ef} \cdot A_{flange}} = 0.035$$

Combination factor that denotes the effectiveness of the total connection

$$\gamma_{test} := \frac{1}{1 + p} = 0.966$$

The combination factor is assumed to be

$$\gamma := 0.93$$

$$e_{web} := h_{flange} + \frac{h_{web}}{2} - x_{ef} = 0.081\,m$$

$$e_{flange} := x_{ef} - \frac{h_{flange}}{2} = 0.282\,m$$

Effective stiffnes

$$EI_{ef} := E_{mean.fin} \cdot \left[I_{tot} + \gamma \cdot \left(\alpha_{ef} \cdot A_{flange} \cdot e_{flange}^{\ 2} + A_{web} \cdot e_{web}^{\ 2} \right) \right] = 6.54 \times 10^{7} \cdot N \cdot m^{2}$$

A3.3 Load combination

Imposed load, office building including partition walls

$$q_{office} := (2.5 + 0.5) \frac{kN}{m^{2}}$$

$$Q_{office} := q_{office} \cdot s_{web} = 1.5 \cdot \frac{kN}{m}$$

Self-weight floor structure, including installations

$$Q_{floor} := A_{web} \cdot \rho_{solid} + A_{flange} \cdot \rho_{concrete} + s_{web} \cdot 0.5 \frac{kN}{m^{2}} = 1.73 \cdot \frac{kN}{m}$$

$$Q_a := 1.35 \cdot \left(Q_{floor} \right) + 1.5 \cdot 0.7 Q_{office} = 3.911 \cdot \frac{kN}{m}$$

eq. 6.10a in section 6.4 in SS-EN1990

$$Q_b := 1.35 \cdot 0.89 \left(Q_{floor} \right) + 1.5 Q_{office} = 4.329 \cdot \frac{kN}{m}$$

eq. 6.10b in section 6.4 in SS-EN1990

$$Q := \max\left(Q_a , Q_b \right) = 4.329 \cdot \frac{kN}{m}$$

A3.3.1 Load effects

Applied moment

$$M_{Ed} := \frac{Q \cdot l_{floor}^{\ 2}}{8} = 77.919 \cdot kNm$$

Concrete stress at top

$$\sigma_{c.c} := -E_{c.ef} \cdot \frac{M_{Ed}}{EI_{ef}} \cdot \left(\gamma \cdot e_{flange} + \frac{h_{flange}}{2} \right) = -3.007 \cdot MPa$$

Concrete stress in connection

$$\sigma_{c.t} := -E_{c.ef} \cdot \frac{M_{Ed}}{EI_{ef}} \cdot \left(\gamma \cdot e_{flange} - \frac{h_{flange}}{2} \right) = -2.298 \cdot MPa$$

Timber stress in connection $\sigma_{t.c} := E_{mean.fin} \cdot \frac{M_{Ed}}{EI_{ef}} \cdot \left(\gamma \cdot e_{web} - \frac{h_{web}}{2} \right) = -2.16 \cdot MPa$

Timber stress in gravity centre of the web $\sigma_{t.t} := E_{mean.fin} \cdot \frac{M_{Ed}}{EI_{ef}} \cdot \gamma \cdot e_{web} = 0.644 \cdot MPa$

Timber stress in the bottom $\sigma_{t.m} := E_{mean.fin} \cdot \frac{M_{Ed}}{EI_{ef}} \left(\gamma \cdot e_{web} + \frac{h_{web}}{2} \right) = 3.449 \cdot MPa$

$$z_0 := \frac{h_{web} \cdot \sigma_{t.m}}{\sigma_{t.m} + \left|\sigma_{t.c}\right|} = 0.403\, m$$

Shear stress $\sigma_{tvd} := \frac{Q \cdot l_{floor} \cdot E_{mean.fin}}{2 \cdot EI_{ef}} \cdot z_0^{\,2} = 0.463 \cdot MPa$

A3.4 Check of performance in SLS

A3.4.1Final deflection

Deflection from permanent load $w_{fin.G} := \frac{\left(Q_{floor}\right) \cdot 5 \cdot l_{floor}^{\,4}}{384 \cdot EI_{ef}} = 7.143 \times 10^{-3}\, m$

Deflection from imposed loads $w_{fin.Q} := \frac{Q_{office} \cdot 5 \cdot l_{floor}^{\,4}}{384 \cdot EI_{ef}} = 6.193 \times 10^{-3}\, m$

Total deflection $w_{fin} := w_{fin.G} + w_{fin.Q}$

Comparing to the limit $w_{fin} = 13.335 \cdot mm$ $\qquad w_{limit} := \frac{l_{floor}}{500} = 24 \cdot mm$

A3.4.2 Fundamental frequency

Volume of the concrete part $V_c := h_{flange} \cdot w_{flange} \cdot l_{floor} = 0.42 \cdot m^3$

Volume of the timber part $V_t := h_{web} \cdot w_{web} \cdot l_{floor} = 1.729 \cdot m^3$

Area of one floor element $A_{floor} := l_{floor} \cdot s_{web} = 6\, m^2$

Mass of the floor structure $$m_{floor} := \frac{\rho_{concrete} \cdot V_c + \rho_{solid} \cdot V_t}{A_{floor}} \cdot \frac{1}{10} \frac{kg}{N} = 296.044 \frac{kg}{m^2}$$

Fundamental frequency $$f_1 := \frac{\pi}{2 \cdot l_{floor}^{\,2}} \cdot \sqrt{\frac{\frac{EI_{ef}}{s_{web}}}{m_{floor}}} = 7.251 \cdot Hz$$

A3.4.3 Instantaneous deflection

The instantaneous deflection should satisfy the following equation:

$$\frac{w}{F} \le a$$ <u>eq. 7.3 in section 7.3</u>

Where w is the maximum instantaneous vertical deflection caused by an applied load of 1 kN at the point of the floor which gives the maximum response and a is a static criterion which is less than 1.5 when deflection under 1kN point load.

Static criterion (assuming average performance of the floor) $$a := 1.5 \frac{mm}{kN}$$

$$P := 1kN$$

Deflection: $$w := \frac{P \cdot l_{floor}^{\,3}}{48 \cdot EI_{ef}} = 0.55 \cdot mm$$

Check of cdeflection criterion $$\frac{w}{P} \le a = 1$$ 1=OK

A3.4.4 Velocity response

Second moment of inertia $$I_{plate} := \alpha_{ef} \cdot \frac{l_{floor} \cdot h_{flange}^{\,3}}{12} = 4.058 \times 10^{-4} m^4$$

Stiffness of the floor about axis parallell to the beams $$EI_b := \alpha_{ef} \cdot E_{mean.fin} \cdot I_{plate} = 3.45 \times 10^6 \cdot N \cdot m^2$$

Number of eigenmodes with eigenfrequencies lower than 40 Hz

$$n_{40} := \left[\left[\left(\frac{40Hz}{f_1} \right)^2 - 1 \right] \left(\frac{b_{floor}}{l_{floor}} \right)^4 \cdot \frac{EI_{ef}}{EI_b} \right]^{0.25} = 4.86$$

<u>eq. 7.7 in Section 7.3</u>

Peak velocity due to impulse for a rectangular floor system simply supported on all four sides

$$v := \frac{4\cdot(0.4 + 0.6\cdot n_{40})}{m_{floor}\cdot b_{floor}\cdot l_{floor} + 200\cdot kg} = 3.097 \times 10^{-4}\cdot\frac{m}{N\cdot s^2}$$

eq. 7.6 in Section 7.3

The velocity respons should fulfill:

$$v \le b^{(f_1\cdot\zeta-1)}$$

eq. 7.4 in Section 7.3

Figure 7.2 in Eurocode 5 gives the relation between a (from the calculation of the instantaneous deflection) and b

$$b := 120$$

Modal damping ratio: $\zeta := 0.01$

7.3.1 (3) in Section 7.3

Criterion $v = 3.097 \times 10^{-4}\cdot\frac{m}{N\cdot s^2}$ < $b^{\frac{f_1\cdot\zeta}{Hz}-1}\cdot 1\frac{m}{N\cdot s^2} = 0.012\cdot\frac{m}{N\cdot s^2}$ OK

A3.5 Check of the capacity

Concrete stress at top $u_{c.c} := \dfrac{|\sigma_{c.c}|}{f_{c.d}} = 0.15$

Concrete stress in connection $u_{ct} := \begin{cases} \dfrac{|\sigma_{c.c}|}{f_{c.d}} & \text{if } \sigma_{c.c} < 0 \\ \dfrac{\sigma_{c.c}}{f_{ctd}} & \text{otherwise} \end{cases}$ $u_{ct} = 0.15$

Timber stress in connection $u_{t.c} := \begin{cases} \dfrac{|\sigma_{t.c}|}{f_{tmd}} & \text{if } \sigma_{t.c} < 0 \\ \dfrac{\sigma_{t.c}}{f_{td}} & \text{otherwise} \end{cases}$ $u_{t.c} = 0.13$

Timber stress in gravity centre of the web $u_{t.t} := \begin{cases} \dfrac{|\sigma_{t.t}|}{f_{tmd}} & \text{if } \sigma_{t.t} < 0 \\ \dfrac{\sigma_{t.t}}{f_{td}} & \text{otherwise} \end{cases}$ $u_{t.t} = 0.065$

Timber stress in the bottom $u_{t.m} := \dfrac{|\sigma_{t.m}|}{f_{tmd}} = 0.208$

Shear stress $u_{tvd} := \dfrac{|\sigma_{tvd}|}{f_{vd}} = 0.188$

Deflection $u_{defl} := \dfrac{w_{fin}}{w_{limit}} = 0.556$

Fundamental frequency $f_1 \geq 7Hz = 1$ $f_1 = 7.251 \cdot Hz$ limit is 7Hz

Instantaneous deflection $\dfrac{\frac{w}{P}}{a} = 0.367$

Velocity response $\dfrac{v}{b^{\left(\frac{f_1}{Hz} \cdot \zeta - 1\right)} \cdot \frac{1}{kg}} = 0.026$

Appendix A4: Wall calculations

$kNm := kN \cdot m$

Results from this Appendix are shown in Section 5.6. This Appendix shows the calculations performed when designing the walls.

4.1 General geometric data

Length of wall $L_{wall} := \begin{bmatrix} 0.8 \\ 1 \end{bmatrix} m$

Height of wall $h_{wall} := 3.6\ m$

Influencing length for imposed loads $l_{inf} := 6\ m$

Height of windows $h_{window} := 1.7\ m$

Width of windows $w_{window} := 1.6\ m$

Height of beam $h_{beam} := \dfrac{h_{wall} - h_{window}}{2} = 0.95\ m$

A4.2 General loads

A4.2.1 Imposed loads

Loads acting on the floors which is supported by the walls and wind loads acting on the walls, creating a bending moment in the walls.

Office load $q_{office} := 2.5\ \dfrac{kN}{m^2}$

Partition walls $q_{walls} := 0.5\ \dfrac{kN}{m^2}$

Total imposed load $q_{imp} := l_{inf} \cdot (q_{office} + q_{walls}) = 18\ \dfrac{kN}{m}$

A4.2.2 Permanent loads

Self weight from floors $g_{floor} := 2\ \dfrac{kN}{m^2}$

Installation load $g_{installations} := 0.5\ \dfrac{kN}{m^2}$

Total permanent load $g_p := l_{inf} \cdot (g_{floor} + g_{installations}) = 15\ \dfrac{kN}{m}$

Reduction factor, for the number of storeys $\alpha_{n1} := \dfrac{2 + (14-2) \cdot 0.7}{14} = 0.743$

$$\alpha_{n6} := \frac{2+(9-2)\cdot 0.7}{9} = 0.767$$

A4.3 Concrete walls

$i := 0..1$

The walls are subjected to both vertical load and horizontal load from the wind. Hence the capacity needs to be checked with regard to combined compression and bending. A wall can be seen as a column between the windows and a beam above the windows.

A4.3.1 Geometric data

Thickness of load bearing concrete part $t_c := 240\ mm$

Thickness of insulation $t_{ins} := 200\ mm$

Thickness of concrete part $t_{c2} := 50\ mm$

Area of load bearing concrete part $A_{c_i} := t_c \cdot L_{wall_i} = \begin{bmatrix} 0.192 \\ 0.24 \end{bmatrix} m^2$

A4.3.2 Self weight of concrete walls

Density of concrete $\rho_c := 25\ \frac{kN}{m^3}$

Density of insulation according to www.rockwool.se $\rho_{ins} := 1.5\ \frac{kN}{m^3}$

A4.3.2.1 Beam parts of the wall

The weight from both the beam above the window and the one below

$$g_{c.beam} := \rho_c \cdot (h_{wall} - h_{window}) \cdot (t_c + t_{c2}) + \rho_{ins} \cdot (h_{wall} - h_{window}) \cdot t_{ins} = 14.345\ \frac{kN}{m}$$

A4.3.2.2 Column parts of the wall

Self weight of "columns" part of the wall

$$g_{c.col} := \rho_c \cdot L_{wall} \cdot h_{wall} \cdot (t_c + t_{c2}) + \rho_{ins} \cdot L_{wall} \cdot h_{wall} \cdot t_{ins} = \begin{bmatrix} 21.744 \\ 27.18 \end{bmatrix} kN$$

Self weight of "beam" part of the wall

$$g_{c.beam2} := 2\ g_{c.beam} \cdot \frac{w_{window}}{2} = 22.952\ kN$$

A4.3.3 Load combinations

Load combinations has been performed in accordance to Eurocode 0.

A4.3.3.1 For beam parts

6.10a $Q_{beam.a} := 1.35 \cdot (g_{c.beam} + g_p) + 1.5 \cdot 0.7 \cdot q_{imp}$

6.10b $Q_{beam.b} := 1.35 \cdot 0.89 \cdot (g_{c.beam} + g_p) + 1.5 \cdot q_{imp}$

$Q_{beam} := \max(Q_{beam.a}, Q_{beam.b}) = 62.258\ \frac{kN}{m}$

A4.3.3.2 Wind loads

Wind pressures (From xx) $w_1 := 1520\ Pa$

$w_2 := 1355\ Pa$

At first floor $H_1 := w_2 \cdot (w_{window} + L_{wall}) = \begin{bmatrix} 3.252 \\ 3.523 \end{bmatrix} \frac{kN}{m}$

At sixth floor $H_6 := H_1 = \begin{bmatrix} 3.252 \\ 3.523 \end{bmatrix} \frac{kN}{m}$

At elenvth floor $H_{11} := w_1 \cdot (w_{window} + L_{wall}) = \begin{bmatrix} 3.648 \\ 3.952 \end{bmatrix} \frac{kN}{m}$

A4.3.3.3 For column parts

6.10a

Permanent loads $G_{col.a} := 1.35\ (g_p \cdot (w_{window} + L_{wall}) + g_{c.col} + 2\ g_{c.beam2}) = \begin{bmatrix} 139.925 \\ 151.313 \end{bmatrix} kN$

Imposed loads $Q_{col.a} := 1.5 \cdot 0.7\ (w_{window} + L_{wall}) \cdot q_{imp} = \begin{bmatrix} 45.36 \\ 49.14 \end{bmatrix} kN$

Wind loads on different floors $H_{col.11.a} := 1.5 \cdot 0.6\ H_{11} = \begin{bmatrix} 3.283 \\ 3.557 \end{bmatrix} \frac{kN}{m}$

$H_{col.6.a} := 1.5 \cdot 0.6\ H_6 = \begin{bmatrix} 2.927 \\ 3.171 \end{bmatrix} \frac{kN}{m}$

$H_{col.1.a} := 1.5 \cdot 0.6\ H_1 = \begin{bmatrix} 2.927 \\ 3.171 \end{bmatrix} \frac{kN}{m}$

6.10b

Permanent loads $G_{col.b} := 1.35 \cdot 0.89\ (g_p \cdot (w_{window} + L_{wall}) + g_{c.col} + 2\ g_{c.beam2}) = \begin{bmatrix} 124.533 \\ 134.669 \end{bmatrix} kN$

<u>Imposed load as leading variable load</u>

Imposed loads $Q_{col.b} := 1.5\ (w_{window} + L_{wall}) \cdot q_{imp} = \begin{bmatrix} 64.8 \\ 70.2 \end{bmatrix} kN$

Wind loads the same as in 6.10a

Wind load as leading variable load

Imposed load the same as in 6.10a

Wind loads on different floors $H_{col.11.b} := 1.5\ H_{11} = \begin{bmatrix} 5.472 \\ 5.928 \end{bmatrix} \frac{kN}{m}$

$$H_{col.6.b} := 1.5\ H_6 = \begin{bmatrix} 4.878 \\ 5.285 \end{bmatrix} \frac{kN}{m}$$

$$H_{col.1.b} := 1.5\ H_1 = \begin{bmatrix} 4.878 \\ 5.285 \end{bmatrix} \frac{kN}{m}$$

A4.3.4 Acting loads

Here, the choice of load combination is done by giving, G, Q and H values from above. And then the calculations are done. These loads should in other words be changed so every load combination is checked. They should also be multiplied with 14, 9 or 4 if the column on floor level 1, 6 or 11 is to be checked respectively.

Design value for permanent load $G := 14 \cdot G_{col.b} = \begin{bmatrix} 1.743 \cdot 10^3 \\ 1.885 \cdot 10^3 \end{bmatrix} kN$

Design value for imposed load $Q := 14 \cdot Q_{col.b} = \begin{bmatrix} 907.2 \\ 982.8 \end{bmatrix} kN$

Design value for wind load $H := H_{col.1.a} = \begin{bmatrix} 2.927 \\ 3.171 \end{bmatrix} \frac{kN}{m}$

Applied axial load $N_{Ed} := G + Q = \begin{bmatrix} 2.651 \cdot 10^3 \\ 2.868 \cdot 10^3 \end{bmatrix} kN$

A4.3.5 Material data

Partial factors, concerete and reinforcing steel $\gamma_c := 1.5$ $\gamma_s := 1.15$

$\gamma_{cE} := 1.2$

A4.3.5.1 Concrete N 35/45

Concrete strength $f_{ck} := 35\ MPa$ $f_{cd} := \frac{f_{ck}}{\gamma_c} = 23.333\ MPa$

$f_{cm} := 43\ MPa$

Elastic modulus $E_{cm} := 34\ GPa$ $E_{cd} := \frac{E_{cm}}{\gamma_{cE}} = 28.333\ GPa$

A4.3.5.2 Reinforcement B500B

Reinforcement strength $f_{yk} := 500\ MPa$ $f_{yd} := \frac{f_{yk}}{\gamma_s} = 434.783\ MPa$

Elastic modulus $E_s := 200\ GPa$

Strain limit for reinforcement $\varepsilon_{yd} := \frac{f_{yd}}{E_s} = 0.002$

Assume 12 mm bars $\phi := 12\ mm$

Area of one bar $A_{si} := \pi \cdot \left(\frac{\phi}{2}\right)^2 = 113.097\ mm^2$

A4.3.5.3 Creep coefficient

Final creep coefficient $\boxed{\varphi_{ef} := \varphi_{RH} \cdot \beta(f_{cm}) \cdot \beta(t_0)}$ eq. B.1 in Appendix B

Indoor environment, assumed $RH := 50\%$

Notional size $h_{0_i} := \frac{2\ t_c \cdot L_{wall_i}}{2\left(t_c + L_{wall_i}\right)} = \begin{bmatrix} 0.185 \\ 0.194 \end{bmatrix} m$

Factor to allow fo the effect of relative humidity

$f_{cm} > 35\ MPa = 1$

$$\varphi_{RH_i} := \left(1 + \frac{1 - RH}{0.1 \cdot \sqrt[3]{h_{0_i} \cdot \frac{1000}{m}}} \cdot \left(\frac{35\ MPa}{f_{cm}}\right)^{0.7}\right) \cdot \left(\frac{35\ MPa}{f_{cm}}\right)^{0.2}$$

$$\varphi_{RH} = \begin{bmatrix} 1.689 \\ 1.678 \end{bmatrix}$$

Factor to allow fo the effect of concrete strength $\beta_{fcm} := 2.56$

Factor to allow fo the effect of concrete age at loading $\beta_{t0} := \frac{1}{0.1 + 28^{0.2}} = 0.488$

Creep coefficient $\varphi := \varphi_{RH} \cdot \beta_{fcm} \cdot \beta_{t0} = \begin{bmatrix} 2.112 \\ 2.098 \end{bmatrix}$

A4.3.6 Column part of wall

A4.3.6.1 First order moment, with regard to unintended imperfections

Unintended inclinations

Height of the wall is the sam as for columns, only one wall is considered, therefore the same inclination as for columns.

Additional first order eccentricity due to unintended inclination

$$e_i := \frac{h_{wall}}{400} = 0.009\ m$$

Intended eccentricity

Intended first order moment due to all intended actions (wind)

$$M_0 := \frac{H \cdot h_{wall}^{\ 2}}{8} = \begin{bmatrix} 4.741 \\ 5.137 \end{bmatrix} kN \cdot m$$

Intended initial eccentricity

$$e_0 := \frac{M_0}{N_{Ed}} = \begin{bmatrix} 0.002 \\ 0.002 \end{bmatrix} m$$

First order moment

$$M_{0Ed_i} := N_{Ed_i} \cdot \left(e_{0_i} + e_i\right) = \begin{bmatrix} 28.597 \\ 30.95 \end{bmatrix} kN \cdot m$$

A4.3.6.2 Second order moment

If the column is regarded as slender the second order moment should be considered.

Slenderness

Second moment of inertia

$$I_{c_i} := \frac{L_{wall_i} \cdot t_c^{\ 3}}{12} = \begin{bmatrix} 9.216 \cdot 10^{-4} \\ 0.001 \end{bmatrix} m^4$$

$$i_{c_i} := \sqrt{\frac{I_{c_i}}{A_{c_i}}} = \begin{bmatrix} 0.069 \\ 0.069 \end{bmatrix} m$$

Slenderness

$$\lambda_c := \frac{h_{wall}}{i_c} = \begin{bmatrix} 51.962 \\ 51.962 \end{bmatrix}$$

Relative normal force

$$n_c := \frac{N_{Ed}}{f_{cd} \cdot A_c} = \begin{bmatrix} 0.592 \\ 0.512 \end{bmatrix}$$

Slenderness limit

$$\lambda_{c.lim} := \frac{10.8}{\sqrt{n_c}} = \begin{bmatrix} 14.041 \\ 15.091 \end{bmatrix}$$

Column regarded as slender:

$$\lambda_c > \lambda_{c.lim} = \begin{bmatrix} 1 \\ 1 \end{bmatrix}$$

1 = yes
0 = no

Second order effect

Design moment, including second order effects

$$\boxed{M_{Ed} := \left(1 + \frac{\beta_m}{\frac{N_B}{N_{Ed}} - 1}\right) \cdot M_{0.Ed}}$$

eq. 8.28 in section 5.8

Approximate value of nominal stiffness

$$EI_i := \frac{0.3}{1 + 0.5\ \varphi_i} \cdot E_{cd} \cdot I_{c_i} = \begin{bmatrix} 3.81 \cdot 10^3 \\ 4.779 \cdot 10^3 \end{bmatrix} \boldsymbol{kN \cdot m^2}$$

Critical load

$$N_{B_i} := \frac{\pi^2 \cdot EI_i}{h_{wall}^{\ 2}} = \begin{bmatrix} 2.901 \cdot 10^3 \\ 3.639 \cdot 10^3 \end{bmatrix} \boldsymbol{kN}$$

Applied load

$$N_{Ed} = \begin{bmatrix} 2.651 \cdot 10^3 \\ 2.868 \cdot 10^3 \end{bmatrix} \boldsymbol{kN}$$

Factor denpending on 1st and 2nd order distribution

$$\beta_m := 1.03$$

Deisgn moment, first and second order moment

$$M_{Ed_i} := \left(1 + \frac{\beta_m}{\dfrac{N_{B_i}}{N_{Ed_i}} - 1} \right) \cdot M_{0Ed_i} = \begin{bmatrix} 340.048 \\ 149.519 \end{bmatrix} \boldsymbol{kN \cdot m}$$

A4.3.6.3 Sectional analysis in ULS

$$\alpha := 0.81$$

$$\beta := 0.416$$

Maximum concrete strain

$$\varepsilon_{cu} := 3.5 \cdot 10^{-3}$$

Maximum steel strain

$$\varepsilon_{yd} = 2.174\ 10^{-3}$$

Spacing between bars

$$s := 0.2\ \boldsymbol{m}$$

Number of bars in each layer

$$n_i := \frac{L_{wall_i}}{s} = \begin{bmatrix} 4 \\ 5 \end{bmatrix}$$

Assumed that all reinforcement is yielding and using 2n bars in total.

$$\boxed{\alpha \cdot f_{cd} \cdot b \cdot x = \blacksquare \cdot N_{Ed}}$$

$$x_i := \frac{N_{Ed_i}}{\alpha \cdot f_{cd} \cdot L_{wall_i}} = \begin{bmatrix} 0.175 \\ 0.152 \end{bmatrix} \boldsymbol{m} \qquad x < t_c = \begin{bmatrix} 1 \\ 1 \end{bmatrix} \qquad \text{1 = OK}$$

$$d := t_c - 0.05\ \boldsymbol{m} = 0.19\ \boldsymbol{m} \qquad d' := 0.05\ \boldsymbol{m}$$

$$\varepsilon'_{s_i} := \frac{x_i - d'}{x_i} \cdot \varepsilon_{cu} = \begin{bmatrix} 2.502 \\ 2.347 \end{bmatrix} 10^{-3} \qquad > \varepsilon_{yd} \qquad \text{OK!}$$

$$\varepsilon_{s_i} := \frac{d - x_i}{x_i} \cdot \varepsilon_{cu} = \begin{bmatrix} 0.293 \\ 0.882 \end{bmatrix} 10^{-3} \qquad < \varepsilon_{yd} \qquad \text{NOT OK!}$$

Assumed that As' is yielding and As is not, 2n bars in total.

$$x_1 := x_0 = 0.175 \ \boldsymbol{m}$$

$$x_1 := \mathbf{root}\left(\alpha \cdot f_{cd} \cdot L_{wall_0} \cdot x_1 - E_s \cdot \frac{d - x_1}{x_1} \cdot \varepsilon_{cu} \cdot n_0 \cdot A_{si} + f_{yd} \cdot n_0 \cdot A_{si} - N_{Ed_0}, x_1\right)$$

$$x_1 = 0.165 \ \boldsymbol{m}$$

$$x_2 := x_1 = 0.152 \ \boldsymbol{m}$$

$$x_2 := \mathbf{root}\left(\alpha \cdot f_{cd} \cdot L_{wall_1} \cdot x_2 - E_s \cdot \frac{d - x_2}{x_2} \cdot \varepsilon_{cu} \cdot n_1 \cdot A_{si} + f_{yd} \cdot n_1 \cdot A_{si} - N_{Ed_1}, x_2\right)$$

$$x_2 = 0.145 \ \boldsymbol{m}$$

New x:

$$\varepsilon_{yd} = 2.174 \ 10^{-3}$$

$$x := \begin{bmatrix} x_1 \\ x_2 \end{bmatrix} = \begin{bmatrix} 0.165 \\ 0.145 \end{bmatrix} \boldsymbol{m}$$

$$\varepsilon'_{s_i} := \frac{x_i - d'}{x_i} \cdot \varepsilon_{cu} = \begin{bmatrix} 2.442 \\ 2.295 \end{bmatrix} 10^{-3} \qquad > \varepsilon_{yd} \qquad \text{OK! yielding}$$

$$\varepsilon_{s_i} := \frac{d - x_i}{x_i} \cdot \varepsilon_{cu} = \begin{bmatrix} 0.52 \\ 1.08 \end{bmatrix} 10^{-3} \qquad < \varepsilon_{yd} \qquad \text{OK! not yielding}$$

<u>Moment capacity</u>

$$M_{Rd_i} := \alpha \cdot f_{cd} \cdot L_{wall_i} \cdot x_i \cdot \left(d - \beta \cdot x_i\right) + E_s \cdot \varepsilon'_{s_i} \cdot n_i \cdot A_{si} \cdot (d - d') - N_{Ed_i} \cdot \left(d - \frac{L_{wall_i}}{2}\right) = \begin{bmatrix} 890.669 \\ 1.281 \cdot 10^{3} \end{bmatrix} \boldsymbol{kN} \cdot \boldsymbol{m}$$

The first capacity value is for a wall with 0.8 metre column with a thickness off 240 mm and influence length of 6 metre. The other value is for a column of 1 metre, thickness 240 mm. The capacity of the second wall-column is very large compared to the applied moment, this is because the thickness should be 225 mm. If the thickness is changed the utilisation ratio would have a better value. This is valid for the calculation of the wall on the first floor.

A4.3.7 Beam part of wall

Assuming simply supported to obtain worst case

$$M_{beam} := \frac{Q_{beam} \cdot w_{window}^{2}}{8} = 19.923\ kN \cdot m$$

A4.3.7.1 Due to deep beam --> strut and tie model

Total height of the beam $h_{beam} = 0.95\ m$

Check of geometry (assuming 60 degree inclination of the strut) $z := \frac{w_{window}}{4} \cdot \tan(60\ deg) = 0.693\ m$ less than total height, ok!

Concrete cover $a_c := 50\ mm$

Support reactions $R := Q_{beam} \cdot \frac{w_{window}}{2} = 49.806\ kN$

Forces acting in the first node (one strut and one tie) $T := \frac{R}{\tan(60\ deg)} = 28.756\ kN$ $C_1 := \frac{R}{\sin(60\ deg)} = 57.511\ kN$

Forces acting in the second node (three struts, one the same as the first node) $C_2 := R = 49.806\ kN$ $C_3 := \frac{R}{\tan(60\ deg)} = 28.756\ kN$ = T OK!

Design of tensile reinforcement

Needed amount of reinforcement area $A_s := \frac{T}{f_{yd}} = 66.138\ mm^2$

Assuming bars with a diameter of 8 mm, and their cross-sectional area $\phi_{long} := 8\ mm$ $A_{si} := \pi \cdot \left(\frac{\phi_{long}}{2}\right)^2 = 50.265\ mm^2$

Amount of needed bars $\frac{A_s}{A_{si}} = 1.316$ $n_{long} := 2$

This amount of reinforcement bars is possible to be covered by the concrete, OK!

Check nodes

Height of tensile zone $u := 2\ a_c = 0.1\ m$

Assumed support length and width of compression zone $a_1 := 150\ mm$ $a_2 := a_1 \cdot \sin(60\ deg) + u \cdot \cos(60\ deg) = 0.18\ m$

Maximum stress $\sigma_{Rd.max} := 0.85\left(\left(1 - \frac{35}{250}\right) \cdot f_{cd}\right) = 17.057\ MPa$

Stress in the support $\sigma_{c1} := \dfrac{R}{t_c \cdot a_1} = 1.384\ \mathbf{MPa}$ $<<$ $\sigma_{Rd.max}$ $OK!$

Stress in the compressive strut $\sigma_{c2} := \dfrac{C_1}{t_c \cdot a_2} = 1.332\ \mathbf{MPa}$

Utilisation $u_{r1} := \dfrac{\sigma_{c1}}{\sigma_{Rd.max}} = 0.081$

$$u_{r2} := \frac{\sigma_{c2}}{\sigma_{Rd.max}} = 0.078$$

Thickness of the beam is sufficient, and the compression is small compared to the capacity.

Shear capacity 6.2.2 in EN 1992-1-1

Shear resistance VRd, without shear reinforcement

$$V_{Rd.c} := \left(\boxed{C_{Rd.c}} \cdot k \cdot \left(100\ \rho_l \cdot f_{ck}\right)^{\frac{1}{3}} + k_1 \cdot \sigma_{cp} \right) b_w \cdot d$$

Constants for the shear capacity calculation

$$C_{Rd.c} := \frac{0.18}{\gamma_c} = 0.12 \qquad v_{min} := 0.035 \cdot 0.15^{\frac{3}{2}} \cdot \left(\frac{f_{ck}}{\mathbf{MPa}} \right)^{\frac{1}{2}} \cdot 1\ \mathbf{MPa} = 0.012\ \mathbf{MPa}$$

$$k := 1 + \sqrt{\frac{200\ \mathbf{mm}}{h_{beam} - a_c}} = 1.471 \qquad \rho_l := \frac{2 \cdot A_{si}}{t_c \cdot \left(h_{beam} - a_c\right)} = 4.654 \cdot 10^{-4}$$

$$\sigma_{cp} := 0\ \mathbf{MPa} \qquad k_1 := 0.15\ \mathbf{mm}$$

Shear capacity $V_{Rd.c} := C_{Rd.c} \cdot k \cdot \left(100 \cdot \rho_l \cdot \dfrac{f_{ck}}{\mathbf{MPa}}\right)^{\frac{1}{3}} \cdot t_c \cdot \left(h_{beam} - a_c\right) \cdot 1\ \mathbf{MPa} = 44.875\ \mathbf{kN}$

Minimum shear capacity $V_{Rd.c.min} := \left(v_{min}\right) \cdot t_c \cdot \left(h_{beam} - a_c\right) = 2.598\ \mathbf{kN}$

Shear capacity with shear reinforcement according to EN1992-1-1 section 6.2.3

Assumed spacing between shear reinforcement $s_{ver} := 400\ \mathbf{mm}$

Minimum shear reinforcement $A_{sw.min} := 0.002 \cdot \left(h_{beam} \cdot t_c\right) = 456\ \mathbf{mm}^2$

Assumed diameter of the bars and their area $\phi_{ver} := 8\ \mathbf{mm}$ $A_{si.ver} := \left(\dfrac{\phi_{ver}}{2}\right)^2 \cdot \pi = 50.265\ \mathbf{mm}^2$

Number of bars $\frac{A_{sw.min}}{A_{si.ver}} = 9.072$

Choosen inclination of cracks $\theta := 40\ \mathbf{deg}$ $\cot(\theta) = 1.192$ $OK!$

Shear capacity with reinforcement $V_{Rd.s} := \frac{4\ A_{si.ver}}{s} \cdot 0.9 \cdot \left(h_{beam} - a_c\right) \cdot f_{yd} \cdot \cot(\theta) = 421.933\ \mathbf{kN}$

Maximum value of shear capacity that can be acounted for $V_{Rd.max} := t_c \cdot 0.9 \cdot \left(h_{beam} - a_c\right) \cdot 0.6 \cdot \frac{f_{cd}}{\cot(\theta) + \tan(\theta)} = \left(1.34 \cdot 10^3\right)\ \mathbf{kN}$

The shear capacity is the smallest of the two above values, but both is greater than the applied shear force. It is also concluded that the thickness of the concrete beam is enough to cover both the shear reinforcement and the bendeing reinforcement with a sufficient concrete cover.

A4.4 CLT-walls

Column subjected to combined compression and bending

$$\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + \frac{\sigma_{m.y.d}}{f_{m.y.d}} \le 1$$

A4.4.1 Geometrical data for CLT

The values calculated by hand for different wall sections.

$t_{wall.1} := 221\ mm$ $I_{y.1.1} := 3.895 \cdot 10^{-4}\ m^4$ $I_{y.2.1} := 4.868 \cdot 10^{-4}\ m^4$

$t_{wall.2} := 259\ mm$ $I_{y.1.2} := 8.286 \cdot 10^{-4}\ m^4$ $I_{y.2.2} := 0.001\ m^4$

$t_{wall.3} := 208\ mm$ $I_{y.1.3} := 3.035 \cdot 10^{-4}\ m^4$ $I_{y.2.3} := 3.793 \cdot 10^{-4}\ m^4$

$t_{wall.4} := 183\ mm$ $I_{y.1.4} := 3.274 \cdot 10^{-4}\ m^4$ $I_{y.2.4} := 4.092 \cdot 10^{-4}\ m^4$

$t_{wall.5} := 120\ mm$ $I_{y.1.5} := 7.89 \cdot 10^{-5}\ m^4$ $I_{y.2.5} := 9.862 \cdot 10^{-5}\ m^4$

$t_{wall.6} := 310\ mm$ $I_{y.1.6} := 0.001323\ m^4$ $I_{y.2.6} := 0.002\ m^4$

$t_{wall.7} := 158\ mm$ $I_{y.1.7} := 2.267 \cdot 10^{-4}\ m^4$ $I_{y.2.7} := 2.833 \cdot 10^{-4}\ m^4$

$S_{beam.1} := 0.014\ m^3$ $I_{y.beam.1} := 0.009\ m^4$

$S_{beam.2} := 0.014\ m^3$ $I_{y.beam.2} := 0.009\ m^4$

$S_{beam.3} := 0.015\ m^3$ $I_{y.beam.3} := 0.009\ m^4$

$S_{beam.4} := 0.01\ m^3$ $I_{y.beam.4} := 0.007\ m^4$

$S_{beam.5} := 0.007\ m^3$ $I_{y.beam.5} := 0.005\ m^4$

$S_{beam.6} := 0.018\ m^3$ $I_{y.beam.6} := 0.011\ m^4$

$S_{beam.7} := 0.007\ m^3$ $I_{y.beam.7} := 0.005\ m^4$

För Lwall = 1m

$t_{fire} := 200.5\ mm$

$I_{fire} := 4.028 \cdot 10^{-4}\ m^4$

Height of wall | $h_{wall} = 3.6\ m$

Width of column $L_{wall} := 0.8\ m$ (Distance between windows)

These values are changed depending on which wall that was to be checked, values from above were used.

$t_{CLT} := t_{wall.6}$

$I_{y.CLT} := I_{y.1.6}$ 1 if 0.8 and 2 if 1

$S_{beam} := S_{beam.6}$

$I_{y.beam} := I_{y.beam.6}$

Cross-section area of column $A_{CLT} := t_{CLT} \cdot L_{wall} = 0.248\ m^2$

Thickness of insulation $t_{ins} := 200\ mm$

Thickness of non load bearing CLT part $t_{CLT.2} := 50\ mm$

A4.4.2 Material data

http://www.martinsons.se/kl-tra-projektera/konstruktionsfakta

Compression parallel to grain $f_{c.0.k.CLT} := 21\ MPa$

Bending parallel to grainl $f_{m.k.CLT} := 24\ MPa$

Shear strength $f_{v.k.CLT} := 4\ MPa$

Elastic modulus
Capacity analysis $E_{0.05.CLT} := 7400\ MPa$

Deformation analysis $E_{0.mean} := 11000\ MPa$

Partial factor $\gamma_{M.CLT} := 1.3$

Strength modification factor $k_{mod.CLT} := 0.7$ Assuming long term load and sevice class 2

Deformation modification factor $k_{def.CLT} := 0.8$

Effect of member size
y-dir

$$k_{h.CLT} := \begin{cases} \text{if } t_{CLT} \le 150\ mm \\ \quad min\left(\left(\dfrac{150\ mm}{t_{CLT}}\right)^{0.2}, 1.3\right) \\ \text{else} \\ \quad 1 \end{cases}$$

Density of CLT $\rho_{CLT} := 4\ \frac{kN}{m^3}$

Density of insulation $\rho_{ins} := 1.5\ \frac{kN}{m^3}$

A4.4.2.1 Design strength values

Compression parallel to grain $f_{c.0.d.CLT} := k_{mod.CLT} \cdot k_{h.CLT} \cdot \frac{f_{c.0.k.CLT}}{\gamma_{M.CLT}} = 11.308\ MPa$

Bending parallel to grain $f_{m.y.d.CLT} := k_{mod.CLT} \cdot k_{h.CLT} \cdot \frac{f_{m.k.CLT}}{\gamma_{M.CLT}} = 12.923\ MPa$

Shear strenght $f_{v.d.CLT} := k_{mod.CLT} \cdot \frac{f_{v.k.CLT}}{\gamma_{M.CLT}} = 2.154\ MPa$

A4.4.2.2 Design strength values for fire

$k_{fi} := 1.25$

$k_{mod.fi} := 1$

$\gamma_{M.fi} := 1$

Compression parallel to grain $f_{c.0.d.fire} := k_{mod.fi} \cdot k_{fi} \cdot \frac{f_{c.0.k.CLT}}{\gamma_{M.fi}} = 26.25\ MPa$

Bending parallel to grain $f_{m.y.d.fire} := k_{mod.fi} \cdot k_{fi} \cdot \frac{f_{m.y.d.CLT}}{\gamma_{M.fi}} = 16.154\ MPa$

A4.4.2.3 Reduction factor for the strength

$i_{y.CLT} := \sqrt{\frac{I_{y.CLT}}{A_{CLT}}} = 0.073\ m$

Slenderness $\lambda_{y.CLT} := \frac{h_{wall}}{i_{y.CLT}}$

Relative slenderness $\lambda_{y.rel.CLT} := \frac{\lambda_{y.CLT}}{\pi} \cdot \sqrt{\frac{f_{c.0.k.CLT}}{E_{0.05.CLT}}} = 0.836$

$\beta_{c.CLT} := 0.2$

$$k_{y.CLT} := 0.5 \cdot \left(1 + \beta_{c.CLT} \cdot \left(\lambda_{y.rel.CLT} - 0.3\right) + \left(\lambda_{y.rel.CLT}\right)^2\right)$$

$$k_{y.c.CLT} := \begin{array}{|l} \text{if } \lambda_{y.rel.CLT} \le 0.3 \\ \quad \| 1 \\ \text{else} \\ \quad \left\| \dfrac{1}{k_{y.CLT} + \sqrt{k_{y.CLT}^{\ 2} - \lambda_{y.rel.CLT}^{\ 2}}} \right. \end{array}$$

A4.4.3 Load combinations

A4.4.3.1 Self-weight of Beam part of the wall

The weight from both the beam above the window and the one below

$$g_{beam.CLT} := \rho_{CLT} \cdot \left(h_{wall} - h_{window}\right) \cdot \left(t_{CLT} + t_{CLT.2}\right) + \rho_{ins} \cdot \left(h_{wall} - h_{window}\right) \cdot t_{ins} = 3.306 \ \frac{kN}{m}$$

A4.4.3.2 Self-weight of Column part of the wall

$$g_{.col.CLT} := \rho_{CLT} \cdot L_{wall} \cdot h_{wall} \cdot \left(t_{CLT} + t_{CLT.2}\right) + \rho_{ins} \cdot L_{wall} \cdot h_{wall} \cdot t_{ins} = 5.011 \ kN$$

Self weight from "beam" part of the wall

$$g_{beam.CLT.2} := 2 \cdot g_{beam.CLT} \cdot \frac{w_{window}}{2} = 5.29 \ kN$$

A4.4.3.3 Load combination for beam parts

6.10a $$Q_{beam.CLT.a} := 1.35 \cdot \left(g_{beam.CLT} + g_p\right) + 1.5 \cdot 0.7 \cdot q_{imp}$$

6.10b $$Q_{beam.CLT.b} := 1.35 \cdot 0.89 \cdot \left(g_{beam.CLT} + g_p\right) + 1.5 \cdot q_{imp}$$

$$Q_{beam.CLT} := \max\left(Q_{beam.CLT.a}, Q_{beam.CLT.b}\right) = 48.995 \ \frac{kN}{m}$$

A4.4.3.4 Load combination for beam parts for fire load

6.11 $$Q_{beam.CLT.fire} := g_{beam.CLT} + g_p + 0.5 \cdot q_{imp} = 27.306 \ \frac{kN}{m}$$

A4.4.3.5 Load combination for column parts

Wind loads

Wind pressures (From Appendix C1)

$w_1 := 1520\ Pa$

$w_2 := 1355\ Pa$

At first floor $H_1 := w_2 \cdot (w_{window} + L_{wall}) = 3.252\ \frac{kN}{m}$

At sixth floor $H_6 := H_1 = 3.252\ \frac{kN}{m}$

At eleventh floor $H_{11} := w_1 \cdot (w_{window} + L_{wall}) = 3.648\ \frac{kN}{m}$

Load combination

6.10a - Permanent load as main load

Permanent load $G_{col.CLT.a} := 1.35\ (g_p \cdot (w_{window} + L_{wall}) + g_{.col.CLT} + 2\ g_{beam.CLT.2}) = 69.647\ kN$

Variable load (imposed) $Q_{col.a} := 1.5 \cdot 0.7\ (w_{window} + L_{wall}) \cdot q_{imp} = 45.36\ kN$

Variable load (wind) $H_{col.11.a} := 1.5 \cdot 0.6\ H_{11} = 3.283\ \frac{kN}{m}$

$H_{col.6.a} := 1.5 \cdot 0.6\ H_6 = 2.927\ \frac{kN}{m}$

$H_{col.1.a} := 1.5 \cdot 0.6\ H_1 = 2.927\ \frac{kN}{m}$

6.10b - Variable load as main load

Permanent load $G_{col.CLT.b} := 1.35 \cdot 0.89\ (g_p \cdot (w_{window} + L_{wall}) + g_{.col.CLT} + 2\ g_{beam.CLT.2}) = 61.986\ kN$

Variable load (imposed) $Q_{col.b} := 1.5\ (w_{window} + L_{wall}) \cdot q_{imp} = 64.8\ kN$

Wind loads are the same as in load combination 6.10a

Wind load as leading variable load

Imposed load the same as in 6.10a

Variable load (wind) $H_{col.11.b} := 1.5\ H_{11} = 5.472\ \frac{kN}{m}$

$H_{col.6.b} := 1.5\ H_6 = 4.878\ \frac{kN}{m}$

$$H_{col.1.b} := 1.5\ H_1 = 4.878\ \frac{kN}{m}$$

A4.4.3.6 Load combination for column parts for fire loads

Permanent load

$$G_{col.fire} := \left(g_p \cdot \left(w_{window} + L_{wall}\right) + g_{col.CLT} + 2\ g_{beam.CLT.2}\right) = 51.59\ kN$$

Variable load (imposed)

$$Q_{col.fire} := \left(w_{window} + L_{wall}\right) \cdot q_{imp} = 43.2\ kN$$

Variable load (wind)
(Combinated with the imposed load having the imposed load as main load)

$$H_{col.11.fire} := 0.2\ H_{11} = 0.73\ \frac{kN}{m}$$

$$H_{col.6.fire} := 0.2\ H_6 = 0.65\ \frac{kN}{m}$$

$$H_{col.1.fire} := 0.2\ H_1 = 0.65\ \frac{kN}{m}$$

A4.4.4 Check of capacity of the CLT-wall

The area between the window can be seen as a column that is subjected to combined compression and bending.

$$\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + \frac{\sigma_{m.y.d}}{f_{m.y.d}} \le 1$$

<u>Permanent as main load</u>

Choose the correct wind load for the current case

$$\sigma_{m.y.d.11.a} := \frac{\dfrac{H_{col.1.a} \cdot h_{wall}^{\ 2}}{8}}{\dfrac{L_{wall} \cdot t_{CLT}^{\ 2}}{6}} = 0.37\ MPa$$

Choose the correct number of storeys for the current case

$$\sigma_{c.0.d.11.a} := \frac{14 \cdot \left(G_{col.CLT.a} + Q_{col.a}\right)}{A_{CLT}} = 6.492\ MPa$$

$$\frac{\sigma_{c.0.d.11.a}}{k_{y.c.CLT} \cdot f_{c.0.d.CLT}} + \frac{\sigma_{m.y.d.11.a}}{f_{m.y.d.CLT}} = 0.743$$

Wind load as main load

Choose the correct wind load for the current case

$$\sigma_{m.y.d.11.b1} := \frac{\frac{H_{col.1.b} \cdot h_{wall}^{2}}{8}}{\frac{L_{wall} \cdot t_{CLT}^{2}}{6}} = 0.617 \ \mathbf{MPa}$$

Choose the correct number of storeys for the current case

$$\sigma_{c.0.d.11.b1} := \frac{14 \cdot \left(G_{col.CLT.b} + Q_{col.a}\right)}{A_{CLT}} = 6.06 \ \mathbf{MPa}$$

$$\frac{\sigma_{c.0.d.11.b1}}{k_{y.c.CLT} \cdot f_{c.0.d.CLT}} + \frac{\sigma_{m.y.d.11.b1}}{f_{m.y.d.CLT}} = 0.715$$

Imposed load as main load

$$\sigma_{m.y.d.11.b2} := \frac{\frac{H_{col.1.a} \cdot h_{wall}^{2}}{8}}{\frac{L_{wall} \cdot t_{CLT}^{2}}{6}} = 0.37 \ \mathbf{MPa}$$

$$\sigma_{c.0.d.11.b2} := \frac{14 \cdot \left(G_{col.CLT.b} + Q_{col.b}\right)}{A_{CLT}} = 7.157 \ \mathbf{MPa}$$

$$\frac{\sigma_{c.0.d.11.b2}}{k_{y.c.CLT} \cdot f_{c.0.d.CLT}} + \frac{\sigma_{m.y.d.11.b2}}{f_{m.y.d.CLT}} = 0.816$$

A4.4.4.1 Check of column with regard to fire

$$\sigma_{m.y.d.fire} := \frac{\frac{H_{col.1.fire} \cdot h_{wall}^{2}}{8}}{\frac{L_{wall} \cdot t_{CLT}^{2}}{6}} = 0.082 \ \mathbf{MPa}$$

$$\sigma_{c.0.d.fire} := \frac{14 \cdot \left(G_{col.fire} + Q_{col.fire}\right)}{A_{CLT}} = 5.351 \ \mathbf{MPa}$$

$$\frac{\sigma_{c.0.d.fire}}{k_{y.c.CLT} \cdot f_{c.0.d.fire}} + \frac{\sigma_{m.y.d.fire}}{f_{m.y.d.fire}} = 0.259$$

A4.4.5 Moment capacity of beam part

Largest bending moment can be found in the middle of the span.

$$M_{Ed.CLT} := \frac{Q_{beam.CLT} \cdot w_{window}^{2}}{8} = 15.678\ kN \cdot m$$

Section modulus

$$W_{CLT} := \frac{t_{CLT} \cdot h_{beam}^{2}}{6} = 0.047\ m^{3}$$

Moment capacity

$$M_{Rd.CLT} := f_{m.y.d.CLT} \cdot W_{CLT} = 602.592\ kN \cdot m$$

A4.4.6 Shear capacity of beam part

The beam should fulfil the following condition regarding shear

$\tau_d < f_{v.d}$ $\tau_d := \frac{S \cdot \boxed{V_{Ed}}}{I \cdot b_{ef}}$ eq. 6.13 in section 6.1

Effective width

$$k_{cr} := 1.0$$

$$b_{eff} := k_{cr} \cdot t_{CLT} = 0.31\ m$$

Shear force in simply supported beam

$$V_{Ed.CLT} := \frac{Q_{beam.CLT} \cdot w_{window}}{2} = 39.196\ kN$$

$$\tau_d := \frac{S_{beam} \cdot V_{Ed.CLT}}{I_{y.beam} \cdot b_{eff}} = 0.207\ MPa$$

A4.4.7 Deflection

Quasi-permanent load combination has been used.

Load combination factor

$$\psi_2 := 0.3$$

Deflection from the permanent load

$$w_{fin.CLT.G} := \frac{(g_{beam.CLT} + g_p) \cdot 5 \cdot w_{window}^{4}}{384 \cdot E_{0.mean} \cdot I_{y.beam}} \cdot (1 + k_{def.CLT}) = 0.023\ mm$$

Deflection from the imposed load

$$w_{fin.CLT.Q} := \frac{q_{imp} \cdot 5 \cdot w_{window}^{\ 4}}{384 \cdot E_{0.mean} \cdot I_{y.beam}} \cdot \left(1 + \psi_2 \cdot k_{def.CLT}\right) = 0.016\ \boldsymbol{mm}$$

Total deflection

$$w_{fin} := w_{fin.CLT.G} + w_{fin.CLT.Q} = 0.039\ \boldsymbol{mm}$$

Deflection limit

$$w_{limit} := \frac{w_{window}}{400} = 4\ \boldsymbol{mm}$$

A4.4.8 Utilisation for beam part of wall

$$u_{M.CLT} := \frac{M_{Ed.CLT}}{M_{Rd.CLT}} = 0.026$$

$$u_{V.CLT} := \frac{\tau_d}{f_{v.d.CLT}} = 0.096$$

$$u_d := \frac{w_{fin}}{w_{limit}} = 0.01$$

Appendix A5: Bracing units

$kNm := kN \cdot m$

In this Appendix the calculations for the design of the bracing units in Section 5.7 in the component study are presented. The same document was used when designing with regard to fire by changing the load combinations and the material properties for timber.

A5.1 Wind loads on the fictive building

This part have been calculated according to **SS-EN 1991-1-4:2005**. Any references made refers to this code.

A5.1.1 Geometry of the building and description of the terrain

Assuming a 15 storey building with a quadratic cross-section with sides measuring 36 metre. The height of each floor is assumed to be 3.6 metre, resulting in total height of 54 metre.

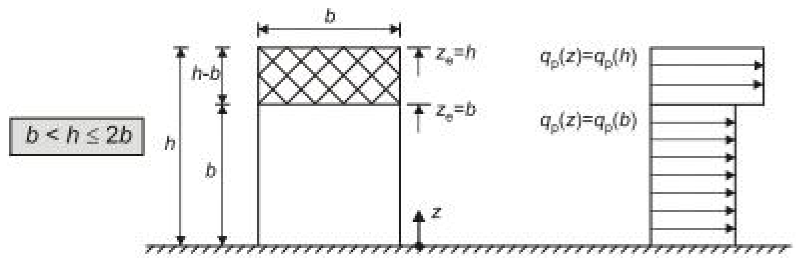
Side length $a := 36m$

Height $h := 54m$

According to **section 7.2.2** the wind load of the house should be divided into different zones. With this height to width ratio two zones is needed, see figure below.

$$b := a = 36m$$

$$b < h < 2 \cdot b = 1$$



Height up to top of zone 1 $z_1 := h = 54m$

Height up to top of zone 2 $z_2 := b = 36m$

Terrain type III is assumed, giving the following minimum and maximum heights of the building.

$z_{min} := 5m$ $z_{min} < z_1 < z_{max}$ Table 4.1 in section 4.3

$z_{max} := 200m$ $z_{min} < z_2 < z_{max}$

A5.1.2 Basic wind velocity

$$\boxed{v_b := c_{dir} \cdot c_{season} \cdot v_{b.0}}$$ eq. 4.1 in section 4.2

Direction factor $c_{dir} := 1$ Assumptions made according to EC1-4, notes in 4.2

Season factor $c_{season} := 1$

Wind velocity in Göteborg $v_{b.0} := 25 \frac{m}{s}$

Basic wind velocity $v_b := c_{dir} \cdot c_{season} \cdot v_{b.0} = 25 \frac{m}{s}$

A5.1.3 Mean wind velocity

$$\boxed{v_m(z) = \blacksquare \cdot c_r(z) \cdot c_0(z) \cdot v_b}$$ eq. 4.3 in section 4.3

Terrain roughness factor, result from assumption of the terrain

$z_0 := 0.3m$

$z_{0.II} := 0.05m$

Table 4.1 in section 4.3

Terrain factor $k_r := 0.19 \cdot \left(\frac{z_0}{z_{0.II}}\right)^{0.07} = 0.215$

Roughness factor for zone 1 $c_{r.1} := k_r \cdot \ln\left(\frac{z_1}{z_0}\right) = 1.119$

Roughness factor for zone 2 $c_{r.2} := k_r \cdot \ln\left(\frac{z_2}{z_0}\right) = 1.031$

Orpograpgy factor $c_0 := 1$

Mean wind velocity zone 1 $v_{m.1} := c_{r.1} \cdot c_0 \cdot v_b = 27.963 \frac{m}{s}$

Mean wind velocity zone 2 $v_{m.2} := c_{r.2} \cdot c_0 \cdot v_b = 25.779 \frac{m}{s}$

A5.1.4 Wind turbulence

Turbulence factor $k_l := 1$

Standard deviation of the turbulence $\sigma_v := k_r \cdot v_b \cdot k_l = 5.385 \frac{m}{s}$

Wind turbulence zone 1 $l_{v.1} := \dfrac{\sigma_v}{v_{m.1}} = 0.193$ <u>eq. 4.7 in section 4.4</u>

Wind turbulence zone 2 $l_{v.2} := \dfrac{\sigma_v}{v_{m.2}} = 0.209$

A5.1.5 Peak velocity pressure

Air density $\rho := 1.25 \dfrac{kg}{m^3}$

Peak velocity is calculated as: $\boxed{q_p(z) = \blacksquare \cdot \left(1 + 7 \cdot I_v(z)\right) \cdot \frac{1}{2} \rho \cdot v_m(z)^2}$ <u>eq. 4.8 in section 4.5</u>

Peak velocity pressure zone 1 $q_{p.1} := \left(1 + 7 \cdot l_{v.1}\right) \cdot 0.5 \rho \cdot v_{m.1}{}^2 = 1.147 \times 10^3 \cdot Pa$

Peak velocity pressure zone 2 $q_{p.2} := \left(1 + 7 \cdot l_{v.2}\right) \cdot 0.5 \rho \cdot v_{m.2}{}^2 = 1.023 \times 10^3 \cdot Pa$

A5.1.6 Wind pressure on surfaces

Wind pressure is calculated as: $\boxed{w := q_p \cdot c_p}$ <u>eq. 5.1 in section 5.2</u>

Length of side parallel to wind direction $d := a = 36 m$

Ratio between hight and side parallel to wind $ratio := \dfrac{h}{d} = 1.5$

Form factors for windward side $C_{pe.10.D} := 0.8$

Form factors for leeward side $C_{pe.10.E} := -0.5 + (-0.7 + 0.5) \cdot \dfrac{(ratio - 1)}{5 - 1} = -0.525$

A5.1.6.1 Total wind pressure

Wind pressure for zone 1 $w_1 := q_{p.1} \cdot \left(C_{pe.10.D} - C_{pe.10.E}\right) = 1.52 \times 10^3 Pa$

Wind pressure for zone 2 $w_2 := q_{p.2} \cdot \left(C_{pe.10.D} - C_{pe.10.E}\right) = 1.355 \times 10^3 Pa$

A5.1.6.2 Distributed wind load

Influencing height for each floor level $h_{floor} := 3.6m$

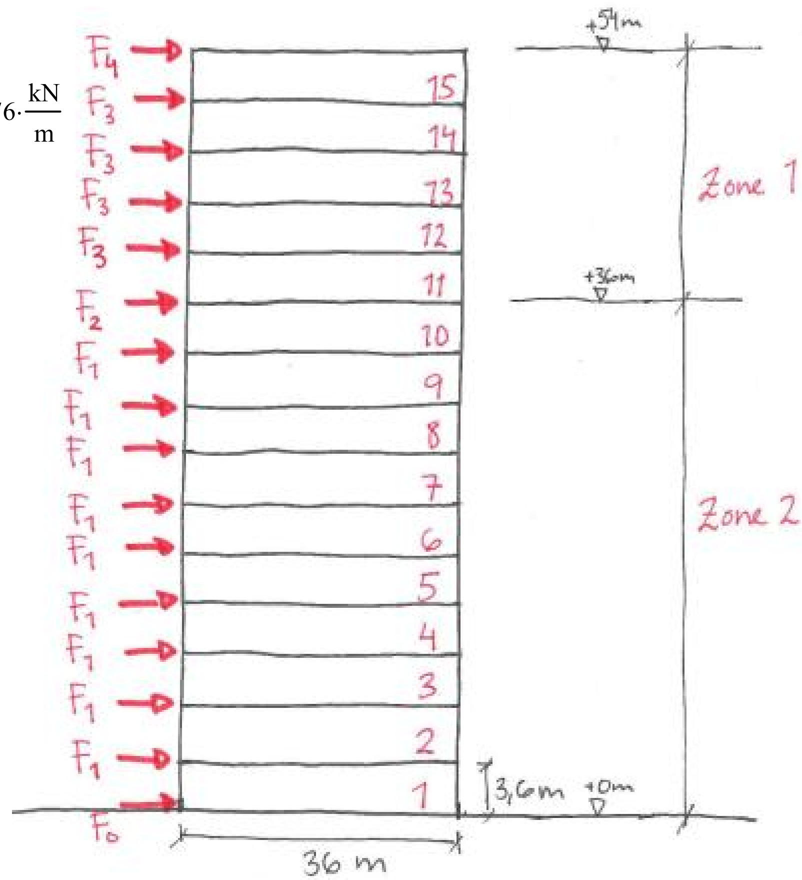
F.0 is not set to zero but the level arm is zero so this force will not have any contribution to bracing members or stabilising walls.

$$f_1 := h_{floor} \cdot w_2 = 4.878 \cdot \frac{kN}{m}$$

$$f_2 := h_{floor} \cdot \left(\frac{w_1 + w_2}{2} \right) = 5.176 \cdot \frac{kN}{m}$$

$$f_3 := h_{floor} \cdot w_1 = 5.473 \cdot \frac{kN}{m}$$

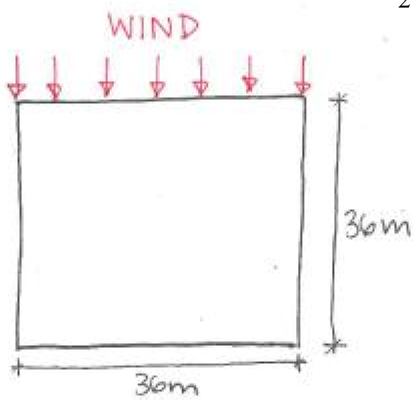
$$f_4 := \frac{h_{floor}}{2} \cdot w_1 = 2.737 \cdot \frac{kN}{m}$$

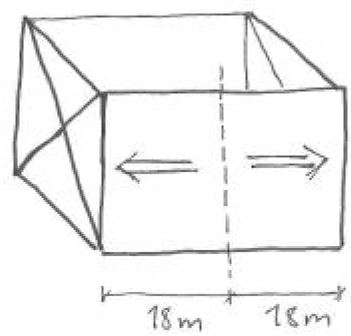


A5.2 Dimensions of diagonal and chevron bracning

Calculations are performed according to Eurocode for timber and steel.

Influence length $l_{infl} := \frac{a}{2} = 18\,m$





Height of one bracing unit $h_{brace} := h_{floor} = 3.6\,m$

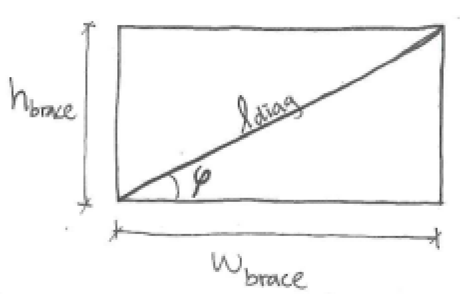
Width of one bracing unit $w_{brace} := \frac{a}{4} = 9\,m$ Have tested for a/4 and a/6

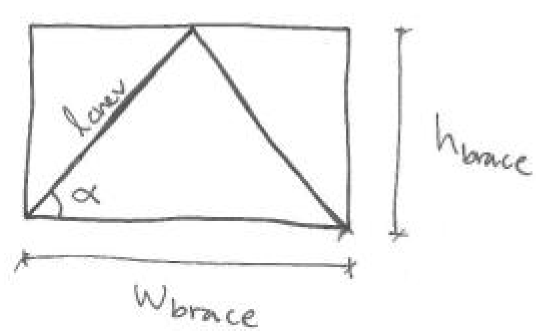
Angle of diagonal memeber $\varphi := \operatorname{atan}\left(\frac{h_{brace}}{w_{brace}}\right) = 21.801 \cdot deg$

Angle of chevron member $\alpha := \operatorname{atan}\left[\frac{h_{brace}}{\left(\frac{w_{brace}}{2}\right)}\right] = 38.66 \cdot deg$

Length of diagonal member $l_{diag} := \sqrt{h_{brace}^{2} + w_{brace}^{2}} = 9.693\,m$

Length of chevron member $l_{chev} := \sqrt{h_{brace}^{2} + \left(\frac{w_{brace}}{2}\right)^{2}} = 5.763\,m$





A5.2.1 Wind load on different storeys

$$F_1 := f_1 \cdot l_{infl} = 87.807 \cdot kN$$

$$F_2 := f_2 \cdot l_{infl} = 93.163 \cdot kN$$

$$F_3 := f_3 \cdot l_{infl} = 98.52 \cdot kN$$

$$F_4 := f_4 \cdot l_{infl} = 49.26 \cdot kN$$

A5.2.1.1 Horisontal load on every third floor

The resultant horizontal force accumulates through the building. Here the force is calculated for ever third floor. The number refers to the storey on which the load is applied.

$$H_{13} := F_4 + 2F_3 = 246.299 \cdot kN$$

$$H_{10} := F_4 + 4F_3 + F_2 = 536.502 \cdot kN$$

$$H_7 := F_4 + 4F_3 + F_2 + 3F_1 = 799.923 \cdot kN$$

$$H_4 := F_4 + 4F_3 + F_2 + 6F_1 = 1.063 \times 10^3 \cdot kN$$

$$H_1 := F_4 + 4F_3 + F_2 + 9F_1 = 1.327 \times 10^3 \cdot kN$$

The forces in the diagonal bracing units and chevron bracing units are then calcualted for different storeys.

Force in diagonal bracing unit

$$P_{13} := \frac{H_{13}}{\cos(\varphi)} = 265.272 \cdot kN$$

$$P_{10} := \frac{H_{10}}{\cos(\varphi)} = 577.83 \cdot kN$$

$$P_7 := \frac{H_7}{\cos(\varphi)} = 861.543 \cdot kN$$

$$P_4 := \frac{H_4}{\cos(\varphi)} = 1.145 \times 10^3 \cdot kN$$

$$P_1 := \frac{H_1}{\cos(\varphi)} = 1.429 \times 10^3 \cdot kN$$

Force in chevron bracing

$$P_{13.chev} := \frac{H_{13}}{\cos(\alpha)\, 2} = 157.708 \cdot kN$$

$$P_{10.chev} := \frac{H_{10}}{\cos(\alpha)\, 2} = 343.529 \cdot kN$$

$$P_{7.chev} := \frac{H_7}{\cos(\alpha)\, 2} = 512.2 \cdot kN$$

$$P_{4.chev} := \frac{H_4}{\cos(\alpha)\, 2} = 680.872 \cdot kN$$

$$P_{1.chev} := \frac{H_1}{\cos(\alpha)\, 2} = 849.544 \cdot kN$$

$$P := \begin{pmatrix} P_{13} \\ P_{10} \\ P_7 \\ P_4 \\ P_1 \end{pmatrix} \qquad P_{chev} := \begin{pmatrix} P_{13.chev} \\ P_{10.chev} \\ P_{7.chev} \\ P_{4.chev} \\ P_{1.chev} \end{pmatrix}$$

A5.2.2 Material data for glulam

A5.2.2.1 Characteristic strenght value

Compression parallell to grain $f_{c.0.k.glulam} := 30MPa$ (Assume strenght class GL30h)

Tension parallell to grain $f_{t.0.k.glulam} := 24MPa$

Elastic modulus $E_{0.05.glulam} := 11300MPa$

Partial factors: $\gamma_{M.glulam} := 1.25$

Assuming short term load and service class 2

$k_{mod.glulam} := 0.9$

$k_{def} := 0.8$

A5.2.3 Single diagonal bracing, glulam

Dimensioning of glulam diagonals, by first assuming a cross-section which the capacity is calcualted for. And then changing cross-section until sufficient dimensions are obtained.

Calculations are made according to **SS-EN 1995-1-1:2004**, all references is made to this Eurocode.

A5.2.3.1 Dimensions

Five different cross-section are calculated for because of the five different loads, corresponding to diagonals on different storeys.

$i := 0..4$

Width of cross-section *Height of cross-section*

$$w_{glulam} := \begin{pmatrix} 2\cdot140 \\ 2\cdot165 \\ 2\cdot165 \\ 2\cdot190 \\ 2\cdot190 \end{pmatrix} mm \qquad h_{glulam} := \begin{pmatrix} 8\cdot45 \\ 7\cdot45 \\ 8\cdot45 \\ 8\cdot45 \\ 9\cdot45 \end{pmatrix} mm$$

Area of cross-section $A_{glulam_i} := w_{glulam_i} \cdot h_{glulam_i}$

Effect of member size

$$k_{h.glulam.y_i} := \begin{cases} \min\left[\left(\dfrac{600mm}{h_{glulam_i}}\right)^{0.1}, 1.1\right] & \text{if } h_{glulam_i} \le 600mm \\ 1 & \text{otherwise} \end{cases}$$

$$k_{h.glulam.z_i} := \begin{cases} \min\left[\left(\dfrac{600mm}{w_{glulam_i}}\right)^{0.1}, 1.1\right] & \text{if } h_{glulam_i} \le 600mm \\ 1 & \text{otherwise} \end{cases}$$

A5.2.3.2 Design strength values

Compression parallell to grain y and z direction

$$f_{c.0.d.glulam.y_i} := k_{mod.glulam} \cdot k_{h.glulam.y_i} \cdot \frac{f_{c.0.k.glulam}}{\gamma_{M.glulam}}$$

$$f_{c.0.d.glulam.z_i} := k_{mod.glulam} \cdot k_{h.glulam.z_i} \cdot \frac{f_{c.0.k.glulam}}{\gamma_{M.glulam}}$$

Tension parallell to grain

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

(Neglecting size effects in tension, because it is unknown which side is the width.)

A5.2.3.3 Compression capacities

A diagonal bracing unit can be modelled as a column subjected to compression, hence the following expression should be fulfilled.

$$\boxed{\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} \le 1}$$

eq. 6.23 in section 6.3

Critical axial load: $\boxed{N_{cr} := k_c \cdot f_{c.0.d} \cdot A}$

Second moment of inertia and slenderness with respect to both directions.

$$I_{glulam.yy_i} := \frac{w_{glulam_i} \cdot \left(h_{glulam_i}\right)^3}{12} \qquad I_{glulam.zz_i} := \frac{\left(w_{glulam_i}\right)^3 \cdot h_{glulam_i}}{12}$$

$$i_{glulam.y_i} := \sqrt{\frac{I_{glulam.yy_i}}{A_{glulam_i}}} \qquad i_{glulam.z_i} := \sqrt{\frac{I_{glulam.zz_i}}{A_{glulam_i}}}$$

$$\lambda_{glulam.y_i} := \frac{l_{diag}}{i_{glulam.y_i}} \qquad \lambda_{glulam.z_i} := \frac{l_{diag}}{i_{glulam.z_i}}$$

$$\lambda_{rel.glulam.y_i} := \frac{\lambda_{glulam.y_i}}{\pi} \cdot \sqrt{\frac{f_{c.0.k.glulam}}{E_{0.05.glulam}}} \qquad \lambda_{rel.glulam.z_i} := \frac{\lambda_{glulam.z_i}}{\pi} \cdot \sqrt{\frac{f_{c.0.k.glulam}}{E_{0.05.glulam}}}$$

Reduction factor of the strength for both directions

$$\beta_{c.glulam} := 0.1$$

$$k_{glulam.y_i} := 0.5 \cdot \left[1 + \beta_{c.glulam} \cdot \left(\lambda_{rel.glulam.y_i} - 0.3\right) + \left(\lambda_{rel.glulam.y_i}\right)^2\right]$$ eq. 6.27 in section 6.3

$$k_{glulam.z_i} := 0.5 \cdot \left[1 + \beta_{c.glulam} \cdot \left(\lambda_{rel.glulam.z_i} - 0.3\right) + \left(\lambda_{rel.glulam.z_i}\right)^2\right]$$

$$k_{c.glulam.y_i} := \frac{1}{k_{glulam.y_i} + \sqrt{\left(k_{glulam.y_i}\right)^2 - \left(\lambda_{rel.glulam.y_i}\right)^2}}$$ eq. 6.25 in section 6.3

$$k_{c.glulam.z_i} := \frac{1}{k_{glulam.z_i} + \sqrt{\left(k_{glulam.z_i}\right)^2 - \left(\lambda_{rel.glulam.z_i}\right)^2}}$$

Critical axial load and the capacity fo the single diagonal bracing unit

By the condition given for columns, $\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} \le 1$ the maximum compression stress can be calculated as:

$$\boxed{\sigma_{c.0.d} := k_c \cdot f_{c.0.d}}$$

Maximum stress with regard to y-direction $\sigma_{c.0.d.glulam.y_i} := k_{c.glulam.y_i} \cdot f_{c.0.d.glulam.y_i}$

Maximum stress with regard to z-direction $\sigma_{c.0.d.glulam.z_i} := k_{c.glulam.z_i} \cdot f_{c.0.d.glulam.z_i}$

Maximum axial load in y- and z-direction

$$N_{cr.glulam.y_i} := \sigma_{c.0.d.glulam.y_i} \cdot A_{glulam_i}$$

$$N_{cr.glulam.z_i} := \sigma_{c.0.d.glulam.z_i} \cdot A_{glulam_i}$$

Maximum axial load $N_{cr.glulam_i} := \min\left(N_{cr.glulam.y_i}, N_{cr.glulam.z_i}\right)$

Compression capacity **Applied load** **Utilisation ratio**

$$N_{cr.glulam} = \begin{pmatrix} 574.61 \\ 733.371 \\ 910.868 \\ 1.223 \times 10^3 \\ 1.511 \times 10^3 \end{pmatrix} \cdot kN \qquad P = \begin{pmatrix} 265.272 \\ 577.83 \\ 861.543 \\ 1.145 \times 10^3 \\ 1.429 \times 10^3 \end{pmatrix} \cdot kN \qquad \frac{P}{N_{cr.glulam}} = \begin{pmatrix} 46.166 \\ 78.791 \\ 94.585 \\ 93.651 \\ 94.597 \end{pmatrix} \cdot \%$$

A5.2.3.4 Tension capacities

Members subjected to tension should fulfill the following condition according to Eurocode.

$$\boxed{\sigma_{t.0.d} \le f_{t.0.d}}$$ eq. 6.1 in section 6.1

Actual tension stress $\sigma_{t.0.glulam_i} := \dfrac{P_i}{A_{glulam_i}}$

$$\sigma_{t.0.glulam} = \begin{pmatrix} 2.632 \\ 5.559 \\ 7.252 \\ 8.372 \\ 9.285 \end{pmatrix} \cdot MPa$$

Tension stress capacity $f_{t.0.d.glulam} = 17.28 \cdot MPa$ $\qquad \sigma_{t.0.glulam} \le f_{t.0.d.glulam} = \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix}$

Maximum axial tension force $N_{t.d.glulam_i} := f_{t.0.d.glulam} \cdot A_{glulam_i}$

Tension capacity **Utilisation ratio**

$$N_{t.d.glulam} = \begin{pmatrix} 1.742 \times 10^3 \\ 1.796 \times 10^3 \\ 2.053 \times 10^3 \\ 2.364 \times 10^3 \\ 2.659 \times 10^3 \end{pmatrix} \cdot kN \qquad \frac{P}{N_{t.d.glulam}} = \begin{pmatrix} 15.23 \\ 32.169 \\ 41.968 \\ 48.448 \\ 53.733 \end{pmatrix} \cdot \%$$

A5.2.3 Chevron bracing, glulam

Dimensioning of glulam diagonals, by first assuming a cross-section which the capacity is calcualted for. And then changing cross-section until sufficient dimensions are obtained.

Calculations are made according to **SS-EN 1995-1-1:2004**, all references is made to this Eurocode.

A5.2.3.1 Dimensions

Width of cross-section

$$w_{glulam.chev} := \begin{pmatrix} 215 \\ 2\cdot 115 \\ 2\cdot 140 \\ 2\cdot 140 \\ 2\cdot 165 \end{pmatrix} mm$$

Height of cross-section

$$h_{glulam.chev} := \begin{pmatrix} 5\cdot 45 \\ 6\cdot 45 \\ 6\cdot 45 \\ 6\cdot 45 \\ 6\cdot 45 \end{pmatrix} mm$$

Area of cross-section

$$A_{glulam.chev_i} := w_{glulam.chev_i} \cdot h_{glulam.chev_i}$$

Effect of member size

$$k_{h.glulam.chev.y_i} := \begin{cases} \min\left[\left(\dfrac{600mm}{h_{glulam.chev_i}}\right)^{0.1}, 1.1\right] & \text{if } h_{glulam.chev_i} \le 600mm \\ 1 & \text{otherwise} \end{cases}$$

$$k_{h.glulam.chev.z_i} := \begin{cases} \min\left[\left(\dfrac{600mm}{w_{glulam.chev_i}}\right)^{0.1}, 1.1\right] & \text{if } h_{glulam.chev_i} \le 600mm \\ 1 & \text{otherwise} \end{cases}$$

A5.2.3.2 Design strength values

Compression parallell to grain y- and z-direction

$$f_{c.0.d.glulam.chev.y_i} := k_{mod.glulam} \cdot k_{h.glulam.chev.y_i} \cdot \frac{f_{c.0.k.glulam}}{\gamma_{M.glulam}}$$

$$f_{c.0.d.glulam.chev.z_i} := k_{mod.glulam} \cdot k_{h.glulam.chev.z_i} \cdot \frac{f_{c.0.k.glulam}}{\gamma_{M.glulam}}$$

Tension parallell to grain

$$f_{t.0.d.glulam.chev} := k_{mod.glulam} \cdot \frac{f_{t.0.k.glulam}}{\gamma_{M.glulam}}$$

(Neglecting size effects in tension, because unknown which side is the width.)

A5.2.3.3 Compression capacities

A chevron bracing unit can be modelled as a column subjected to compression, hence the following expression should be fulfilled.

$$\boxed{\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} \le 1}$$

eq. 6.23 in section 6.3

Critical axial load: $\boxed{N_{cr} := k_c \cdot f_{c.0.d} \cdot A}$

Second moment of inertia and slenderness

$$I_{glulam.chev.yy_i} := \frac{w_{glulam.chev_i} \cdot \left(h_{glulam.chev_i}\right)^3}{12}$$

$$i_{glulam.chev.y_i} := \sqrt{\frac{I_{glulam.chev.yy_i}}{A_{glulam.chev_i}}}$$

$$\lambda_{glulam.chev.y_i} := \frac{l_{chev}}{i_{glulam.chev.y_i}}$$

$$\lambda_{rel.glulam.chev.y_i} := \frac{\lambda_{glulam.chev.y_i}}{\pi} \cdot \sqrt{\frac{f_{c.0.k.glulam}}{E_{0.05.glulam}}}$$

$$I_{glulam.chev.zz_i} := \frac{\left(w_{glulam.chev_i}\right)^3 \cdot h_{glulam.chev_i}}{12}$$

$$i_{glulam.chev.z_i} := \sqrt{\frac{I_{glulam.chev.zz_i}}{A_{glulam.chev_i}}}$$

$$\lambda_{glulam.chev.z_i} := \frac{l_{chev}}{i_{glulam.chev.z_i}}$$

$$\lambda_{rel.glulam.chev.z_i} := \frac{\lambda_{glulam.chev.z_i}}{\pi} \cdot \sqrt{\frac{f_{c.0.k.glulam}}{E_{0.05.glulam}}}$$

Reduction factor for the capacity

$\beta_{c.glulam.chev} := 0.1$

$k_{glulam.chev.y_i} := 0.5\cdot\left[1 + \beta_{c.glulam.chev}\cdot\left(\lambda_{rel.glulam.chev.y_i} - 0.3\right) + \left(\lambda_{rel.glulam.chev.y_i}\right)^2\right]$

$k_{glulam.chev.z_i} := 0.5\cdot\left[1 + \beta_{c.glulam.chev}\cdot\left(\lambda_{rel.glulam.chev.z_i} - 0.3\right) + \left(\lambda_{rel.glulam.chev.z_i}\right)^2\right]$

$$k_{c.glulam.chev.y_i} := \frac{1}{k_{glulam.chev.y_i} + \sqrt{\left(k_{glulam.chev.y_i}\right)^2 - \left(\lambda_{rel.glulam.chev.y_i}\right)^2}}$$

$$k_{c.glulam.chev.z_i} := \frac{1}{k_{glulam.chev.z_i} + \sqrt{\left(k_{glulam.chev.z_i}\right)^2 - \left(\lambda_{rel.glulam.chev.z_i}\right)^2}}$$

Critical axial load and the capacity fo the diagonal

By the condition given for columns, $\frac{\sigma_{c.0.d}}{k_{c.y}\cdot f_{c.0.d}} \le 1$ the maximum compression stress can be calculated as:

$\boxed{\sigma_{c.0.d} := k_c\cdot f_{c.0.d}}$

Maximum stress with regard to y-direction $\sigma_{c.0.d.glulam.chev.y_i} := k_{c.glulam.chev.y_i}\cdot f_{c.0.d.glulam.chev.y_i}$

Maximum stress with regard to z-direction $\sigma_{c.0.d.glulam.chev.z_i} := k_{c.glulam.chev.z_i}\cdot f_{c.0.d.glulam.chev.z_i}$

Maximum axial load in y.- and z-direction

$N_{cr.glulam.chev.y_i} := \sigma_{c.0.d.glulam.chev.y_i}\cdot A_{glulam.chev_i}$

$N_{cr.glulam.chev.z_i} := \sigma_{c.0.d.glulam.chev.z_i}\cdot A_{glulam.chev_i}$

Maximum axial load $N_{cr.glulam.chev_i} := \min\left(N_{cr.glulam.chev.y_i}, N_{cr.glulam.chev.z_i}\right)$

Compression capacity **Applied load** **Utilisation ratio**

$$N_{cr.glulam.chev} = \begin{pmatrix} 455.795 \\ 661.614 \\ 1.044\times10^3 \\ 1.044\times10^3 \\ 1.231\times10^3 \end{pmatrix}\cdot kN \qquad P_{chev} = \begin{pmatrix} 157.708 \\ 343.529 \\ 512.2 \\ 680.872 \\ 849.544 \end{pmatrix}\cdot kN \qquad \frac{P_{chev}}{N_{cr.glulam.chev}} = \begin{pmatrix} 34.601 \\ 51.923 \\ 49.039 \\ 65.188 \\ 69.013 \end{pmatrix}\cdot\%$$

A5.2.3.4 Tension capacities

Members subjected to tension should fulfill the following condition according to Eurocode:

$$\boxed{\sigma_{t.0.d} \le f_{t.0.d}}$$ eq. 6.1 in section 6.1

Actual tension stress $$\sigma_{t.0.glulam.chev_i} := \frac{P_{chev_i}}{A_{glulam.chev_i}}$$

$$\sigma_{t.0.glulam.chev} = \begin{pmatrix} 3.26 \\ 5.532 \\ 6.775 \\ 9.006 \\ 9.535 \end{pmatrix} \cdot MPa$$

Tension stress capacity $$f_{t.0.d.glulam.chev} = 17.28 \cdot MPa$$

$$\sigma_{t.0.glulam.chev} \le f_{t.0.d.glulam.chev} = \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix}$$

Tension capacity expressed in kN

Maximum axial tension force $$N_{t.d.glulam.chev_i} := f_{t.0.d.glulam.chev} \cdot A_{glulam.chev_i}$$

Tension capacity

$$N_{t.d.glulam.chev} = \begin{pmatrix} 835.92 \\ 1.073 \times 10^3 \\ 1.306 \times 10^3 \\ 1.306 \times 10^3 \\ 1.54 \times 10^3 \end{pmatrix} \cdot kN$$

Utilisation ratio

$$\frac{P}{N_{t.d.glulam.chev}} = \begin{pmatrix} 31.734 \\ 53.847 \\ 65.949 \\ 87.667 \\ 92.811 \end{pmatrix} \cdot \%$$

A5.2.4 Steel members

Dimensions of the steel members are designed with help of table from Tibnor. The table tells the capacity of a column in compression. In this case the members can be subjected to tension as well. Therefore their tension capacity is controlled.

The areas and loads are changed when testing for different lengths of the members.

Force in diagonal member $N_{Ed} := \begin{pmatrix} 158 \\ 344 \\ 512 \\ 381 \\ 850 \end{pmatrix} kN$ (The loads are for the length of 5.7 metre)

According to SS-EN 1993-1-1:2005 section 6.2.3 the design value for the thension force Ned for each cross-section should satisfy:

$$\boxed{N_{Ed} \le N_{t.Rd}}$$ eq. 6.5 in section 6.2

Tension capacity $$\boxed{N_{t.Rd} := \frac{A \cdot f_y}{\gamma_{M0}}}$$

A5.2.4.1 *Material properties and geometries*

Yield strength $f_y := 355MPa$

Partial factor $\gamma_{M0} := 1$

Areas for the cross-section obtained in the tables $A_{VKR} := \begin{pmatrix} 2060 \\ 3520 \\ 4160 \\ 5090 \\ 5490 \end{pmatrix} mm^2$ $A_{KCKR} := \begin{pmatrix} 1710 \\ 3060 \\ 4030 \\ 4670 \\ 5770 \end{pmatrix} mm^2$

A5.2.4.2 *Tension capacity*

VKR-profiles $N_{t.Rd.VKR} := \frac{A_{VKR} \cdot f_y}{\gamma_{M0}}$ *KCKR-profiles* $N_{t.Rd.KCKR} := \frac{A_{KCKR} \cdot f_y}{\gamma_{M0}}$

Utilisation ratio VKR-profiles

$$\frac{N_{Ed}}{N_{t.Rd.VKR}} = \begin{pmatrix} 21.605 \\ 27.529 \\ 34.67 \\ 21.085 \\ 43.613 \end{pmatrix} \cdot \%$$

Utilisation ratio KCKR-profiles

$$\frac{N_{Ed}}{N_{t.Rd.KCKR}} = \begin{pmatrix} 26.028 \\ 31.667 \\ 35.788 \\ 22.982 \\ 41.497 \end{pmatrix} \cdot \%$$

Appendix A6

The tables in this appendix contain complementary information to the tables in Chapter 5.

A6.1 Columns

Table 1 Dimensions of glulam columns. Complementary information to Table 3 in Chapter 5

Glulam, Lc40	Dimension [mm]	Height [mm]	Width [mm]	Utilisation [%]
0.5 MN	280×270(f)	6×45	2×140	38.2
1 MN	330×270(f)	6×45	2×165	64.8
2 MN	330×360	8×45	2×165	95.9
3 MN	430×405	9×45	2×215	98.4
4 MN	430×540	12×45	2×215	99.5
5 MN	570×540	12×45	3×190	94.1
6 MN	645×540	12×45	3×215	99.9
7 MN	645×630	14×45	3×215	99.9

Table 2 Dimensions of glulam columns. Complementary information to Table 3 in Chapter 5. The values in the column with dimensions after fire are referring to the dimensions the column after 90 min of standard fire.

Load ULS	**Load fire**	**Dim. before fire**	**Dim. after fire**	**Utilisation ULS load**	**Utilisation Fire load**
0.5	0.267	280×270	163×153	38.2	54.6
1	0.533	330×270	213×153	64.8	83.6
2	1.066	330×360	213×243	95.9	69.5
3	1.599	430×405	313×288	98.4	53.3
4	2.132	430×540	313×423	99.5	47.7
5	2.655	570×540	453×423	94.1	40.0
6	3.198	645×540	528×423	99.9	41.2
7	3.731	645×630	528×513	99.9	39.2

Table 3 Dimensions of concrete columns. Complementary information to Table 3 in Chapter 5. Additional dimensions due to fire are not included in this table.

Concrete, C30/37	Dimension [mm]	Reinforcement	M_{Rd} [kNm]	M_{Ed} [kNm]	Utilisation [%]
0.5 MN	224×224	2 + 2 ϕ20	44.700	36.891	82.5
1 MN	266×266	2 + 2 ϕ20	65.826	62.962	95.6
2 MN	324×324	3 + 3 ϕ20	92.390	78.428	84.9
3 MN	374×374	4 + 4 ϕ20	105.094	75.488	71.8
4 MN	412×412	5 + 5 ϕ20	86.188	84.962	98.6
5 MN	458×458	6 + 6 ϕ20	103.146	84.609	82.0
6 MN	495×495	7 + 7 ϕ20	94.279	91.568	97.1
7 MN	534×534	8 + 8 ϕ20	110.253	97.178	88.1

Table 4 Dimensions of HEA columns. Complementary information to Table 3 in Chapter 5. In column two and three the additional fire protection can be seen.

HEA S355J2	Profile	Height [mm]	Width [mm]	Utilisation [%]
0.5 MN	HEA160	152 + 2×30.8	160 + 2×30.8	82.0
1 MN	HEA200	190 + 2×30.8	200 + 2×30.8	71.7
2 MN	HEA260	250 + 2×30.8	260 + 2×30.8	91.3
3 MN	HEA300	290 + 2×30.8	300 + 2×30.8	97.7
4 MN	HEA400	390 + 2×30.8	300 + 2×30.8	88.9
5 MN	HEA450	440 + 2×30.8	300 + 2×30.8	99.6
6 MN	HEA600	590 + 2×30.8	300 + 2×30.8	97.1
7 MN	HEA800	790 + 2×30.8	300 + 2×30.8	92.1

Table 5 Dimensions of VKR columns. Complementary information to Table 3 in Chapter 5. In column two and three the additional fire protection can be seen.

VKR S355J2H	Height [mm]	Width [mm]	Thickness [mm]	Utilisation [%]
0.5 MN	100 + 2×30.8	100 + 2×30.8	8	99.8
1 MN	150 + 2×30.8	150 + 2×30.8	6.3	99.0
2 MN	180 + 2×30.8	180 + 2×30.8	10	98.5
3 MN	250 + 2×30.8	250 + 2×30.8	10	95.8
4 MN	250 + 2×30.8	250 + 2×30.8	16	82.8
5 MN	350 + 2×30.8	350 + 2×30.8	12.5	87.1
6 MN	300 + 30.8	300 + 30.8	16	99.3
7 MN	350 + 30.8	350 + 30.8	16	96.7

A6.2 Beams

Table 6 Dimensions for glulam beams with an influence length of 10 metres. Complementary information to Table 6 in Chapter 5.

Glulam, L40c, I Infl. length 10m	Dimension [mm]	Height [mm]	Width [mm]	Height /width	Utilisation M [%]	Utilisation V [%]	Utilisation deflection [%]
4 m	495×330	11×45	2×165	1.5	56.0	92.9	58.8
6 m	630×380	14×45	2×190	1.7	69.3	95.5	84.2
8 m	810×380	18×45	2×190	2.1	74.8	99.5	94.5
10 m	990×430	22×45	2×215	2.3	69.7	90.7	90.3
12 m	1170×430	26×45	2×215	2.7	72.3	92.5	95.1

Table 7 Dimensions for glulam beams with an influence length of 10 metres. Complementary information to Table 6 in Chapter 5.

Glulam, L40c, II Infl. length 10m	Dimension [mm]	Height [mm]	Width [mm]	Height /width	Utilisation M [%]	Utilisation V [%]	Utilisation deflection [%]
4 m	810×190	18×45	190	4.3	37.0	98.5	23.3
6 m	1080×215	24×45	215	5.0	41.6	98.4	29.5
8 m	1125×280	25×45	2×140	4.0	52.7	97.3	47.9
10 m	1395×280	31×45	2×140	5.0	53.8	98.6	49.4
12 m	1440×330	32×45	2×165	4.4	62.0	97.8	66.3

Table 8 Dimensions for Kerto-S beams with an influence length of 10 metres. Complementary information to Table 6 in Chapter 5.

LVL, Kerto-S, I Infl. length 10m	Dimension [mm]	Height [mm]	Width [mm]	Height /width	Utilisation M [%]	Utilisation V [%]	Utilisation deflection [%]
4 m	430×300	430	4×75	1.4	55.9	96.3	92.9
6 m	600×375	600	5×75	1.6	52.1	83.5	93.3
8 m	800×375	800	5×75	2.1	52.4	84.0	94.0
10 m	940×450	940	6×75	2.1	49.9	75.2	95.5
12 m	1130×450	1130	6×75	2.5	50.1	75.5	95.9

Table 9 Dimensions for Kerto –S beams with an influence length of 10 metres. Complementary information to Table 6 in Chapter 5.

LVL, Kerto-S, II Infl. length 10m	Dimension [mm]	Height [mm]	Width [mm]	Height /width	Utilisation M [%]	Utilisation V [%]	Utilisation deflection [%]
4 m	800×225	800	3×75	3.6	21.6	69.3	19.3
6 m	850×225	850	3×75	3.8	43.2	97.9	54.5
8 m	1120×225	1120	3×75	5.0	44.4	99.6	56.8
10 m	1100×300	1100	4×75	3.7	54.3	95.6	88.6
12 m	1290×300	1290	4×75	4.3	57.1	98.3	95.5

Table 10 Dimensions for HEA beams with an influence length of 10 metres. Complementary information to Table 6 in Chapter 5.

HEA, infl. 10 m	Profile	Height [mm]	Width [mm]	Utilisation [%]
4 m	HEA280	270 + 30.8	280 + 2×30.8	95.9
6 m	HEA400	390 + 15.4	300 + 2×15.4	93.3
8 m	HEA550	540 + 15.4	300 + 2×15.4	90.4
10 m	HEA700	690 + 15.4	300 + 2×15.4	91.6
12 m	HEA900	890 + 15.4	300 + 2×15.4	78.9

Table 11 Dimensions for HEB beams with an influence length of 10 metres. Complementary information to Table 6 in Chapter 5.

HEB, infl. 10 m	Profile	Height [mm]	Width [mm]	Utilisation [%]
4 m	HEB 260	260 + 30.8	260 + 2×30.8	89.6
6 m	HEB360	360 + 30.8	300 + 2×30.8	96.8
8 m	HEB500	500 + 15.4	300 + 2×15.4	95.4
10 m	HEB650	650+ 15.4	300 + 2×15.4	93.6
12 m	HEB800	800+ 15.4	300 + 2×15.4	92.9

Table 12 Dimensions for Concrete beams with an influence length of 10 metres. Complementary information to Table 6 in Chapter 5.

Concrete RB/F Infl. 10 m	Dimension [mm]	RB/F
4 m	400×200	20/40
6 m	500×300	30/50
8 m	600×400	40/60
10 m	700×400	40/70
12 m	800×400	40/80

Table 13 Dimensions for glulam beams with an influence length of 6 metres. Complementary information to Table 7 in Chapter 5.

Glulam, L40c, I Infl. length 6m	Dimension [mm]	Height [mm]	Width [mm]	Height /width	Utilisation M [%]	Utilisation V [%]	Utilisation deflection [%]
4 m	405×230	9×45	2×115	1.8	70.6	97.6	92.4
6 m	540×330	12×45	2×165	1.6	64.7	77.3	92.9
8 m	720×330	16×45	2×165	2.2	65.8	77.8	93.7
10 m	855×380	19×45	2×190	2.3	64.0	71.8	96.1
12 m	1035×380	23×45	2×190	2.7	63.3	71.7	94.5

Table 14 Dimensions for glulam beams with an influence length of 6 metres. Complementary information to Table 7 in Chapter 5.

Glulam, L40c, II Infl. length 6m	Dimension [mm]	Height [mm]	Width [mm]	Height /width	Utilisation M [%]	Utilisation V [%]	Utilisation deflection [%]
4 m	675×165(f)	15×45	165	4.1	36.9	81.8	27.9
6 m	765×190	17×45	190	4.0	56.4	94.4	56.5
8 m	855×215	19×45	215	4.0	71.2	100	85.2
10 m	1035×230	23×45	2×115	4.5	71.4	97.1	88.4
12 m	1170×280	26×45	2×140	4.2	66.8	85.5	88.0

Table 15 Dimensions for Kerto-S beams with an influence length of 6 metres. Complementary information to Table 7 in Chapter 5.

LVL, Kerto-S, I Infl. length 6m	Dimension [mm]	Height [mm]	Width [mm]	Height /width	Utilisation M [%]	Utilisation V [%]	Utilisation deflection [%]
4 m	400×225	400	3×75	1.8	51.8	83.0	92.5
6 m	540×300	540	4×75	1.8	48.4	69.8	96.4
8 m	720×300	720	4×75	2.4	48.8	70.3	97.3
10 m	840×375	840	5×75	2.2	45.4	61.0	97.4
12 m	1020×375	1020	5×75	2.7	44.7	60.9	95.1

Table 16 Dimensions for Kerto-S beams with an influence length of 6 metres. Complementary information to Table 7 in Chapter 5.

LVL, Kerto-S, II Infl. length 6m	Dimension [mm]	Height [mm]	Width [mm]	Height /width	Utilisation M [%]	Utilisation V [%]	Utilisation deflection [%]
4 m	500×225(f)	500	3×75	2.2	33.2	66.6	47.6
6 m	750×225(f)	750	3×75	5.0	33.5	67.1	48.0
8 m	800×225	800	3×75	3.6	52.4	84.0	94.0
10 m	1000×225	1000	3×75	4.4	52.7	84.5	94.8
12 m	1200×225	1200	3×75	5.3	53.0	85.0	95.5

Table 17 Dimensions for HEA beams with an influence length of 6 metres. Complementary information to Table 7 in Chapter 5.

HEA, infl. 6 m	Profile	Height [mm]	Width [mm]	Utilisation [%]
4 m	HEA260	250 + 30.8	260 + 2×30.8	76.7
6 m	HEA340	330 + 30.8	300 + 2×30.8	87.9
8 m	HEA450	440 + 15.4	300 + 2×15.4	90.7
10 m	HEA600	590 + 15.4	300 + 2×15.4	80.6
12 m	HEA700	690 + 15.4	300 + 2×15.4	89.8

Table 18 Dimensions for HEB beams with an influence length of 6 metres. Complementary information to Table 7 in Chapter 5.

HEB, infl. 6 m	Profile	Height [mm]	Width [mm]	Utilisation [%]
4 m	HEB240	240 + 30.8	240 + 2×30.8	72.7
6 m	HEB300	300 + 30.8	300 + 2×30.8	95.1
8 m	HEB450	450 + 15.4	300 + 2×15.4	73.6
10 m	HEB550	550 + 15.4	300 + 2×15.4	82.7
12 m	HEB650	650 + 15.4	300 + 2×15.4	92.0

Table 19 Dimensions for concrete beams with an influence length of 6 metres. Complementary information to Table 7 in Chapter 5.

Concrete RB/F Infl. 6 m	Dimension [mm]	RB/F
4 m	300×200	20/30
6 m	500×200	20/50
8 m	500×300	30/50
10 m	600×400	40/60
12 m	700×400	40/70

A6.3 Floor elements

Table 20 Dimensions of timber-concrete composite floor elements. Complementary information to Table 8 in Chapter 5.

Composite floor	Height	Width	Utilisation deflection [%]	Fundamental frequency [Hz]
6 m	290	165	100	7.68
8 m	390	210	84.3	7.01
10 m	550	220	62.5	7.03
12 m	725	220	47.2	7.04

Table 21 Dimensions of hollow core floor elements. Complementary information to Table 8 in Chapter 5.

Hollow core slab	Height	HD/F
6 m	200	HD/F 120/20
8 m	200	HD/F 120/20
10 m	270	HD/F 120/27
12 m	270	HD/F 120/27

Table 22 Dimensions of TT floor elements. Complementary information to Table 8 in Chapter 5.

TT-slab	Height	TT/F
6 m	200	TT/F 240/20
8 m	200	TT/F 240/20
10 m	300	TT/F 240/30
12 m	400	TT/F 240/40

A6.4 Timber wall elements

Table 23 Dimensions for timber walls without fire gypsum board. Complementary information to Table 12 in Chapter 5.

Infl. 6m Lwall 0.8m	Thickness [mm]	Utilisation Column[%]	Utilisation, M beam [%]	Utilisation, V beam [%]	Utilisation, u beam [%]
1st floor	310	81.6	2.6	9.6	1.0
6th floor	259	71.0	3.1	10.8	1.2
11th floor	158	95.2	5.0	15.7	2.1

Table 24 Dimensions for timber walls without fire gypsum board. Complementary information to Table 13 in Chapter 5.

Infl. 6m Lwall 1.0 m	Thickness [mm]	Utilisation Column[%]	Utilisation, M beam [%]	Utilisation, V beam [%]	Utilisation, u beam [%]
1st floor	259	95.4	3.1	10.8	1.2
6th floor	259	62.6	3.1	10.8	1.2
11th floor	158	82.5	5.0	15.7	2.1

Table 25 Dimensions for timber walls without fire gypsum board. Complementary information to Table 10 in Chapter 5.

Infl. 4m Lwall 0.8m	Thickness [mm]	Utilisation Column[%]	Utilisation, M beam [%]	Utilisation, V beam [%]	Utilisation, u beam [%]
1st floor	259	78.1	2.1	7.5	0.8
6th floor	221	86.9	2.5	8.7	0.8
11th floor	158	70.4	3.4	10.7	1.4

Table 26 Dimensions for timber walls without fire gypsum board. Complementary information to Table 11 in Chapter 5.

Infl. 4m Lwall 1.0m	Thickness [mm]	Utilisation Column[%]	Utilisation, M beam [%]	Utilisation, V beam [%]	Utilisation, u beam [%]
1st floor	259	68.8	2.1	7.5	0.8
6th floor	208	92.4	2.6	9.8	0.8

A6.5 Diagonal bracing

Table 27 Dimensions for glulam diagonal bracing with a buckling length of 9.7 metres. Complementary information to Table 15 in Chapter 5.

Glulam (9.7 m)	Storey	Height [mm]	Width [mm]	$N_{c.Rd}$ [kN]]	$N_{t.Rd}$ [kN]	Utilisation [%]
265 kN	13 (f)	8×45	2×140	575	1742	46.2
578 kN	10	7×45	2×165	733	1796	78.9
862 kN	7	8×45	2×165	911	2053	94.6
1145 kN	4	8×45	2×190	1223	2364	93.6
1429 kN	1	9×45	2×190	1511	2659	94.6

Table 28 Dimensions for glulam diagonal bracing with a buckling length of 7 metres. Complementary information to Table 14 in Chapter 5.

Glulam (7 m)	Storey	Height [mm]	Width [mm]	$N_{c.Rd}$ [kN]	$N_{t.Rd}$ [kN]	Utilisation [%]
287 kN	13 (f)	7×45	2×115	539	1252	53.3
626 kN	10 (f)	7×45	2×140	926	1524	67.5
933 kN	7	7×45	2×165	1325	1796	70.4
1240 kN	4	7×45	2×165	1325	1796	93.6
1547 kN	1	8×45	2×165	1626	2053	95.1

Table 29 Dimensions for VKR diagonal bracing with a buckling length of 10 metres. Complementary information to Table 15 in Chapter 5.

VKR S355J2H (10 m)	Storey	Height [mm]	Width [mm]	t [mm]	$N_{c,Rd}$ [kN]	$N_{t,Rd}$ [kN]	Utilisation [%]
265 kN	13	180 + 2×30.8	180 + 2×30.8	6.3	395	1537	67.1
578 kN	10	180 + 2×30.8	180 + 2×30.8	10	585	2375	98.8
862 kN	7	200 + 2×30.8	200 + 2×30.8	12.5	959	3270	89.9
1145 kN	4	200 + 2×30.8	200 + 2×30.8	16	1150	4083	99.6
1429 kN	1	250 + 2×30.8	250 + 2×30.8	10	1510	3369	94.6

Table 30 Dimensions for VKR diagonal bracing with a buckling length of 7 metres. Complementary information to Table 14 in Chapter 5.

VKR S355J2H (7 m)	Storey	Height [mm]	Width [mm]	t [mm]	$N_{c,Rd}$ [kN]	$N_{t,Rd}$ [kN]	Utilisation [%]
287 kN	13	140 + 2×46.2	140 + 2×46.2	5	294	948	97.6
626 kN	10	150 + 2×30.8	150 + 2×30.8	10	641	1949	97.7
933 kN	7	180 + 2×30.8	180 + 2×30.8	10	1080	2375	86.4
1240 kN	4	200 + 2×30.8	200 + 2×30.8	10	1430	2659	86.7
1547 kN	1	200 + 2×30.8	200 + 2×30.8	12.5	1730	3270	89.4

Table 31 Dimensions for KCKR diagonal bracing with a buckling length of 10 metres. Complementary information to Table 15 in Chapter 5.

KCKR S355J2H (10 m)	Storey	Diameter [mm]	t [mm]	$N_{c.Rd}$ [kN]	$N_{t.Rd}$ [kN]	Utilisation [%]
265 kN	13	168.3 + 2×30.8	8	290	1431	91.4
578 kN	10	244.5 + 2×30.8	8	622	2109	92.9
862 kN	7	273.0 + 2×30.8	10	1020	2932	84.5
1145 kN	4	273.0 + 2×30.8	12.5	1240	3621	92.3
1429 kN	1	323.9 + 2×30.8	10	1550	3500	92.2

Table 32 Dimensions for KCKR diagonal bracing with a buckling length of 7 metres. Complementary information to Table 14 in Chapter 5.

KCKR S355J2H (7 m)	Storey	Diameter [mm]	t [mm]	$N_{c.Rd}$ [kN]	$N_{t.Rd}$ [kN]	Utilisation [%]
287 kN	13	168.3 + 2×30.8	6	310	1086	92.6
626 kN	10	193.7 + 2×30.8	10	709	2048	88.3
933 kN	7	219.1 + 2×30.8	10	970	2332	96.2
1240 kN	4	244.5 + 2×30.8	10	1260	2616	98.4
1547 kN	1	273.0 + 2×30.8	10	1610	2932	96.1

A6.6 Chevron bracing

Table 33 Dimensions for glulam chevron bracing with a buckling length of 4.7 metres. Complementary information to Table 16 in Chapter 5.

Glulam (4.7 m)	Storey	Height [mm]	Width [mm]	$N_{c.Rd}$ [kN]	$N_{t.Rd}$ [kN]	Utilisation [%]
192 kN	13 (f)	5×45	215	656	836	29.3
419 kN	10 (f)	6×45	2×115	937	1073	44.7
625 kN	7 (f)	7×45	2×115	1093	1252	57.2
830 kN	4 (f)	6×45	2×140	1378	1306	63.6
1036 kN	1 (f)	6×45	2×165	1624	1540	67.3

Table 34 Dimensions for glulam chevron bracing with a buckling length of 5.7 metres. Complementary information to Table 17 in Chapter 5.

Glulam (5.7 m)	Storey	Height [mm]	Width [mm]	$N_{c.Rd}$ [kN]	$N_{t.Rd}$ [kN]	Utilisation [%]
158 kN	13 (f)	5×45	215	456	836	34.6
344 kN	10 (f)	6×45	2×115	662	1073	51.9
512 kN	7 (f)	6×45	2×140	1044	1306	49.0
681 kN	4 (f)	6×45	2×140	1044	1306	65.2
850 kN	1 (f)	6×45	2×165	1231	1540	69.0

Table 35 Dimensions for VKR chevron bracing with a buckling length of 4.7 metres. Complementary information to Table 16 in Chapter 5.

VKR S355J2H (4.7 m)	Storey	Height [mm]	Width [mm]	t [mm]	$N_{c,Rd}$ [kN]	$N_{t,Rd}$ [kN]	Utilisation [%]
192 kN	13	100 + 2×46.2	100 + 2×46.2	5	225	664	85.5
419 kN	10	120 + 2×46.2	120 + 2×46.2	6.3	456	1001	91.9
625 kN	7	140 + 2×30.8	140 + 2×30.8	6.3	680	1182	91.9
830 kN	4	140 + 2×30.8	140 + 2×30.8	8	834	1477	99.5
1036 kN	1	150 + 2×30.8	150 + 2×30.8	10	1186	1949	87.4

Table 36 Dimensions for VKR chevron bracing with a buckling length of 5.7 metres. Complementary information to Table 17 in Chapter 5.

VKR S355J2H (5.7 m)	Storey	Height [mm]	Width [mm]	t [mm]	$N_{c,Rd}$ [kN]	$N_{t,Rd}$ [kN]	Utilisation [%]
158 kN	13	120 + 2×46.2	120 + 2×46.2	4.5	248	731	63.7
344 kN	10	120 + 2×30.8	120 + 2×30.8	8	399	1250	86.2
512 kN	7	140 + 2×30.8	140 + 2×30.8	8	625	1477	81.9
681 kN	4	140 + 2×30.8	140 + 2×30.8	10	745	1807	91.4
850 kN	1	150 + 2×30.8	150 + 2×30.8	10	905	1949	93.9

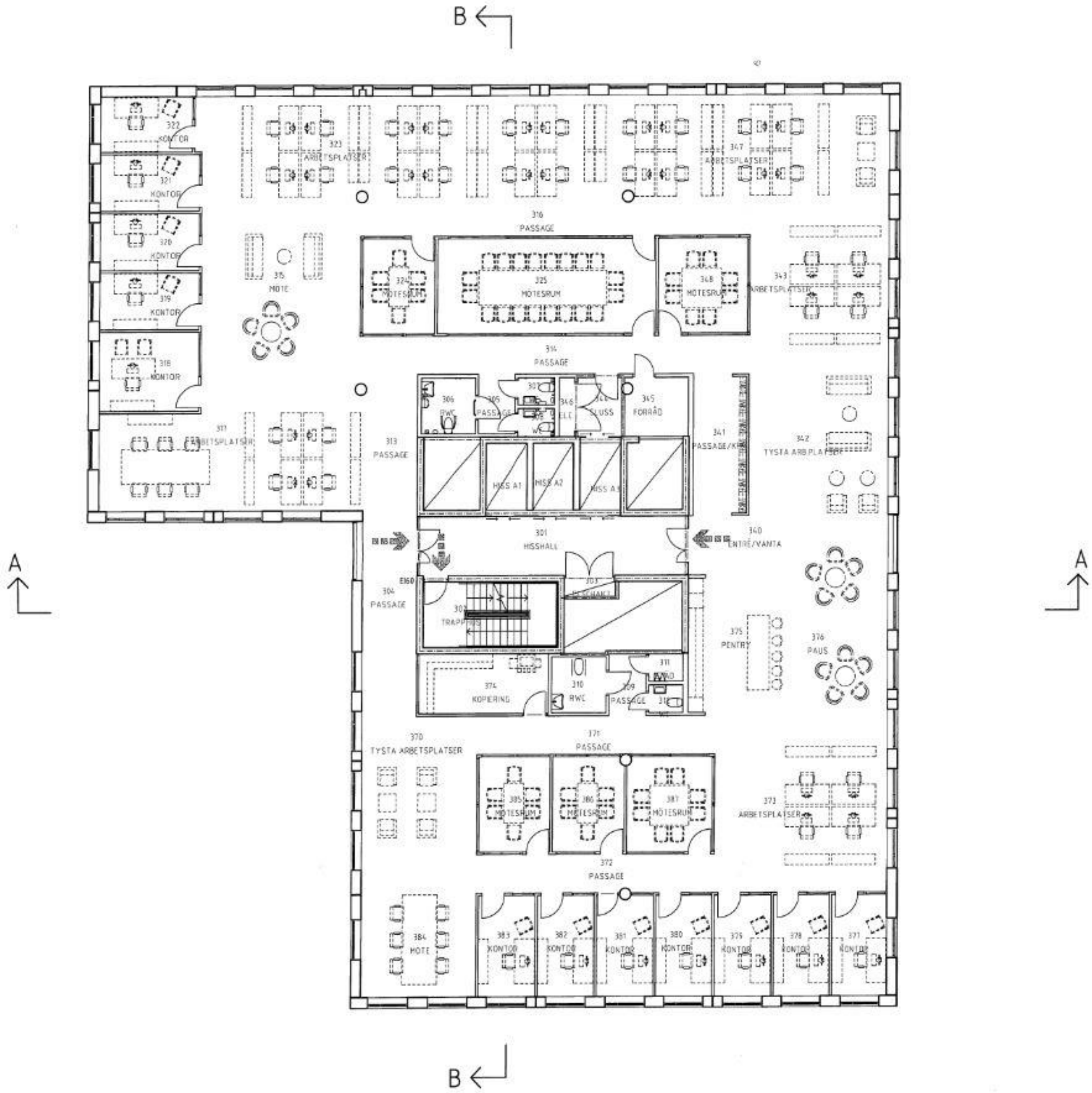
Table 37 Dimensions for KCKR chevron bracing with a buckling length of 4.7 metres. Complementary information to Table 16 in Chapter 5.

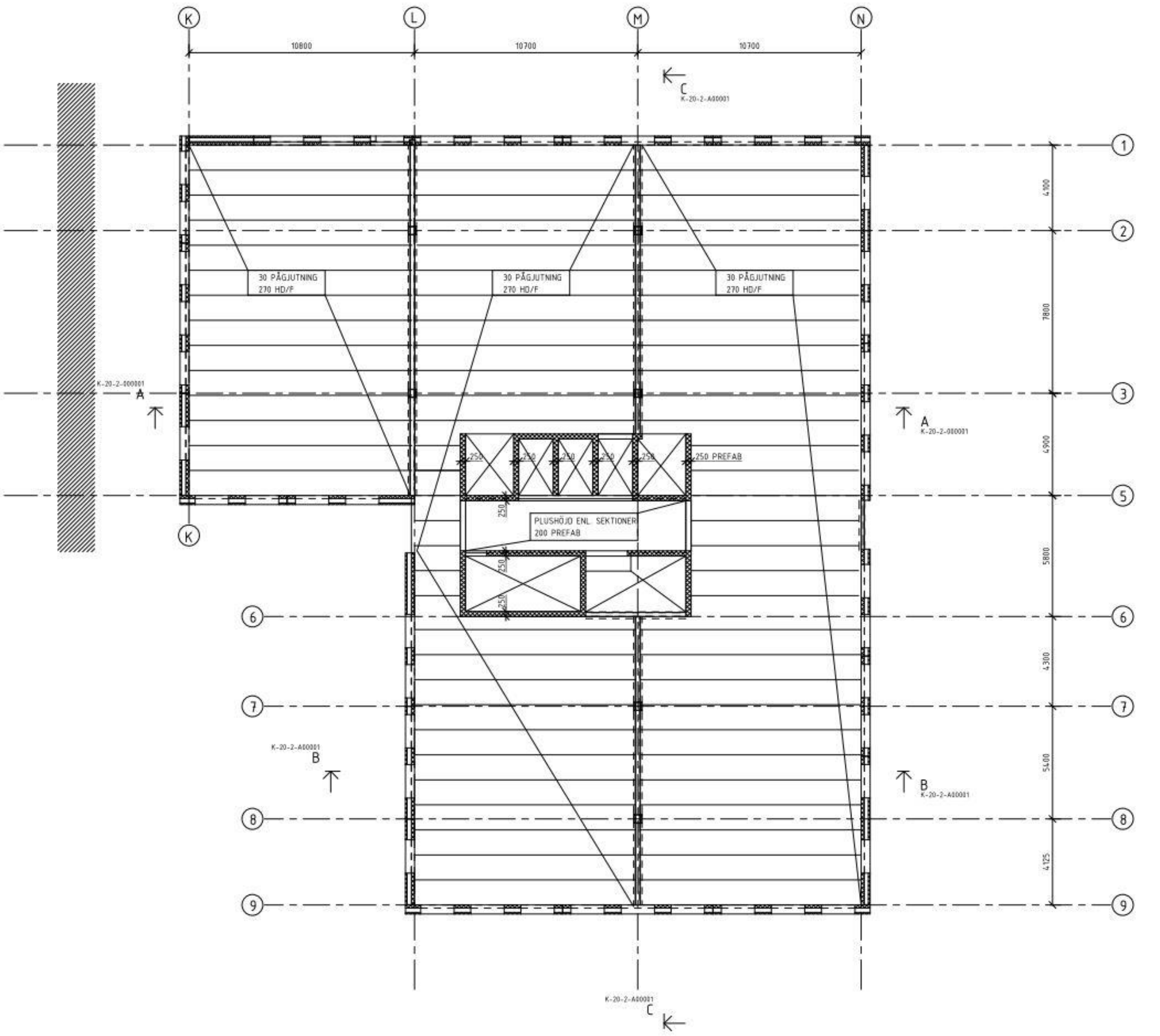
KCKR S355J2H (4.7 m)	Storey	Diameter [mm]	t [mm]	$N_{c.Rd}$ [kN]	$N_{t.Rd}$ [kN]	Utilisation [%]
192 kN	13	139.7 + 2×30.8	4	241	607	79.7
419 kN	10	139.7 + 2×30.8	8	449	1175	93.3
625 kN	7	168.3 + 2×30.8	8	706	1431	88.6
830 kN	4	193.7 + 2×30.8	8	958	1658	86.6
1036 kN	1	193.7 + 2×30.8	10	1172	2048	88.4

Table 38 Dimensions for KCKR chevron bracing with a buckling length of 5.7 metres. Complementary information to Table 17 in Chapter 5.

KCKR S355J2H (5.7 m)	Storey	Diameter [mm]	t [mm]	$N_{c.Rd}$ [kN]	$N_{t.Rd}$ [kN]	Utilisation [%]
158 kN	13	139.7 + 2×30.8	4	180	607	87.6
344 kN	10	168.3 + 2×30.8	6	424	1086	81.2
512 kN	7	168.3 + 2×30.8	8	549	1431	93.3
681 kN	4	193.7 + 2×30.8	8	773	1658	88.1
850 kN	1	193.7 + 2×30.8	10	944	2048	90.1

Appendix B: Drawings of the reference building

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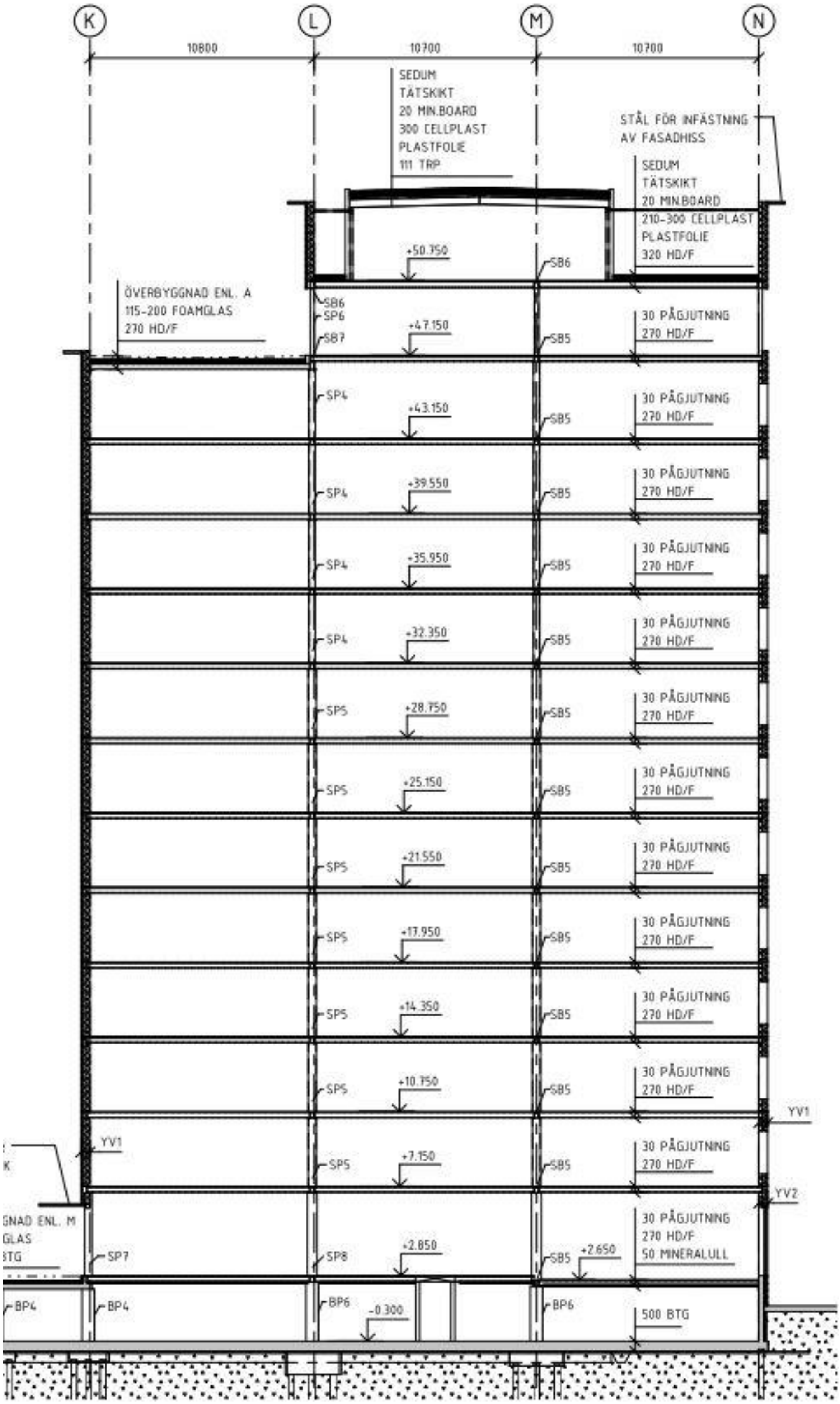
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STÅL FÖR INFÄSTNING
AV FASADHISS
SEDUM
TÄTSKIKT
20 MIN.BOARD
210-300 CELLPLAST
PLASTFOLIE
320 HD/F
+50.750
SB6
ÖVERBYGGNAD ENL. A
115-200 FOAMGLAS
270 HD/F
SB6
SP6
SB7
+47.150
SB5
30 PÅGJUTNING
270 HD/F
SP4
+43.150
SB5
+39.550
+35.950
+32.350
SP5
+28.750
+25.150
+21.550
+17.950
+14.350
+10.750
YV1
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YV2
SP7
SP8
+2.850
+2.650
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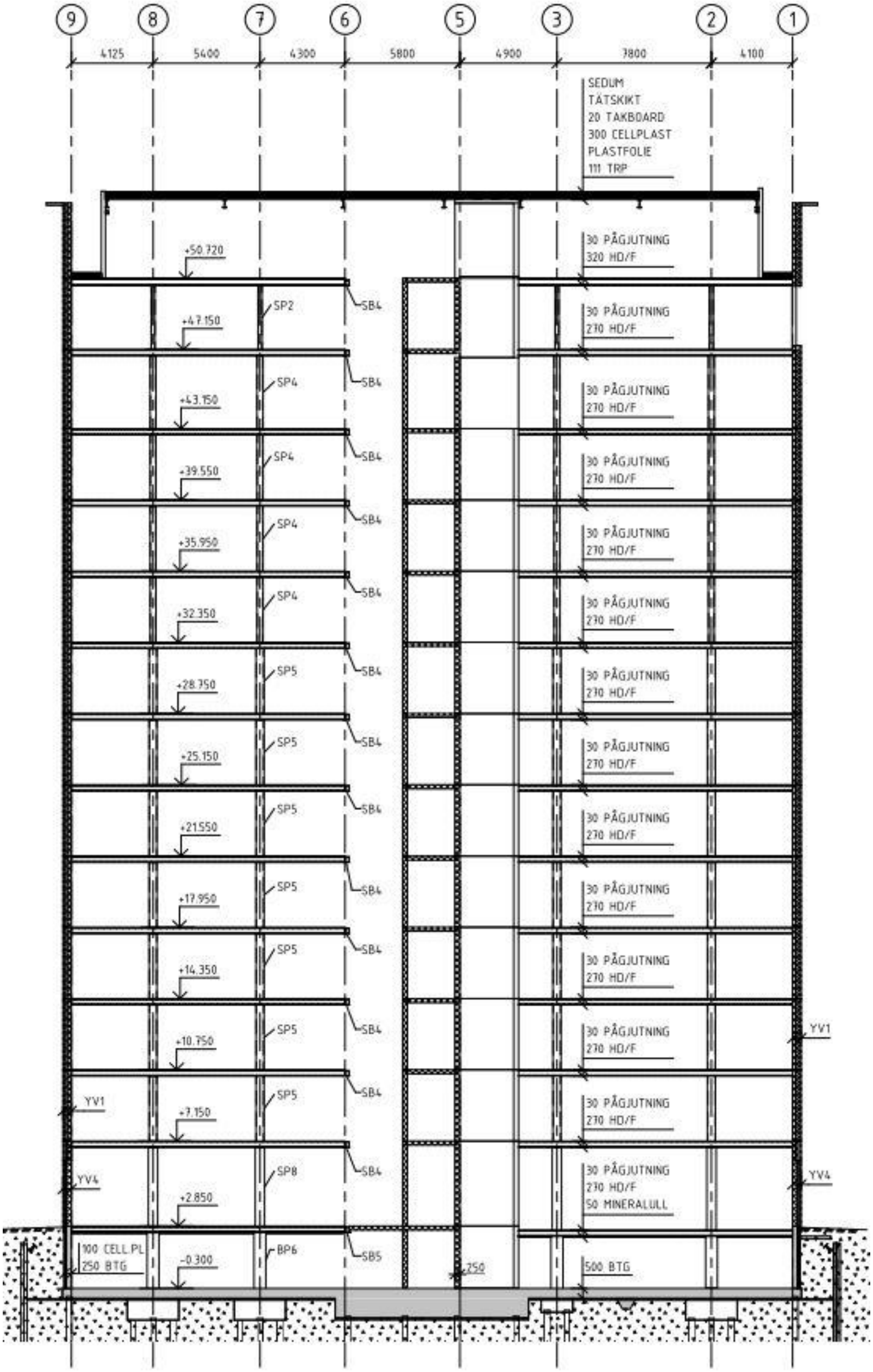
BP4
BP6
-0.300
500 BTG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6
5
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2
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7800
4100
SEDUM
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20 TAKBOARD
300 CELLPLAST
PLASTFOLIE
111 TRP
30 PÅGJUTNING
320 HD/F
+50.720
+47.150
+43.150
+39.550
+35.950
+32.350
+28.750
+25.150
+21.550
+17.950
+14.350
+10.750
+7.150
+2.850
-0.300
SP2
SP4
SP5
SP8
BP6
SB4
SB5
30 PÅGJUTNING
270 HD/F
50 MINERALULL
YV1
YV4
100 CELL.PL
250 BTG
250
500 BTG

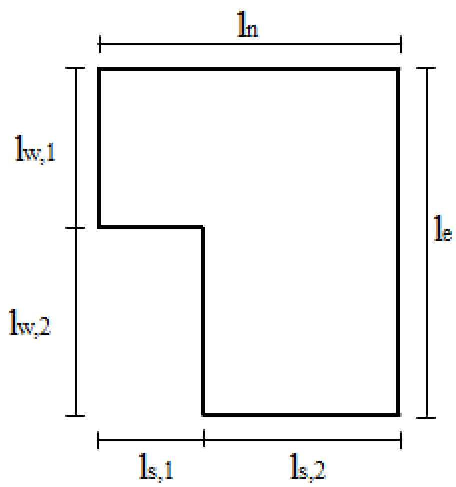
B:4

Appendix C1: Wind loads on the reference building

In this Appendix the calculations of the wind loads acting on the reference building are presented. These values are used for the development of structural systems. This part have been calculated according to **SS-EN 1991-1-4:2005**. Any references made refers to this code.

C1.1 Geometry & description of the terrain

Thickness of wall	$t_{wall} := 0.43m$
Lenght of north facade	$l_n := 32.2m + 2t_{wall} = 33.06\,m$
Lenght of east facade	$l_e := 36.425m + 2t_{wall} = 37.285\,m$
Lenght of south facade	$l_{s.1} := 10.8m$
	$l_{s.2} := 21.4m + 2t_{wall} = 22.26\,m$
Lenght of west facade	$l_{w.1} := 16.8m + 2t_{wall} = 17.66\,m$
	$l_{w.2} := 19.625m$
Total height of the building	$h_{tot} := 52.65m$

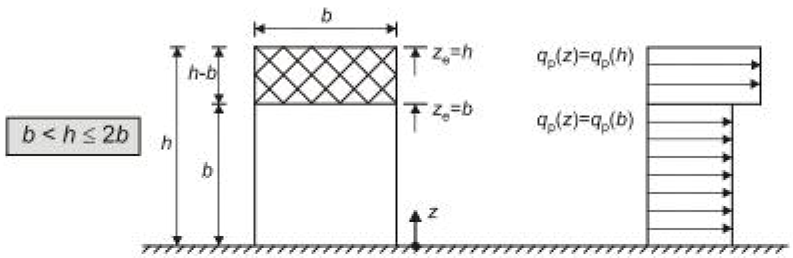


It is assumed that the effect from the wind is equally large on the west and east side. The same assumtion is made for the north and the south side.

According to **section 7.2.2** the wind load of the house should be divided into different zones. With this height to width ratio two zones is needed, see figure below.

$b_1 := l_n = 33.06\,m$ $\qquad$ $b_2 := l_e = 37.285\,m$

$b_1 < h_{tot} \le 2 \cdot b_1 = 1$ $\qquad$ $b_2 < h_{tot} \le 2 \cdot b_2 = 1$



Height of zones when wind from north

Height up to top of zone 1 $\quad z_{1.n} := h_{tot} = 52.65\,m$

Height up to top of zone 2 $\quad z_{2.n} := b_1 = 33.06\,m$

Height of zones when wind from east

Height up to top of zone 1 $\quad z_{1.e} := h_{tot} = 52.65\,m$

Height up to top of zone 2 $\quad z_{2.e} := b_2 = 37.285\,m$

Terrain type III is assumed, giving the following minimum and maximum heights of the building.

$z_{min} := 5m$ $\qquad$ $z_{min} < z_1 < z_{max}$ $\qquad$ Table 4.1 in section 4.3

$z_{max} := 200m$ $\qquad$ $z_{min} < z_2 < z_{max}$

C1.2 Basic wind velocity

$\boxed{v_b := c_{dir} \cdot c_{season} \cdot v_{b.0}}$ $\qquad$ eq. 4.1 in section 4.2

Direction factor $\quad c_{dir} := 1$ $\qquad$ Assumptions made according to EC1-4, notes in 4.2

Season factor $\quad c_{season} := 1$

Wind velocity in Göteborg $\quad v_{b.0} := 25\frac{m}{s}$

Basic wind velocity $\quad v_b := c_{dir} \cdot c_{season} \cdot v_{b.0} = 25\frac{m}{s}$

C1.3 Mean wind velocity

$$\boxed{v_m(z) = \blacksquare \cdot c_r(z) \cdot c_0(z) \cdot v_b}$$ <u>eq. 4.3 in section 4.3</u>

Terrain roughness factor, result from assumption of the terrain

$z_0 := 0.3m$ <u>Table 4.1 in section 4.3</u>

$z_{0.II} := 0.05m$

Terrain factor $k_r := 0.19 \cdot \left(\dfrac{z_0}{z_{0.II}}\right)^{0.07} = 0.215$

Mean wind velocity when wind from north

Roughness factor for zone 1 $c_{r.1.n} := k_r \cdot \ln\left(\dfrac{z_{1.n}}{z_0}\right) = 1.113$

Roughness factor for zone 2 $c_{r.2.n} := k_r \cdot \ln\left(\dfrac{z_{2.n}}{z_0}\right) = 1.013$

Orpograpgy factor $c_0 := 1$

Mean wind velocity zone 1 $v_{m.1.n} := c_{r.1.n} \cdot c_0 \cdot v_b = 27.826 \dfrac{m}{s}$

Mean wind velocity zone 2 $v_{m.2.n} := c_{r.2.n} \cdot c_0 \cdot v_b = 25.321 \dfrac{m}{s}$

Mean wind velocity when wind from east

Roughness factor for zone 1 $c_{r.1.e} := k_r \cdot \ln\left(\dfrac{z_{1.e}}{z_0}\right) = 1.113$

Roughness factor for zone 2 $c_{r.2.e} := k_r \cdot \ln\left(\dfrac{z_{2.e}}{z_0}\right) = 1.039$

Orpograpgy factor $c_0 = 1$

Mean wind velocity zone 1 $v_{m.1.e} := c_{r.1.e} \cdot c_0 \cdot v_b = 27.826 \dfrac{m}{s}$

Mean wind velocity zone 2 $v_{m.2.e} := c_{r.2.e} \cdot c_0 \cdot v_b = 25.968 \dfrac{m}{s}$

C1.4 Wind turbulence

Turbulence factor $k_l := 1$

Standard deviation of the turbulence $\sigma_v := k_r \cdot v_b \cdot k_l = 5.385 \frac{m}{s}$

Wind turbulence when wind from north

Wind turbulence zone 1 $l_{v.1.n} := \frac{\sigma_v}{v_{m.1.n}} = 0.194$ eq. 4.7 in section 4.4

Wind turbulence zone 2 $l_{v.2.n} := \frac{\sigma_v}{v_{m.2.n}} = 0.213$

Wind turbulence when wind from east

Wind turbulence zone 1 $l_{v.1.e} := \frac{\sigma_v}{v_{m.1.e}} = 0.194$ eq. 4.7 in section 4.4

Wind turbulence zone 2 $l_{v.2.e} := \frac{\sigma_v}{v_{m.2.e}} = 0.207$

C1.5 Peak velocity pressure

Air density $\rho := 1.25 \frac{kg}{m^3}$

Peak velocity is calculated as: $\boxed{q_p(z) = \blacksquare \cdot \left(1 + 7 \cdot I_v(z)\right) \cdot \frac{1}{2} \rho \cdot v_m(z)^2}$ eq. 4.8 in section 4.5

Peak velocity pressure when wind from north

Peak velocity pressure zone 1 $q_{p.1.n} := \left(1 + 7 \cdot l_{v.1.n}\right) \cdot 0.5 \rho \cdot v_{m.1.n}{}^2 = 1.139 \times 10^3 \cdot Pa$

Peak velocity pressure zone 2 $q_{p.2.n} := \left(1 + 7 \cdot l_{v.2.n}\right) \cdot 0.5 \rho \cdot v_{m.2.n}{}^2 = 997.217 \cdot Pa$

Peak velocity pressure when wind from east

Peak velocity pressure zone 1 $q_{p.1.e} := \left(1 + 7 \cdot l_{v.1.e}\right) \cdot 0.5 \rho \cdot v_{m.1.e}{}^2 = 1.139 \times 10^3 \cdot Pa$

Peak velocity pressure zone 2 $q_{p.2.e} := \left(1 + 7 \cdot l_{v.2.e}\right) \cdot 0.5 \rho \cdot v_{m.2.e}{}^2 = 1.033 \times 10^3 \cdot Pa$

C1.6 Peak velocity pressure

Wind pressure is calculated as: $\boxed{w := q_p \cdot c_p}$ eq. 5.1 in section 5.2

Wind from east

$e_e := \min(l_e, 2 \cdot h_{tot}) = 37.285\,m$

$d_e := l_n = 33.06\,m$

$d_e < e_e < 5d_e = 1$

$ratio_e := \dfrac{h_{tot}}{d_e} = 1.593$

Wind from north

$e_n := \min(l_n, 2 \cdot h_{tot}) = 33.06\,m$

$d_n := l_e = 37.285\,m$

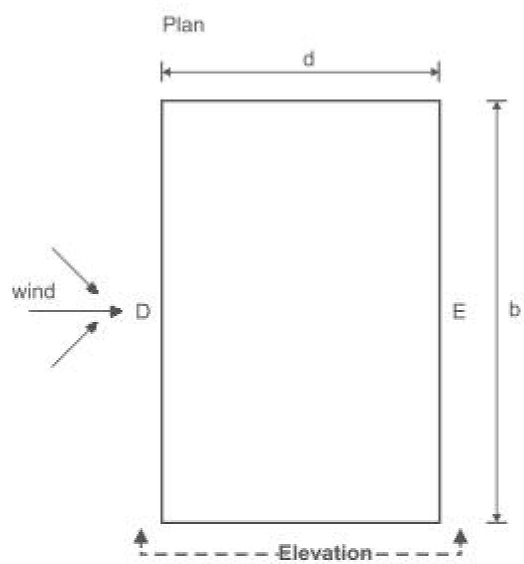
$d_e < e_e < 5d_e = 1$

$ratio_n := \dfrac{h_{tot}}{d_n} = 1.412$

Form factors for windward side (same for east and north) $C_{pe.10.D} := 0.8$

Form factors for leeward side when wind from east $C_{pe.10.E.e} := -0.5 + (-0.7 + 0.5) \cdot \dfrac{(ratio_e - 1)}{5 - 1} = -0.53$

Form factors for leeward side when wind from north $C_{pe.10.E.n} := -0.5 + (-0.7 + 0.5) \cdot \dfrac{(ratio_n - 1)}{5 - 1} = -0.521$



C1.6.1 Total wind pressure

Wind pressure on zone 1

Wind from east $w_{e.1.D.e} := q_{p.1.e} \cdot C_{pe.10.D} = 911.584\,Pa$

$w_{e.1.E.e} := q_{p.1.e} \cdot C_{pe.10.E.e} = -603.501\,Pa$

Wind from north

$w_{e.1.D.n} := q_{p.1.n} \cdot C_{pe.10.D} = 911.584\,Pa$

$w_{e.1.E.n} := q_{p.1.n} \cdot C_{pe.10.E.n} = -593.219\,Pa$

Wind pressure on zone 2

Wind from east

$w_{e.2.D.e} := q_{p.2.e} \cdot C_{pe.10.D} = 826.586\,Pa$

$w_{e.2.E.e} := q_{p.2.e} \cdot C_{pe.10.E.e} = -547.229\,Pa$

Wind from north

$w_{e.2.D.n} := q_{p.2.n} \cdot C_{pe.10.D} = 797.773\,Pa$

$w_{e.2.E.n} := q_{p.2.n} \cdot C_{pe.10.E.n} = -519.156\,Pa$

Total wind pressure when wind from east

Total wind pressure on zone 1 $w_{1.e} := w_{e.1.D.e} - w_{e.1.E.e} = 1.515 \times 10^3 \cdot Pa$

Total wind pressure on zone 2 $w_{2.e} := w_{e.2.D.e} - w_{e.2.E.e} = 1.374 \times 10^3\,Pa$

Total wind pressure when wind from north

Total wind pressure on zone 1 $w_{1.n} := w_{e.1.D.n} - w_{e.1.E.n} = 1.505 \times 10^3\,Pa$

Total wind pressure on zone 2 $w_{2.n} := w_{e.2.D.n} - w_{e.2.E.n} = 1.317 \times 10^3\,Pa$

C1.6.2 Distributed wind load

Influencing height of each storey $h_1 := 2.15m$

$h_2 := 2.15m + 1.8m = 3.95\,m$

$h_3 := 3.6m$

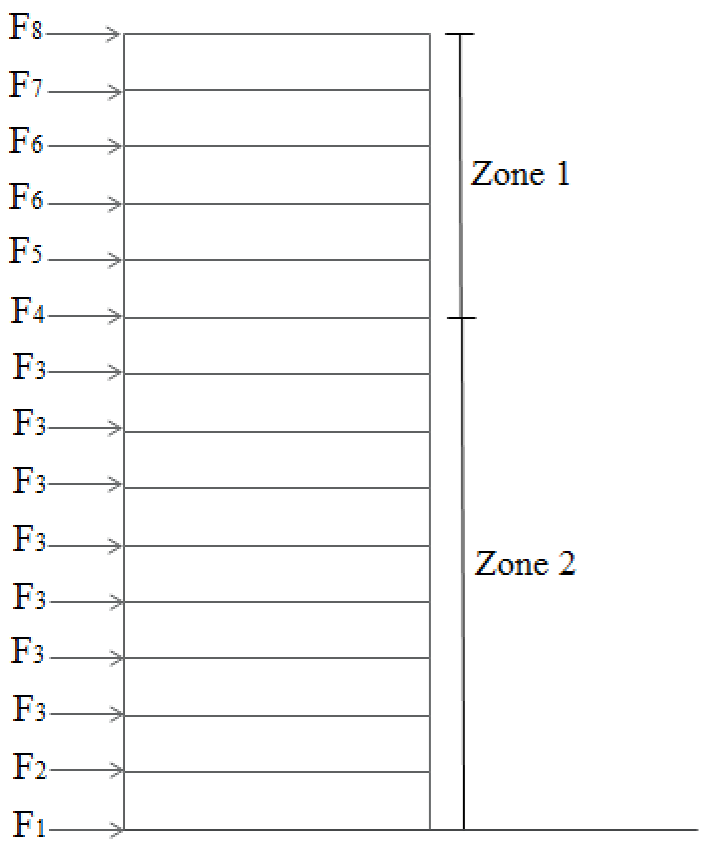
$h_4 := 2m + 1.8m$

$h_5 := 2.375m + 1.8m$

$h_6 := 2.375m$

F.0 is not set to zero but the level arm is zero so this force will not have any contribution to bracing members or stabilising walls.

C1.6.2.1 Wind force



Wind from east

$$F_{1.e} := h_1 \cdot w_{2.e} = 2.954 \cdot \frac{kN}{m}$$

$$F_{2.e} := h_2 \cdot w_{2.e} = 5.427 \cdot \frac{kN}{m}$$

$$F_{3.e} := h_3 \cdot w_{2.e} = 4.946 \cdot \frac{kN}{m}$$

$$F_{4.e} := \frac{h_3}{2} \cdot w_{2.e} + \frac{h_3}{2} \cdot w_{1.e} = 5.2 \cdot \frac{kN}{m}$$

Wind from north

$$F_{1.n} := h_1 \cdot w_{2.n} = 2.831 \cdot \frac{kN}{m}$$

$$F_{2.n} := h_2 \cdot w_{2.n} = 5.202 \cdot \frac{kN}{m}$$

$$F_{3.n} := h_3 \cdot w_{2.n} = 4.741 \cdot \frac{kN}{m}$$

$$F_{4.n} := \frac{h_3}{2} \cdot w_{2.n} + \frac{h_3}{2} \cdot w_{1.e} = 5.098 \cdot \frac{kN}{m}$$

$$F_{5.e} := h_3 \cdot w_{1.e} = 5.454 \cdot \frac{kN}{m}$$

$$F_{5.n} := h_3 \cdot w_{1.n} = 5.417 \cdot \frac{kN}{m}$$

$$F_{6.e} := h_4 \cdot w_{1.e} = 5.757 \cdot \frac{kN}{m}$$

$$F_{6.n} := h_4 \cdot w_{1.n} = 5.718 \cdot \frac{kN}{m}$$

$$F_{7.e} := h_5 \cdot w_{1.e} = 6.325 \cdot \frac{kN}{m}$$

$$F_{7.n} := h_5 \cdot w_{1.n} = 6.283 \cdot \frac{kN}{m}$$

$$F_{8.e} := h_6 \cdot w_{1.e} = 3.598 \cdot \frac{kN}{m}$$

$$F_{8.n} := h_6 \cdot w_{1.n} = 3.574 \cdot \frac{kN}{m}$$

Appendix C2: Load combinations used in the development of the concepts

This appendix was used to calculate the applied loads for different parts for the concepts. All concepts differs from each other and this Appendix only shows how the calculations are performed principally. Tributary areas changes between different concepts. This document was used in a combination with Appendix A1- A2 and A4. The obtained loads where then used to obtain dimensions for different members in the concepts, see Chapter 6 for the dimensions of the conepts.

C2.1 Snow loads

C2.1.1 Snow on the roof

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

Shape coefficient
Inclination less than 30 deg $\mu := 0.8$

Characteristic snowload on roof $s_{roof} := s_k \cdot \mu = 1.2 \cdot \frac{kN}{m^2}$

C2.1.2 Snow on the balcony

Shape coefficient $\mu_b := \mu_s + \mu_w$

Shape coefficient due to sliding of snow from roof. $\mu_s := 0$ (since the inclination is less than 15 deg)

Width of the roof $b_r := 22.26m$

Width of the balcony $b_b := 10.8m$

Height to the top of the roof from the balcony $h := 8.35m$

Snow density $\gamma_{snow} := 2 \frac{kN}{m^2}$

Shape coefficient due to wind $\mu_w := \frac{b_r + b_b}{2 \cdot h} = 1.98$ < $\gamma_{snow} \cdot \frac{\frac{h}{m}}{s_k} = 11.133$

Shape coefficient $\mu_b := \mu_s + \mu_w = 1.98$

Drift length $l_s := 10m$

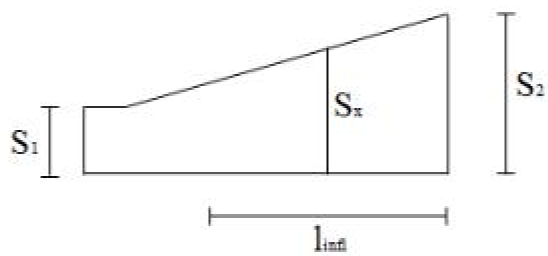
Characteristic snow load on the balcony $s_1 := \mu \cdot s_k = 1.2 \cdot \frac{kN}{m^2}$

$s_2 := \mu_b \cdot s_k = 2.969 \cdot \frac{kN}{m^2}$

Influence length for the snow load on the balcony beam $l_{snow.b.beam} := 6.563m$

Characteristic snow at load dividing line $s_x := (s_2 - s_1) \cdot \frac{(l_s - l_{snow.b.beam})}{l_s} + s_1 = 1.808 \cdot \frac{kN}{m^2}$

Mean value for the snow load on the balcony beam $s_b := \frac{s_1 + s_x}{2} = 1.504 \cdot \frac{kN}{m^2}$



C2.2 Self-weight for the roof

Hollow core slab $g_{r.slab} := 4 \frac{kN}{m^2}$

Weight of the sedum $g_{r.sedum} := 0.5 \frac{kN}{m^2}$

Insulation, cellualar plastic $g_{r.ins} := 0.06 \frac{kN}{m^2}$

Total self-weight $g_{roof} := g_{r.slab} + g_{r.sedum} + g_{r.ins} = 4.56 \cdot \frac{kN}{m^2}$

C2.3 Load combinations for beams

Factor for snow $\psi_{0.snow} := 0.7$ $\psi_{1.snow} := 0.5$ $\psi_{2.snow} := 0.2$

Influence length $l_{inf.r} := 7.675m$

C2.3.1 Roof beams

6.10a (used when calculating the rest of the house, because snow load is not considered as main load)

$$Q_{a.r} := \left(1.35 \cdot g_{roof} + 1.5 \cdot \psi_{0.snow} \cdot s_{roof}\right) \cdot l_{inf.r}$$

6.10b

$$Q_{b.r} := \left(1.35 \cdot 0.89 \cdot g_{roof} + 1.5 \cdot s_{roof}\right) \cdot l_{inf.r}$$

ULS

$$Q_{roof} := \max\left(Q_{a.r}, Q_{b.r}\right) = 56.918 \cdot \frac{kN}{m}$$

FIRE

$$Q_{roof.fire} := \left(g_{roof} + \psi_{1.snow} \cdot s_k\right) \cdot l_{inf.r} = 40.754 \cdot \frac{kN}{m}$$

C2.3.2 Balcony beams

Imposed load for a balcony

$$q_{bal} := 5 \cdot \frac{kN}{m^2} \qquad \psi_{0.bal} := 0.7 \qquad \psi_{1.bal} := 0.7$$

Assumed hollow core slab and insulation

$$g_{bal} := g_{r.slab} + g_{r.ins} = 4.06 \cdot \frac{kN}{m^2}$$

Influence length

$$l_{inf.b} := 7.675m$$

6.10a

$$Q_{a.b} := \left(1.35 \cdot g_{bal} + 1.5 \psi_{0.bal} \cdot q_{bal} + 1.5 \cdot \psi_{0.snow} \cdot s_b\right) \cdot l_{inf.r}$$

6.10b

$$Q_{b.b} := \left(1.35 \cdot 0.89 \cdot g_{bal} + 1.5 \cdot q_{bal} + 1.5 \cdot \psi_{0.snow} \cdot s_b\right) \cdot l_{inf.r}$$

ULS

$$Q_{bal} := \max\left(Q_{a.b}, Q_{b.b}\right) = 107.123 \cdot \frac{kN}{m}$$

FIRE

$$Q_{bal.fire} := \left(g_{bal} + \psi_{1.bal} \cdot q_{bal} + \psi_{2.snow} \cdot s_b\right) \cdot l_{inf.r} = 60.332 \cdot \frac{kN}{m}$$

C2.3.3 Beams for office floor

Cassette floors self-weight

$$g_{slab} := 1 \frac{kN}{m^2}$$

Office load

$$q_{office} := 2.5 \frac{kN}{m^2} \qquad \psi_{0.office} := 0.7 \qquad \psi_{1.office} := 0.5$$

Installations

$$g_{ins} := 0.3 \frac{kN}{m^2}$$

Partition walls $q_{part} := 0.5 \dfrac{kN}{m^2}$ $\psi_{0.part} := 0.7$ $\psi_{1.part} := 0.5$ $\psi_{2.part} := 0.3$

Influence length $l_{inf.f} := 7.675m$

6.10a $Q_{a.f} := \left[1.35 \cdot \left(g_{slab} + g_{ins}\right) + 1.5\psi_{0.office} \cdot q_{office} + 1.5 \cdot \psi_{0.part} q_{part}\right] \cdot l_{inf.f}$

6.10b $Q_{b.f} := \left[1.35 \cdot 0.89 \cdot \left(g_{slab} + g_{ins}\right) + 1.5 \cdot q_{office} + 1.5 \cdot \psi_{0.part} q_{part}\right] \cdot l_{inf.f}$

ULS $Q_f := \max\left(Q_{a.f}, Q_{b.f}\right) = 44.799 \cdot \dfrac{kN}{m}$

FIRE $Q_{f.fire} := \left(g_{slab} + g_{ins} + \psi_{1.office} \cdot q_{office} + \psi_{2.part} \cdot q_{part}\right) \cdot l_{inf.f} = 20.723 \cdot \dfrac{kN}{m}$

C2.4 Load combinations for columns

Office load is the main imposed load.

Tributary area (Changed for the different Concepts) $A_{trib} := 48.736m^2$

Roof self-weight $g_{roof} = 4.56 \cdot \dfrac{kN}{m^2}$

Slab self weight $g_{slab} = 1 \cdot \dfrac{kN}{m^2}$

Installations $g_{ins} = 0.3 \cdot \dfrac{kN}{m^2}$

Imposed load (both office and installation floor) $q_{office} = 2.5 \cdot \dfrac{kN}{m^2}$

Partition walls $q_{part} = 0.5 \cdot \dfrac{kN}{m^2}$

Snow load $s_{roof} = 1.2 \cdot \dfrac{kN}{m^2}$

C2.4.1 For columns on floor 11 and above

Floors above the column $n_{11} := 3$

6.10a

$$Q_{a.c11} := \left[\begin{array}{l} 1.35\cdot\left[g_{roof} + n_{11}\cdot\left(g_{slab} + g_{ins}\right)\right] \cdots \\ + n_{11}\cdot\left(1.5\cdot\psi_{0.office}q_{office} + 1.5\cdot\psi_{0.part}q_{part}\right) + 1.5\cdot\psi_{0.snow}s_{roof} \end{array}\right]\cdot A_{trib} = 1.079\cdot MN$$

6.10b

$$Q_{b.c11} := \left[\begin{array}{l} 1.35\cdot 0.89\cdot\left[g_{roof} + n_{11}\cdot\left(g_{slab} + g_{ins}\right)\right] \cdots \\ + n_{11}\cdot\left(1.5q_{office} + 1.5\cdot\psi_{0.part}q_{part}\right) + 1.5\cdot\psi_{0.snow}s_{roof} \end{array}\right]\cdot A_{trib} = 1.182\cdot MN$$

ULS

$$Q_{c.11} := \max\left(Q_{a.c11}, Q_{b.c11}\right) = 1.182\cdot MN$$

FIRE

$$Q_{fire.c11} := \left[\begin{array}{l} g_{roof} + n_{11}\cdot\left(g_{slab} + g_{ins}\right) \cdots \\ + n_{11}\cdot\left(\psi_{1.office}q_{office} + 1.5\cdot\psi_{2.part}q_{part}\right) + \psi_{2.snow}s_{roof} \end{array}\right]\cdot A_{trib} = 0.64\cdot MN$$

C2.4.2 For columns on floor 6 to 10

Floors above the column $n_6 := 8$

Area for columns above floor 10 $A_{11} := 0.174m^2$

Density for glulam $\rho_{glulam} := 0.43\frac{kN}{m^3}$

Height of columns $h_{col} := 3.6m$

A columns self-weight
(If steel column is used, self weight from Tibnor was used) $g_{col} := A_{11}\cdot\rho_{glulam}\cdot h_{col} = 2.694\times 10^{-4}\cdot MN$

Cross section for the beams $A_{beam} := 0.1463m^2$

Density for LVL $\rho_{lvl} := 0.51\frac{kN}{m^3}$

Influence length from the beams $l_{inf.beam} := 6.91m$

A beams self-weight $g_{beam} := A_{beam} \cdot l_{inf.beam} \cdot \rho_{lvl} = 0.516 \cdot kN$

Office beam 2
HEA500 $g_{beam} := 155 \dfrac{kg}{m} \cdot g \cdot l_{inf.beam} = 10.503 \cdot kN$

6.10a

$$Q_{a.c6} := \begin{bmatrix} 1.35 \cdot \left[g_{roof} + n_6 \cdot \left(g_{slab} + g_{ins} \right) \right] \dots \\ + n_6 \cdot \left(1.5 \cdot \psi_{0.office} q_{office} + 1.5 \cdot \psi_{0.part} q_{part} \right) + 1.5 \cdot \psi_{0.snow} s_{roof} \end{bmatrix} \cdot A_{trib} \dots = 2.39 \cdot MN$$
$$+ 1.35 n_6 \cdot \left(g_{beam} + g_{col} \right)$$

6.10b

$$Q_{b.c6} := \begin{bmatrix} 1.35 \cdot 0.89 \cdot \left[g_{roof} + n_6 \cdot \left(g_{slab} + g_{ins} \right) \right] \dots \\ + n_6 \cdot \left(1.5 q_{office} + 1.5 \cdot \psi_{0.part} q_{part} \right) + 1.5 \cdot \psi_{0.snow} s_{roof} \end{bmatrix} \cdot A_{trib} \dots = 2.708 \cdot MN$$
$$+ 1.35 \cdot 0.89 n_6 \cdot \left(g_{beam} + g_{col} \right)$$

ULS

$$Q_{c.6} := \max\left(Q_{a.c6}, Q_{b.c6} \right) = 2.708 \times 10^6 N$$

FIRE

$$Q_{fire.c6} := \begin{bmatrix} g_{roof} + n_6 \cdot \left(g_{slab} + g_{ins} \right) \dots \\ + n_6 \cdot \left(\psi_{1.office} q_{office} + 1.5 \cdot \psi_{2.part} q_{part} \right) + \psi_{2.snow} s_{roof} \end{bmatrix} \cdot A_{trib} \dots = 1.402 \cdot MN$$
$$+ n_6 \cdot \left(g_{beam} + g_{col} \right)$$

C2.4.3 For columns on floor 2 to 5

Floors above the
column $n_2 := 12$

"Area" for columns above floor 2 $A_6 := 0.2322 m^2$

A columns self-weight
(If steel, self-weight from Tibnors tables) $g_{col} := A_6 \cdot \rho_{glulam} \cdot h_{col} = 3.594 \times 10^{-4} \cdot MN$

6.10a

$$Q_{a.c2} := \begin{bmatrix} 1.35 \cdot \left[g_{roof} + n_2 \cdot \left(g_{slab} + g_{ins} \right) \right] \dots \\ + n_2 \cdot \left(1.5 \cdot \psi_{0.office} q_{office} + 1.5 \cdot \psi_{0.part} q_{part} \right) + 1.5 \cdot \psi_{0.snow} s_{roof} \end{bmatrix} \cdot A_{trib} \dots = 3.406 \cdot MN$$
$$+ 1.35 n_2 \cdot \left(g_{beam} + g_{col} \right)$$

6.10b

$$Q_{b.c2} := \left[\begin{array}{l} 1.35 \cdot 0.89 \cdot \left[g_{roof} + n_2 \cdot \left(g_{slab} + g_{ins} \right) \right] \cdots \\ + n_2 \cdot \left(1.5 q_{office} + 1.5 \cdot \psi_{0.part} q_{part} \right) + 1.5 \cdot \psi_{0.snow} s_{roof} \end{array} \right] \cdot A_{trib} \cdots = 3.899 \cdot MN$$
$$+ 1.35 \cdot 0.89 n_2 \cdot \left(g_{beam} + g_{col} \right)$$

ULS

$$Q_{c.2} := \max\left(Q_{a.c2}, Q_{b.c2} \right) = 3.899 \cdot MN$$

FIRE

$$Q_{fire.c2} := \left[\begin{array}{l} g_{roof} + n_2 \cdot \left(g_{slab} + g_{ins} \right) \cdots \\ + n_2 \cdot \left(\psi_{1.office} q_{office} + 1.5 \cdot \psi_{2.part} q_{part} \right) + \psi_{2.snow} s_{roof} \end{array} \right] \cdot A_{trib} \cdots = 1.987 \cdot MN$$
$$+ n_2 \cdot \left(g_{beam} + g_{col} \right)$$

C2.5 Load combinations for walls

C2.5.1 For walls on floor 11 and above

Influence length $$l_{inf.wall} := \frac{l_{inf.f}}{2} = 3.838 \, m$$

Self-weight of a wall, taken from Appendix A4: Wall calculations $$g_{w11} := 3.986 kN + 2.879 kN = 6.865 \cdot kN$$

6.10a

$$G_{a.w.11} := \left[1.35 \cdot \left[g_{roof} + n_{11} \cdot \left(g_{slab} + g_{ins} \right) \right] \right] \cdot l_{inf.wall} \cdot 2.4m + 1.35 \cdot g_{w11} \cdot n_{11} = 132.991 \cdot kN$$

$$Q_{a.w11} := \left[n_{11} \cdot \left(1.5 \cdot \psi_{0.office} q_{office} + 1.5 \cdot \psi_{0.part} q_{part} \right) + 1.5 \cdot \psi_{0.snow} s_{roof} \right] \cdot l_{inf.wall} \cdot 2.4m = 98.639 \cdot kN$$

Remember the wind!

6.10b

$$G_{b.w.11} := \left[1.35 \cdot 0.89 \cdot \left[g_{roof} + n_{11} \cdot \left(g_{slab} + g_{ins} \right) \right] \right] \cdot l_{inf.wall} \cdot 2.4m + 1.35 \cdot 0.89 \cdot n_{11} \cdot g_{w11} = 118.362 \, m \cdot \frac{kN}{m}$$

$$Q_{b.w11} := \left[n_{11} \cdot \left(1.5 q_{office} + 1.5 \cdot \psi_{0.part} q_{part} \right) + 1.5 \cdot \psi_{0.snow} s_{roof} \right] \cdot l_{inf.wall} \cdot 2.4m = 129.723 \, m \cdot \frac{kN}{m}$$

$$H_{bw11} := 3.283 \frac{kN}{m}$$

FIRE

$$G_{fire.w11} := \left[g_{roof} + n_{11}\cdot\left(g_{slab} + g_{ins}\right)\right]\cdot l_{inf.wall} = 32.465\cdot\frac{kN}{m}$$

$$Q_{fire.w11} := \left[n_{11}\cdot\left(\psi_{1.office}\,q_{office} + \psi_{2.part}\,q_{part}\right) + \psi_{2.snow}\,s_{roof}\right]\cdot l_{inf.wall} = 17.038\cdot\frac{kN}{m}$$

C2.5.2 For walls on floor 6 to 10

6.10a

$$G_{a.w.6} := \left[1.35\cdot\left[g_{roof} + n_{6}\cdot\left(g_{slab} + g_{ins}\right)\right]\right]\cdot l_{inf.wall} = 77.502\cdot\frac{kN}{m}$$

$$Q_{a.w.6} := \left[n_{6}\cdot\left(1.5\cdot\psi_{0.office}\,q_{office} + 1.5\cdot\psi_{0.part}\,q_{part}\right) + 1.5\cdot\psi_{0.snow}\,s_{roof}\right]\cdot l_{inf.wall} = 101.54\cdot\frac{kN}{m}$$

6.10b

$$G_{b.w.16} := \left[1.35\cdot 0.89\cdot\left[g_{roof} + n_{6}\cdot\left(g_{slab} + g_{ins}\right)\right]\right]\cdot l_{inf.wall} = 68.977\cdot\frac{kN}{m}$$

$$Q_{b.w.6} := \left[n_{6}\cdot\left(1.5q_{office} + 1.5\cdot\psi_{0.part}\,q_{part}\right) + 1.5\cdot\psi_{0.snow}\,s_{roof}\right]\cdot l_{inf.wall} = 136.078\cdot\frac{kN}{m}$$

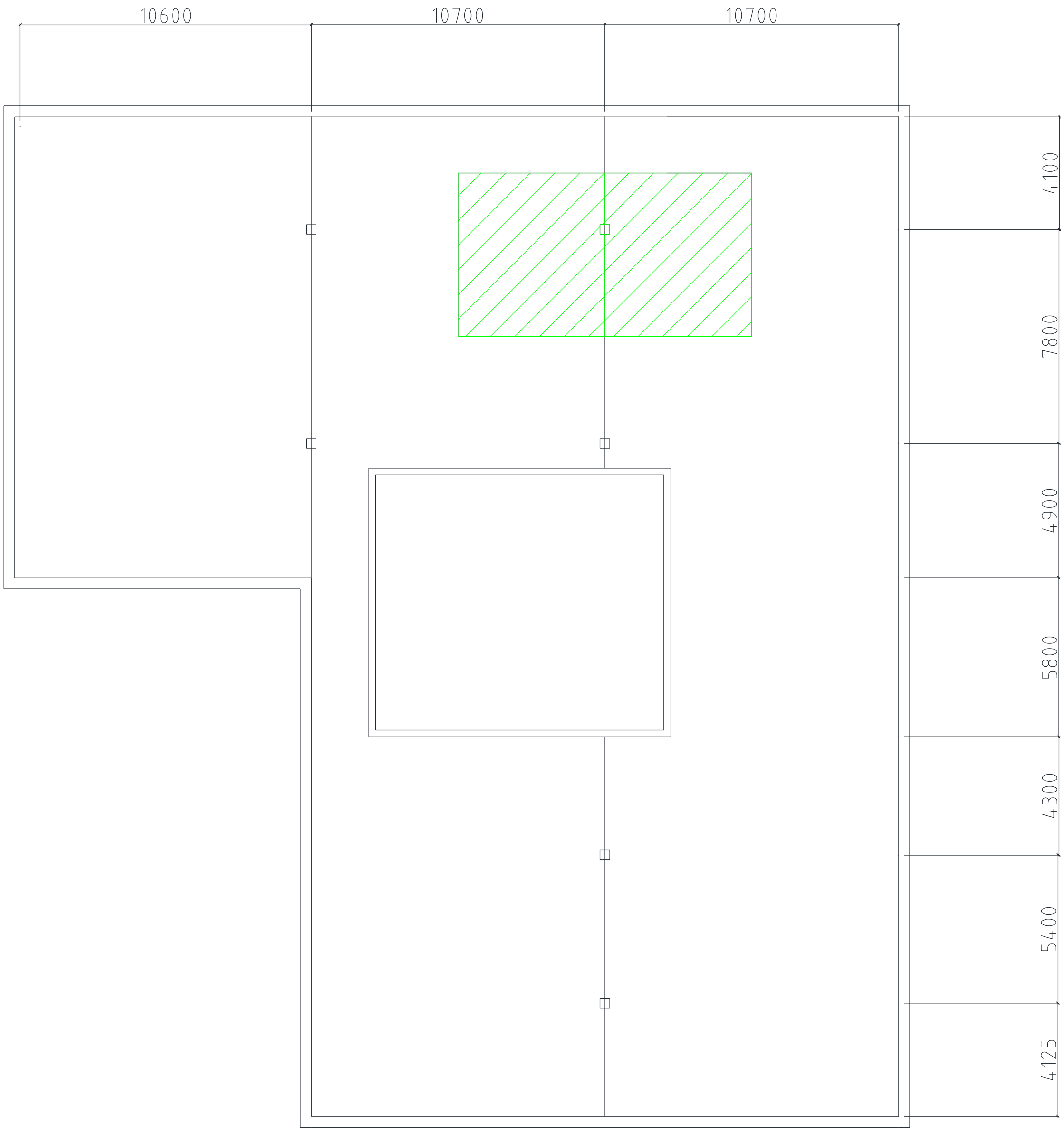
FIRE

$$G_{fire.w.6} := \left[g_{roof} + n_{6}\cdot\left(g_{slab} + g_{ins}\right)\right]\cdot l_{inf.wall} = 57.409\cdot\frac{kN}{m}$$

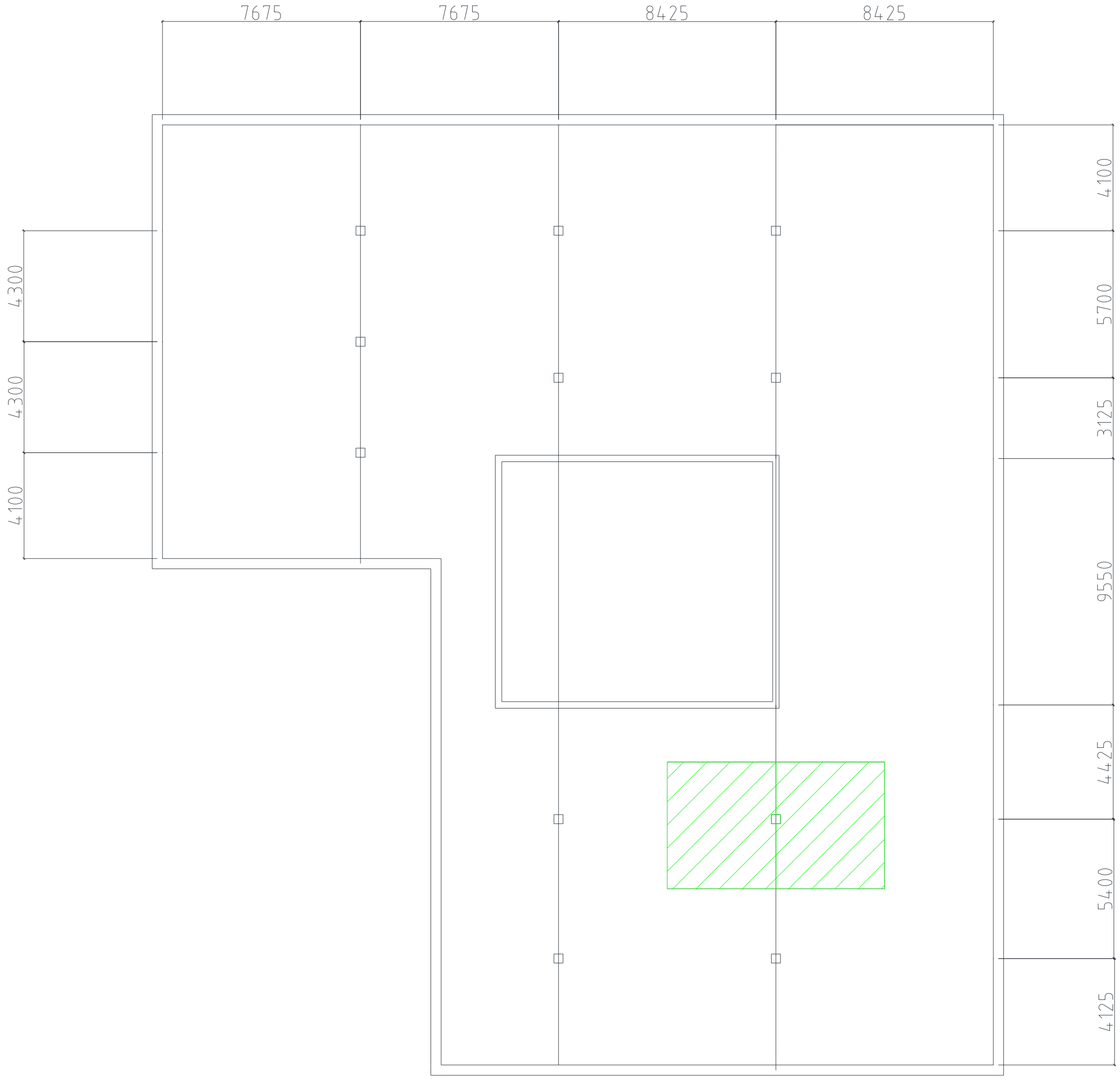
$$Q_{fire.w.6} := \left[n_{6}\cdot\left(\psi_{1.office}\,q_{office} + \psi_{2.part}\,q_{part}\right) + \psi_{2.snow}\,s_{roof}\right]\cdot l_{inf.wall} = 43.901\cdot\frac{kN}{m}$$

Appendix C3: Drawings of the reference building and the concepts

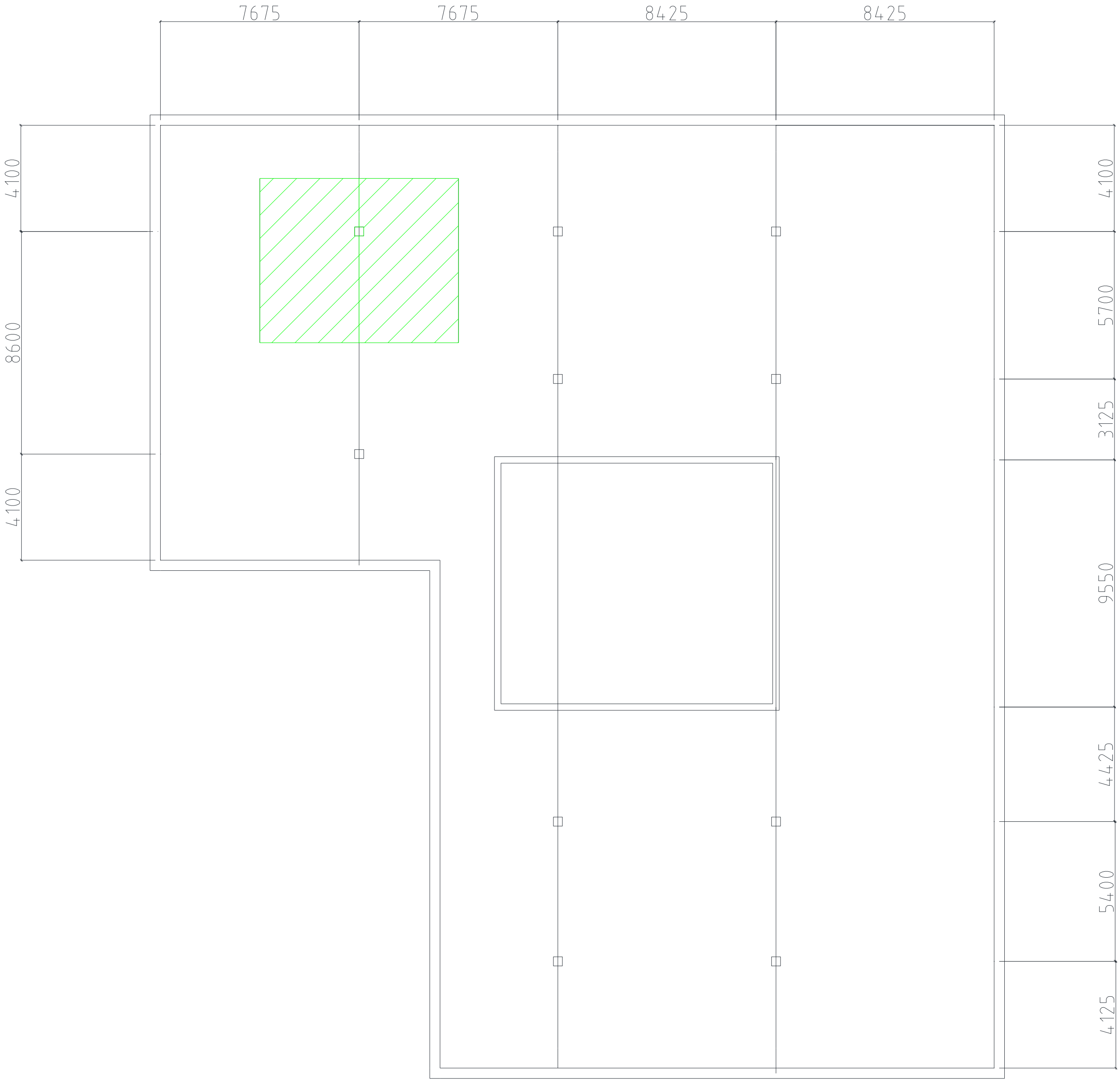
C3.1 Reference building



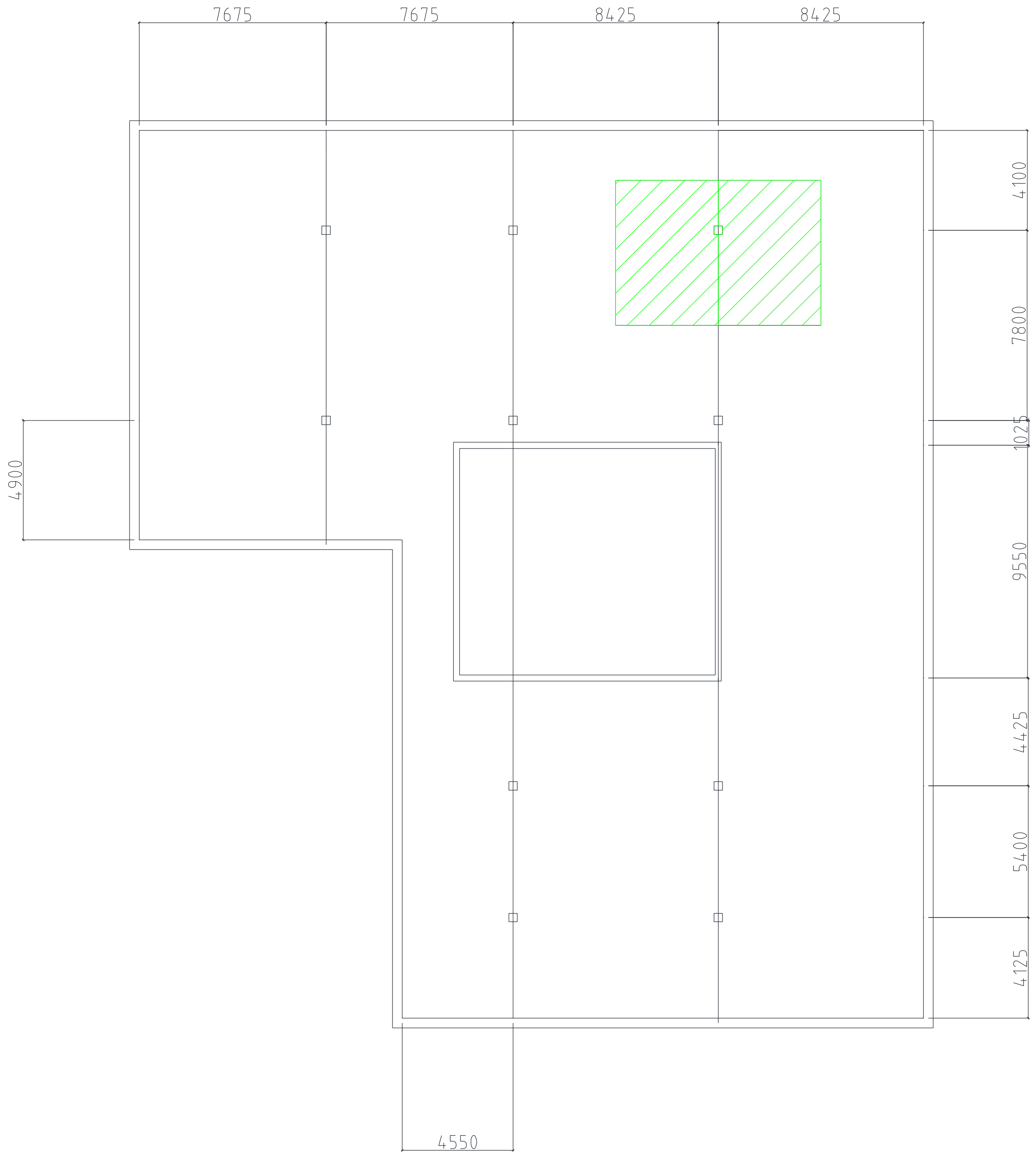
C3.2 Concept 1 and 2



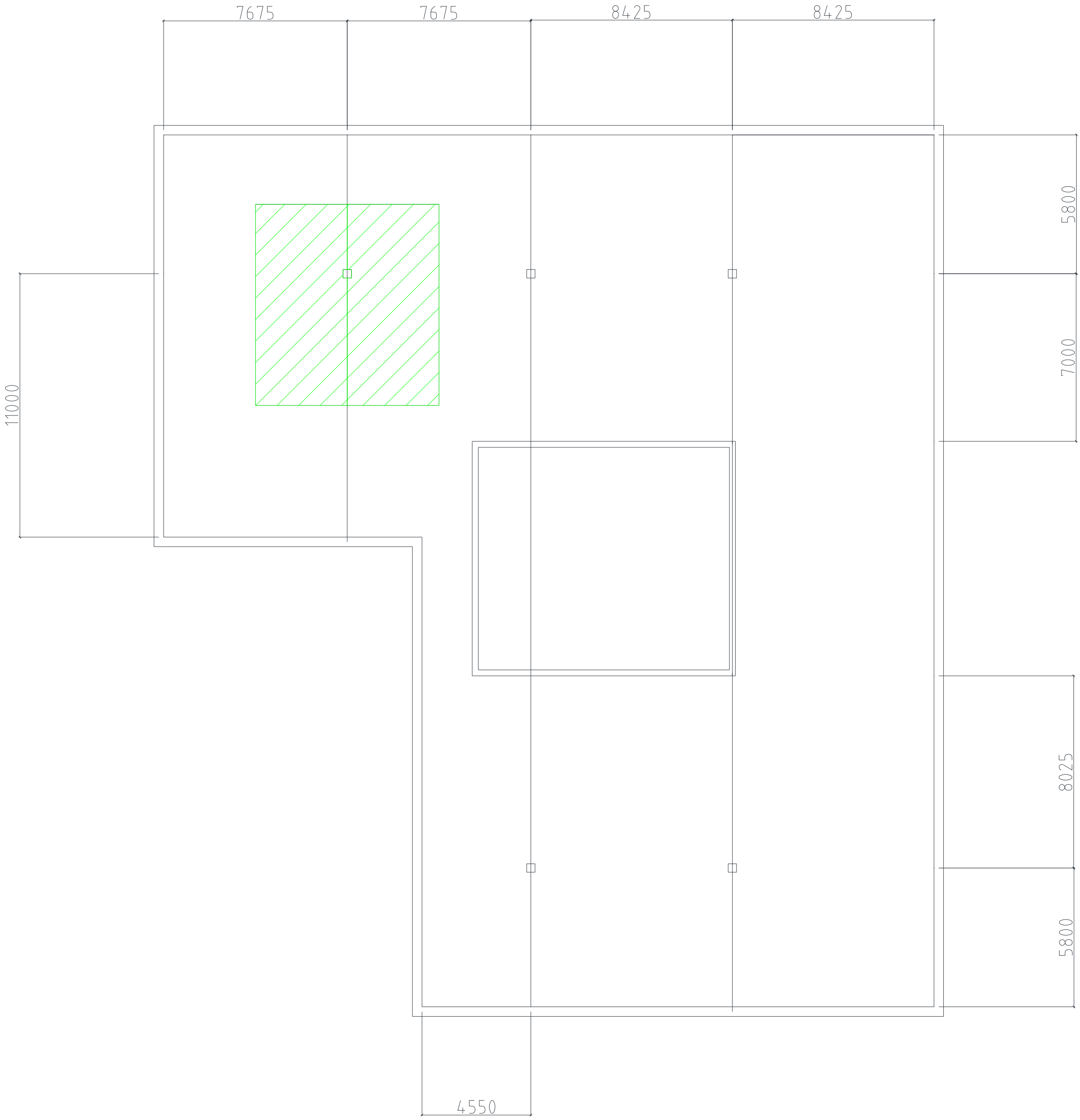
C3.3 Concept 3



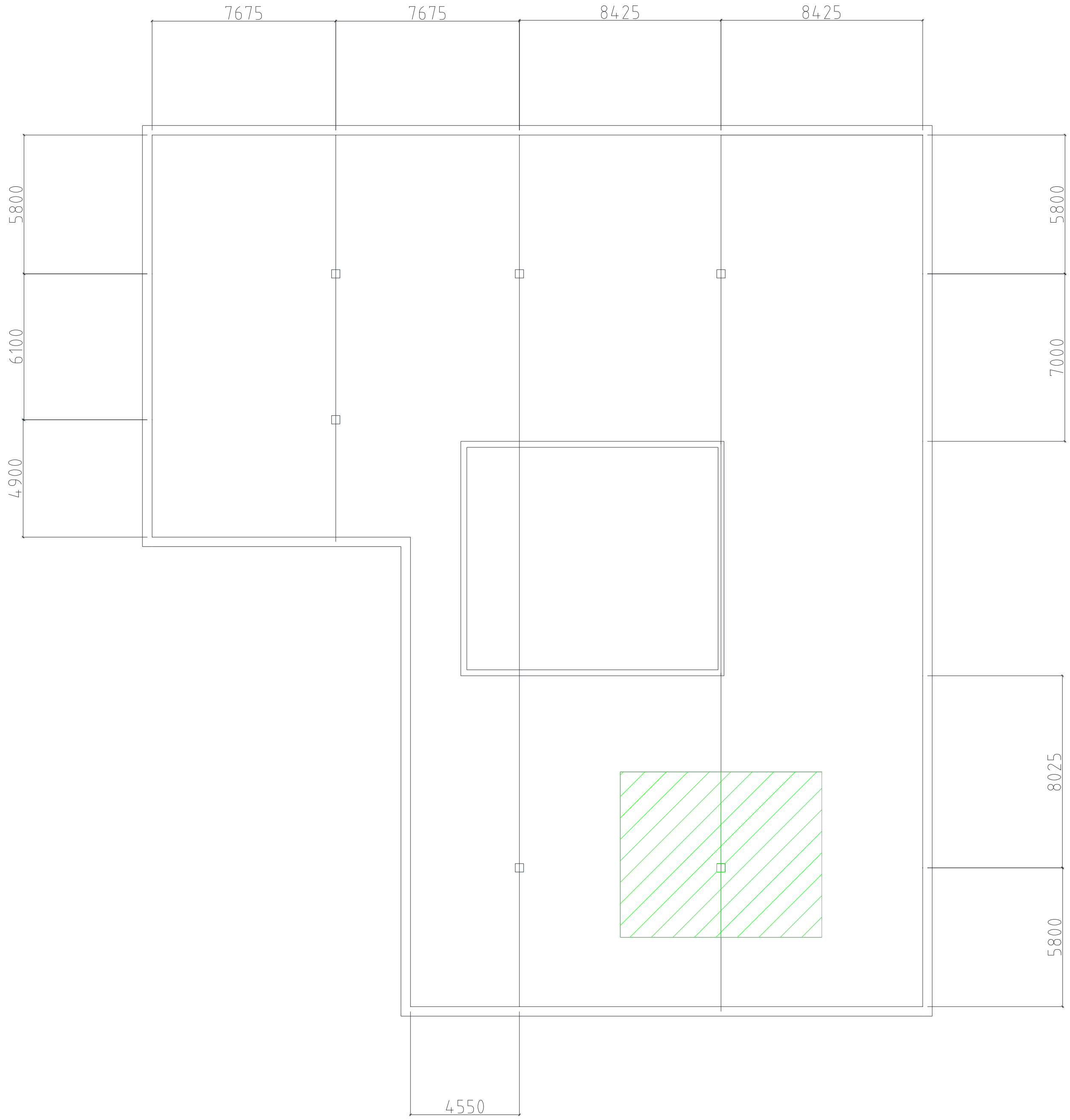
C3.4 Concept 4 and 5



C3.5 Concept 6, the first iteration



C3.6 Concept 6, the second interation



Appendix D1: Vertical deformation of the concrete core

This appendix contains the calculations of the vertical deformation of the concrete core. The result is presented in Section 7.3.

D1.1 Deformations due to creep

Mean value of elastic modulus (C45/55) $E_{cm} := 36GPa$

Characteristic value for the strenght $f_{ck} := 45MPa$

Mean value for the strength $f_{cm} := 53MPa$

Relative humidity (indoor climate) $RH := 50\%$

D1.1.1 Final creep coefficient for concrete

Final creep coefficent $\varphi(t, t_0) := \varphi_{RH} \cdot \beta(f_{cm}) \cdot \beta(t_0)$

Cross-sectional area of the concrete core $A_c := 0.25m \cdot (9 \cdot 3m + 3 \cdot 2.8m + 4m + 6m + 4.4m) = 12.45\,m^2$

Circumference of the concrete core

$$u := 18 \cdot 3m + 4m + 3.25m + 4 \cdot 3m + 2 \cdot 2.8m \ldots + 2 \cdot 3m + 2 \cdot 2.2m + 9 \cdot 0.25m = 91.5\,m$$

Age of concrete when load is applied $t_0 := 28$

Notional size

$$h_0 := \frac{2 \cdot A_c}{u} = 0.272\,m$$

Factor that considers the relative humidity (fcm > 35 MPa)

$$\varphi_{RH} := \left[1 + \frac{1 - RH}{0.1 \cdot \sqrt[3]{\dfrac{h_0}{mm}}} \cdot \left(\frac{35}{\dfrac{f_{cm}}{MPa}}\right)^{0.7}\right] \cdot \left(\frac{35}{\dfrac{f_{cm}}{MPa}}\right)^{0.2} = 1.451$$

Factor that considers the concrete strength class

$$\beta_{fcm} := \frac{16.8}{\sqrt{\dfrac{f_{ck}}{MPa} + 8}} = 2.308$$

Factor that considers the concrete age when loaded $\beta_{t0} := \dfrac{1}{0.1 + t_0^{\,0.2}} = 0.488$

Final creep coefficent $\varphi := \varphi_{RH} \cdot \beta_{fcm} \cdot \beta_{t0} = 1.636$

D1.1.2 Effective modulus of elasticity

Effective modulus of elasticity based on the first creep function $E_{c.ef} := \dfrac{E_{cm}}{1 + \varphi} = 13.657 \cdot GPa$

D1.1.3 Load acting on the concrete core

Density of concrete $\rho_c := 24 \dfrac{kN}{m^3}$

Height of storeys from the 2nd to the 14th storey

$$h_{floor} := \begin{pmatrix} 4.75 \\ 3.6 \\ 4 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \end{pmatrix} m \qquad i := 0..12$$

Self-weight of the concrete core $G_{c_i} := \rho_c \cdot A_c \cdot h_{floor_i}$

Area of floor inside of the core $A_{floor} := 11m \cdot 2.4m = 26.4\,m^2$

Thickness of floor inside the core $t_{floor} := 300mm$

Self -weight of floor $G_{floor} := A_{floor} \cdot t_{floor} \cdot \rho_c = 190.08 \cdot kN$

Total self-weight $G_{tot_i} := G_{c_i} + G_{floor}$

Loads from self-weight on each storey

$$P_{core_i} := \left| \begin{array}{l} P_{core_0} \leftarrow G_{tot_0} \\ \text{for } i \in 1..12 \\ \quad P_{core_i} \leftarrow P_{core_{i-1}} + G_{tot_i} \end{array} \right.$$

D1.1.4 Creep deformations

The creep deformations are calculated according to Hooke's law.

Applied compression stress $\sigma_{c_i} := \dfrac{P_{core_i}}{A_c}$

Creep strain $\varepsilon_{c_i} := \dfrac{\sigma_{c_i}}{E_{c.ef}}$

Creep deformations for each storey $\delta_i := \varepsilon_{c_i} \cdot h_{floor_i}$

Total creep deformation $\delta_{c.tot} := \sum_i \delta_i$ $\delta_{c.tot} = 2.582 \cdot mm$

Total creep deformations on each storey

$k := 12..0$

$$\delta_{c.t_k} := \left| \begin{array}{l} \delta_{c.t_{12}} \leftarrow \delta_{12} \\ \text{for } k \in 11..0 \\ \quad \delta_{c.t_k} \leftarrow \delta_{c.t_{k+1}} + \delta_k \end{array} \right.$$

D1.2 Deformation due to shrinkage

Equation for the final shrinkage strain $\varepsilon_{cs} := \varepsilon_{cd} + \varepsilon_{ca}$

Starting value $\varepsilon_{cdi} := 0.297 \cdot 10^{-3}$

Factor that considers the ambient relative humidity $\beta_{RH} := 1.36$

Notional size $h_0 = 0.272\,m$

Coefficient that depends on the size of the section $k_h := 0.85 + (0.75 - 0.85)\dfrac{(h_0 - 200mm)}{300mm - 200mm} = 0.778$

Drying shrinkage strain $\varepsilon_{cd} := k_h \cdot \beta_{RH} \cdot \varepsilon_{cdi} = 3.142 \times 10^{-4}$

Autogenous shrinkage $\varepsilon_{ca} := 0.0875 \cdot 10^{-3}$

Final shrinkage strain $\varepsilon_{cs} := \varepsilon_{cd} + \varepsilon_{ca} = 4.017 \times 10^{-4}$

D1.2.1 Shrinkage deformations

Shrinkage deformation on each storey $\Delta L_i := \varepsilon_{cs} \cdot h_{floor_i}$

Total shrinkage deformations $\delta_{s.tot} := \sum_i \Delta L_i$ $\delta_{s.tot} = 19.422 \cdot mm$

Total shrinkage deformations on each storey

$$\delta_{s.t_k} := \left| \begin{array}{l} \delta_{s.t_{12}} \leftarrow \Delta L_{12} \\ \text{for } k \in 11..0 \\ \quad \delta_{s.t_k} \leftarrow \delta_{s.t_{k+1}} + \Delta L_k \end{array} \right.$$

D1.3 Total vertical deformation

Total vertical deformation $\delta_{tot} := \delta_{c.tot} + \delta_{s.tot} = 22.004 \cdot mm$

The total vertical deformation per storey

$\delta_{t.tot} := \delta_{s.t} + \delta_{c.t} =$ (table below) $\cdot mm$

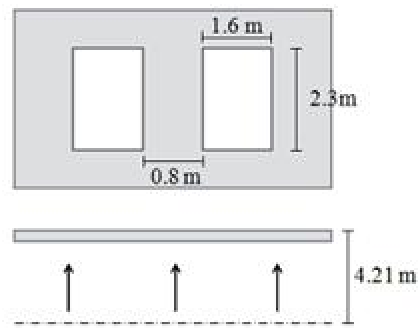
	0
0	22.004
1	20.051
2	18.544
3	16.837
4	15.274
5	13.684
6	12.067
7	10.424
8	8.754
9	7.056
10	5.333
11	3.582
12	1.804

(The value in the first row corresponds to the deformations on the 14th floor and the value on the last row corresponds to the deformations on the 2nd floor.)

Appendix D2: Vertical deformations of walls

This appendix contains the calculations of the vertical deformation of the timber walls used for both Concept 3 and Concept 4. The results are presented in Section 7.3.

D2.1 Geometric data



Width of window $w_{window} := 1.6m$

Height of window $h_{window} := 2.3m$

Height of walls from the 2nd to the 14th storey

$i := 0..12$

$$h_{wall} := \begin{pmatrix} 4.75 \\ 3.6 \\ 4 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \end{pmatrix} m$$

Height of beam part of the wall $$h_{beam_i} := \frac{h_{wall_i} - h_{window}}{2}$$

Width of column part of the wall $w_{col} := 0.8m$

Influence lenght $l_{infl} := 4.21m$

Influence width $w_{infl} := w_{col} + w_{window} = 2.4\,m$

Thickness of the walls

11th to 14th storey $t_{11} := 158mm$

6th to 10th storey $t_6 := 221mm$

2nd to 5th storey $t_2 := 259mm$

$$t := \begin{pmatrix} t_{11} \\ t_{11} \\ t_{11} \\ t_{11} \\ t_6 \\ t_6 \\ t_6 \\ t_6 \\ t_6 \\ t_2 \\ t_2 \\ t_2 \\ t_2 \end{pmatrix}$$

Thickness of material parallell to the grain (CLT walls has panels in both direction but only the ones that has the fibres parallell to the grains contributes to the load bearing capacity)

$$t_{parallell} := \begin{pmatrix} 95 \\ 95 \\ 95 \\ 95 \\ 95 \\ 95 \\ 95 \\ 95 \\ 95 \\ 133 \\ 133 \\ 133 \\ 133 \end{pmatrix} mm$$

Area of material loaded parallell to the grain(column part of the wall) $A_{col_i} := t_{parallell_i} \cdot w_{col}$

Thickness of insulation $t_{ins} := 200mm$

Thickness of non load bearing CLT part $t_{CLT} := 50mm$

D2.2 Deformations due to creep

D2.2.1 Loads acting on the walls

Self-weight of roof $g_{roof} := 4.56\,\frac{kN}{m^2}$ $G_{roof} := g_{roof} \cdot l_{infl} \cdot w_{infl} = 46.074 \cdot kN$

Self-weight of floor $g_{floor} := 1\,\frac{kN}{m^2}$ $G_{floor} := g_{floor} \cdot l_{infl} \cdot w_{infl} = 10.104 \cdot kN$

Self-weight of installations $g_{inst} := 0.3\,\frac{kN}{m^2}$ $G_{inst} := g_{inst} \cdot l_{infl} \cdot w_{infl} = 3.031 \cdot kN$

Density of CLT $\rho_{CLT} := 4\,\frac{kN}{m^3}$

Density of insulation $\rho_{ins} := 1.5\,\frac{kN}{m^3}$

Self-weight of column part of the wall $G_{col_i} := \rho_{CLT} \cdot w_{col} \cdot h_{wall_i} \cdot \left(t_i + t_{CLT}\right) + \rho_{ins} \cdot w_{col} \cdot h_{wall_i} \cdot t_{ins}$

Self-weight of beam part of the wall

$$G_{beam_i} := \begin{bmatrix} \rho_{CLT} \cdot \left(h_{wall_i} - h_{window}\right) \cdot \left(t_i + t_{CLT}\right) \cdots \\ + \rho_{ins} \cdot \left(h_{wall_i} - h_{window}\right) \cdot t_{ins} \end{bmatrix} \cdot w_{window}$$

Office load $q_{office} := 2.5\,\frac{kN}{m^2}$ $Q_{office} := q_{office} \cdot l_{infl} \cdot w_{infl} = 25.26 \cdot kN$

Load from partition walls $q_{part} := 0.5\,\frac{kN}{m^2}$ $Q_{part} := q_{part} \cdot l_{infl} \cdot w_{infl} = 5.052 \cdot kN$

Snow load $q_{snow} := 1.2\,\frac{kN}{m^2}$ $Q_{snow} := q_{snow} \cdot l_{infl} \cdot w_{infl} = 12.125 \cdot kN$

Coefficients for variable loads $\psi_{0.snow} := 0.7$

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

D2.2.1.1 Loads from self-weight of walls

$$P_{col_i} := \begin{array}{|l} P_{col_0} \leftarrow G_{col_0} \\ \text{for } i \in 1..12 \\ \quad P_{col_i} \leftarrow P_{col_{i-1}} + G_{col_i} \end{array}$$

D2.2.1.2 Imposed loads and permanent loads

Imposed load on each storey

$$Q_{imp} := \begin{pmatrix} \psi_{0.snow} \cdot Q_{snow} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \end{pmatrix}$$

Permanent load on each storey

$$G_{perm} := \begin{pmatrix} G_{roof} + G_{beam_0} \\ G_{floor} + G_{inst} + G_{beam_1} \\ G_{floor} + G_{inst} + G_{beam_2} \\ G_{floor} + G_{inst} + G_{beam_3} \\ G_{floor} + G_{inst} + G_{beam_4} \\ G_{floor} + G_{inst} + G_{beam_5} \\ G_{floor} + G_{inst} + G_{beam_6} \\ G_{floor} + G_{inst} + G_{beam_7} \\ G_{floor} + G_{inst} + G_{beam_8} \\ G_{floor} + G_{inst} + G_{beam_9} \\ G_{floor} + G_{inst} + G_{beam_{10}} \\ G_{floor} + G_{inst} + G_{beam_{11}} \\ G_{floor} + G_{inst} + G_{beam_{12}} \end{pmatrix}$$

Accumulated imposed load on each storey

$$P_{imp_i} := \begin{array}{|l} P_{imp_0} \leftarrow Q_{imp_0} \\ \text{for } i \in 1..12 \\ \quad P_{imp_i} \leftarrow P_{imp_{i-1}} + Q_{imp_i} \end{array}$$

Accumulated permanent load on each storey

$$P_{perm_i} := \begin{array}{|l} P_{perm_0} \leftarrow G_{perm_0} \\ \text{for } i \in 1..12 \\ \quad P_{perm_i} \leftarrow P_{perm_{i-1}} + G_{perm_i} \end{array}$$

D2.2.1.3 Load combination

$Q_i := P_{col_i} + P_{imp_i} + P_{perm_i}$

D2.2.2 Material data

Modulus of elasticity, parallel to grains $E_{0.mean} := 11000MPa$

Deformation modification factor
-Service class 1 (indoor environment) $k_{def} := 0.3$

Final mean value modulus for elasticity $E_{mean.fin} := \frac{E_{0.mean}}{1 + k_{def}} = 8.462 \times 10^3 \cdot MPa$

D2.2.3 Calculation of vertical deformation due to creep

The creep deformations are calculated according to Hooke's law.

Applied compression stress $\sigma_i := \frac{Q_i}{A_{col_i}}$

Creep strain $\varepsilon_i := \frac{\sigma_i}{E_{mean.fin}}$

Creep deformations for each storey $\delta_i := \varepsilon_i \cdot h_{wall_i}$

Total creep deformation $\delta_{c.tot} := \sum_i \delta_i$ $\delta_{c.tot} = 22.326 \cdot mm$

Total creep deformations on each storey

$k := 12..0$

$$\delta_{c.t_k} := \left| \begin{array}{l} \delta_{c.t_{12}} \leftarrow \delta_{12} \\ \text{for } k \in 11..0 \\ \quad \delta_{c.t_k} \leftarrow \delta_{c.t_{k+1}} + \delta_k \end{array} \right.$$

D2.3 Deformation due to shrinkage

Maximum shrinkage parallell to grain $\alpha_{f.par} := 0.3\%$

Annual change in RH $\Delta u := 2\%$

Fiber saturation point $u_f := 30\%$

Drying shrinkage strain $\Delta\alpha_{par} := \frac{\Delta u}{u_f} \cdot \alpha_{f.par} = 2 \times 10^{-4}$

Shrinkage deformation on each storey $\Delta L_{par_i} := \Delta\alpha_{par} \cdot h_{wall_i}$

Shrinkage deformation $\delta_{s.tot} := \sum_i \left(\Delta L_{par_i}\right)$ $\delta_{s.tot} = 9.67 \cdot mm$

Total shrinkage deformations on each storey

$$\delta_{s.t_k} := \left| \begin{array}{l} \delta_{s.t_{12}} \leftarrow \Delta L_{par_{12}} \\ \text{for } k \in 11..0 \\ \quad \delta_{s.t_k} \leftarrow \delta_{s.t_{k+1}} + \Delta L_{par_k} \end{array} \right.$$

D2.4 Total vertical deformation

Total vertical deformation $\delta_{tot} := \delta_{c.tot} + \delta_{s.tot} = 31.996 \cdot mm$

The total vertical deformation per storey

(The value in the first row corresponds tc the deformations on the 14th floor and the value on the last row correspond to the deformations on the 2nd floor.)

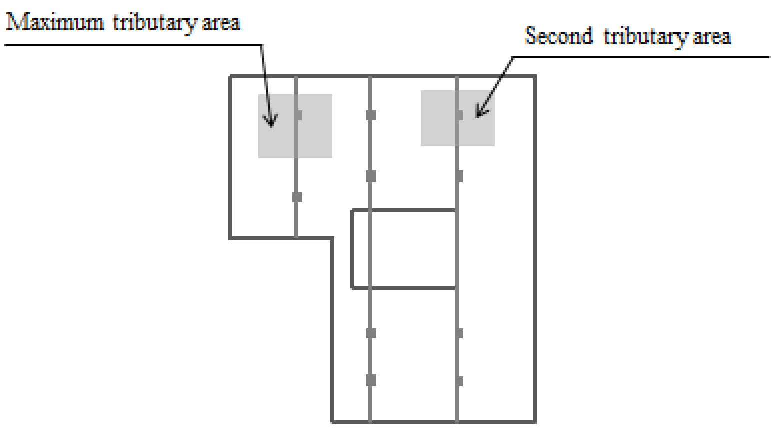
$\delta_{t.tot} := \delta_{s.t} + \delta_{c.t} =$ (table below) $\cdot mm$

	0
0	31.996
1	30.579
2	29.238
3	27.446
4	25.567
5	23.415
6	20.99
7	18.291
8	15.32
9	12.075
10	9.353
11	6.434
12	3.316

Appendix D3: Vertical deformations in the columns for Concept 3

This Appendix presnts the calculations performed on vertical deformations for the columns in Concept 3. The results from these calculations are presented in Section 7.3.

D3.1 Geometries data



Tributary area, maximum (reaches the 12th storey, up to balcony)

$A_{trib.max} := 48.7m^2$

Tributary area, for second column (all the way up to 14th storey)

$A_{trib} := 41.4m^2$

D3.2 Loads

D3.2.1 Snow load

Snow load on the roof

$q_{snow} := 1.2\frac{kN}{m^2}$

$Q_{snow} := q_{snow} \cdot A_{trib} = 49.68 \cdot kN$

Snow load on the balcony

$q_{snow.bal} := 1.5\frac{kN}{m^2}$

$Q_{snow.max} := q_{snow.bal} \cdot A_{trib.max} = 73.05 \cdot kN$

Self-weight of roof

$g_{roof} := 4.56\frac{kN}{m^2}$

$G_{roof} := g_{roof} \cdot A_{trib} = 188.784 \cdot kN$

D3.2.2 Self-weights

Self-weight of roof for balcony

$g_{b.roof} := 4.06 \frac{kN}{m^2}$

$G_{b.roof} := g_{b.roof} \cdot A_{trib.max} = 197.722 \cdot kN$

Influence length of beams Concept 3

$l_{inf.max} := 6.35m$ $\quad l_{inf} := 4.9125m$

Density for LVL-beams

$\rho_{lvl} := 510 \frac{kg}{m^3}$

Weight of roof beam, Kerto-S 800x225, Concept 3

$g_{r.b} := \rho_{lvl} \cdot 0.8m \cdot 0.22m$ $\quad G_{r.b} := l_{inf} \cdot g_{r.b} \cdot g = 4.324 \cdot kN$

Weight of balcony beam, HEA650 Concept 3

$g_{b.b} := 190 \frac{kg}{m}$ $\quad G_{b.b} := l_{inf.max} \cdot g_{b.b} \cdot g = 11.832 \cdot kN$

Weight of office beam, Kerto-S 650x225, Concept 3

$g_{o.b} := \rho_{lvl} \cdot 0.65m \cdot 0.225m$

$G_{o.b} := l_{inf} \cdot g_{o.b} \cdot g = 3.593 \cdot kN$

Weight of office beam 2, HEA500 Concept 3

$g_{o.b.max} := 155 \frac{kg}{m}$

$G_{o.b.max} := l_{inf.max} \cdot g_{o.b.max} \cdot g = 9.652 \cdot kN$

Self-weight of the timber slab

$g_{slab} := 1 \frac{kN}{m^2}$

$G_{slab} := g_{slab} \cdot A_{trib} = 41.4 \cdot kN$

$G_{slab.max} := g_{slab} \cdot A_{trib.max} = 48.7 \cdot kN$

Self-weight of installations

$g_{ins} := 0.3 \frac{kN}{m^2}$

$G_{ins} := g_{ins} \cdot A_{trib} = 12.42 \cdot kN$

$G_{ins.max} := g_{ins} \cdot A_{trib.max} = 14.61 \cdot kN$

D3.2.3 Imposed loads

Office load

$q_{office} := 2.5 \frac{kN}{m^2}$

$Q_{office} := q_{office} \cdot A_{trib} = 103.5 \cdot kN$

$Q_{office.max} := q_{office} \cdot A_{trib.max} = 121.75 \cdot kN$

Balcony load

$q_{bal} := 5 \frac{kN}{m^2}$

$Q_{bal} := q_{bal} \cdot A_{trib.max} = 243.5 \cdot kN$

Loads from partition walls

$q_{part} := 0.5 \frac{kN}{m^2}$

$Q_{part} := q_{part} \cdot A_{trib} = 20.7 \cdot kN$

$Q_{part.max} := q_{part} \cdot A_{trib.max} = 24.35 \cdot kN$

Coefficients for variable loads

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

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

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

Cross-sectional area of the columns

$A_{11} := 0.33m \cdot 0.36m = 0.119\, m^2$

$A_6 := 0.43m \cdot 0.405m = 0.174\, m^2$

$A_2 := 0.43m \cdot 0.54m = 0.232\, m^2$

$i := 0 .. 12$

$j := 2 .. 12$

Area and height of the columns

$$A_{col} := \begin{pmatrix} A_{11} \\ A_{11} \\ A_{11} \\ A_{11} \\ A_6 \\ A_6 \\ A_6 \\ A_6 \\ A_6 \\ A_2 \\ A_2 \\ A_2 \\ A_2 \end{pmatrix} \qquad h_{col} := \begin{pmatrix} 4.75 \\ 3.6 \\ 4 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \end{pmatrix} m$$

Density of columns

$$\rho_{mean.gl} := 440 \frac{kg}{m^3}$$

Self-weight of columns

$$G_{col_i} := A_{col_i} \cdot \rho_{mean.gl} \cdot h_{col_i} \cdot g \qquad G_{col} = \cdot kN$$

	0
0	2.435
1	1.845
2	2.05
3	1.845
4	2.705
5	2.705
6	2.705
7	2.705
8	2.705
9	3.607
10	3.607
11	3.607
12	3.607

D3.2.4 Accumulating loads and load combination

Loads from self-weight on columns

The load accumumates

$$P_{col_i} := \left| \begin{array}{l} P_{col_0} \leftarrow G_{col_0} \\ \text{for } i \in 1..12 \\ \quad P_{col_i} \leftarrow P_{col_{i-1}} + G_{col_i} \end{array} \right. \qquad P_{col.max_j} := \left| \begin{array}{l} P_{col.max_2} \leftarrow G_{col_2} \\ \text{for } j \in 3..12 \\ \quad P_{col.max_j} \leftarrow P_{col.max_{j-1}} + G_{col_j} \end{array} \right.$$

Imposed loads, office load as main load, snow and partion walls is combinated with their respectively coefficient

Load combination

$$Q_{imp} := \begin{pmatrix} \psi_{0.snow} \cdot Q_{snow} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \end{pmatrix} \qquad Q_{imp.max} := \begin{pmatrix} 0kN \\ 0kN \\ \psi_{0.snow} \cdot Q_{snow.max} + \psi_{0.bal} \cdot Q_{bal} \\ Q_{office.max} + \psi_{0.part} \cdot Q_{part.max} \\ Q_{office.max} + \psi_{0.part} \cdot Q_{part.max} \\ Q_{office.max} + \psi_{0.part} \cdot Q_{part.max} \\ Q_{office.max} + \psi_{0.part} \cdot Q_{part.max} \\ Q_{office.max} + \psi_{0.part} \cdot Q_{part.max} \\ Q_{office.max} + \psi_{0.part} \cdot Q_{part.max} \\ Q_{office.max} + \psi_{0.part} \cdot Q_{part.max} \\ Q_{office.max} + \psi_{0.part} \cdot Q_{part.max} \\ Q_{office.max} + \psi_{0.part} \cdot Q_{part.max} \\ Q_{office.max} + \psi_{0.part} \cdot Q_{part.max} \end{pmatrix}$$

$$P_{imp_i} := \left| \begin{array}{l} P_{imp_0} \leftarrow Q_{imp_0} \\ \text{for } i \in 1 .. 12 \\ \quad P_{imp_i} \leftarrow P_{imp_{i-1}} + Q_{imp_i} \end{array} \right. \qquad P_{imp.max_j} := \left| \begin{array}{l} P_{imp.max_2} \leftarrow Q_{imp.max_2} \\ \text{for } j \in 3 .. 12 \\ \quad P_{imp.max_j} \leftarrow P_{imp.max_{j-1}} + Q_{imp.max_j} \end{array} \right.$$

Loads from roof, floor and beam

$$G_{perm} := \begin{pmatrix} G_{roof} + G_{r.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \end{pmatrix} \qquad G_{perm.max} := \begin{pmatrix} 0kN \\ 0kN \\ G_{b.roof} + G_{b.b} \\ G_{slab.max} + G_{ins.max} + G_{o.b.max} \\ G_{slab.max} + G_{ins.max} + G_{o.b.max} \\ G_{slab.max} + G_{ins.max} + G_{o.b.max} \\ G_{slab.max} + G_{ins.max} + G_{o.b.max} \\ G_{slab.max} + G_{ins.max} + G_{o.b.max} \\ G_{slab.max} + G_{ins.max} + G_{o.b.max} \\ G_{slab.max} + G_{ins.max} + G_{o.b.max} \\ G_{slab.max} + G_{ins.max} + G_{o.b.max} \\ G_{slab.max} + G_{ins.max} + G_{o.b.max} \\ G_{slab.max} + G_{ins.max} + G_{o.b.max} \end{pmatrix}$$

$$P_{perm_i} := \left| \begin{array}{l} P_{perm_0} \leftarrow G_{perm_0} \\ \text{for } i \in 1..12 \\ \quad P_{perm_i} \leftarrow P_{perm_{i-1}} + G_{perm} \end{array} \right. \qquad P_{perm.max_j} := \left| \begin{array}{l} P_{perm.max_2} \leftarrow G_{perm.max_2} \\ \text{for } j \in 3..12 \\ \quad P_{perm.max_j} \leftarrow P_{perm.max_{j-1}} + G_{perm.ma} \end{array} \right.$$

D3.2.5 Summation of combinated loads

Since the deformations are irreversible the characteristic load combinations is chosen.

Snow load or partition walls as main load will never be the worst case, hence office load is main load.

$$Q_i := P_{col_i} + P_{imp_i} + P_{perm_i} \qquad Q_{max_i} := P_{col.max_i} + P_{imp.max_i} + P_{perm.max_i}$$

$$Q = \begin{array}{|c|c|}\hline & 0 \\ \hline 0 & 230.319 \\ 1 & 407.568 \\ 2 & 585.022 \\ 3 & 762.27 \\ 4 & 940.379 \\ 5 & 1.118\cdot10^3 \\ 6 & 1.297\cdot10^3 \\ 7 & 1.475\cdot10^3 \\ 8 & 1.653\cdot10^3 \\ 9 & 1.832\cdot10^3 \\ 10 & 2.011\cdot10^3 \\ 11 & 2.19\cdot10^3 \\ 12 & 2.369\cdot10^3 \\ \hline\end{array} \cdot kN \qquad Q_{max} = \begin{array}{|c|c|}\hline & 0 \\ \hline 0 & 0 \\ 1 & 0 \\ 2 & 433.189 \\ 3 & 646.792 \\ 4 & 861.254 \\ 5 & 1.076\cdot10^3 \\ 6 & 1.29\cdot10^3 \\ 7 & 1.505\cdot10^3 \\ 8 & 1.719\cdot10^3 \\ 9 & 1.934\cdot10^3 \\ 10 & 2.15\cdot10^3 \\ 11 & 2.365\cdot10^3 \\ 12 & 2.581\cdot10^3 \\ \hline\end{array} \cdot kN$$

D3.3 Material data

Timber columns of Lc40

Modulus of elasticity, parallel to grains $E_{0.mean.gl} := 13000MPa$

Deformation modification factor
-Service class 1 (indoor environment) $k_{def.gl} := 0.6$

Final mean value modulus for elasticity $E_{mean.fin.gl} := \dfrac{E_{0.mean.gl}}{1 + k_{def.gl}} = 8.125 \times 10^3 \cdot MPa$

D3.4 Calculation of vertical deformation due to creep

Applied compression stress on each column

$$\sigma_i := \frac{Q_i}{A_{col_i}} \qquad \sigma_{max_i} := \frac{Q_{max_i}}{A_{col_i}}$$

$\sigma =$

	0
0	1.939
1	3.431
2	4.924
3	6.416
4	5.4
5	6.423
6	7.445
7	8.468
8	9.491
9	7.889
10	8.66
11	9.431
12	10.202

$\cdot MPa$

$\sigma_{max} =$

	0
0	0
1	0
2	3.646
3	5.444
4	4.945
5	6.177
6	7.408
7	8.64
8	9.871
9	8.331
10	9.259
11	10.186
12	11.114

$\cdot MPa$

Strain on each storey $\varepsilon_i := \frac{\sigma_i}{E_{mean.fin.gl}}$ $\varepsilon_{max_i} := \frac{\sigma_{max_i}}{E_{mean.fin.gl}}$

Deformations on each storey $\delta_i := \varepsilon_i \cdot h_{col_i}$ $\delta_{max_i} := \varepsilon_{max_i} \cdot h_{col_i}$

$\delta =$

	0
0	1.133
1	1.52
2	2.424
3	2.843
4	2.393
5	2.846
6	3.299
7	3.752
8	4.205
9	3.495
10	3.837
11	4.179
12	4.52

$\cdot mm$

$\delta_{max} =$

	0
0	0
1	0
2	1.795
3	2.412
4	2.191
5	2.737
6	3.283
7	3.828
8	4.374
9	3.691
10	4.102
11	4.513
12	4.924

$\cdot mm$

D3.4.1 Creep deformations

$k := 12..0$

Total vertical deformations due to creep

$$\delta_{c.tot} := \sum_i \delta_i \qquad \delta_{c.tot.max} := \sum_i \delta_{max_i}$$

$$\delta_{c.tot} = 40.446 \cdot mm \qquad \delta_{c.tot.max} = 37.851 \cdot mm$$

Creep deformations for each storey

$$\delta_{c.t_k} := \left| \begin{array}{l} \delta_{c.t_{12}} \leftarrow \delta_{12} \\ \text{for } k \in 11..0 \\ \quad \delta_{c.t_k} \leftarrow \delta_{c.t_{k+1}} + \delta_k \end{array} \right. \qquad \delta_{c.t.max_k} := \left| \begin{array}{l} \delta_{c.t.max_{12}} \leftarrow \delta_{max_{12}} \\ \text{for } k \in 11..2 \\ \quad \delta_{c.t.max_k} \leftarrow \delta_{c.t.max_{k+1}} + \delta_{max_k} \end{array} \right.$$

D3.5 Deformations of beams due to compression perpendicular to the grains

D3.5.1 Check of compression perpendicular to the grain

The following expression shall be satisfied

$$\sigma_{c.90.d} \le k_{c.90} \cdot f_{c.90.d} \qquad \text{where} \qquad \sigma_{c.90.d} := \frac{F_{c.90.d}}{A_{ef}}$$

Compression capacity perpendicular to the grain $f_{c.90.d} := 6MPa$

Modification factor $k_{c.90} := 1.5$

Width of columns $w_{11} := 0.360m$

$w_6 := 0.430m$

$w_2 := 0.540m$

Thickness of beams $t_{beam} := 0.225m$

Area of compressive zone

$$A_{ef} := \begin{pmatrix} w_{11} + 2\cdot 30mm \\ w_{11} + 2\cdot 30mm \\ w_{11} + 2\cdot 30mm \\ w_{11} + 2\cdot 30mm \\ w_6 + 2\cdot 30mm \\ w_6 + 2\cdot 30mm \\ w_6 + 2\cdot 30mm \\ w_6 + 2\cdot 30mm \\ w_6 + 2\cdot 30mm \\ w_2 + 2\cdot 30mm \\ w_2 + 2\cdot 30mm \\ w_2 + 2\cdot 30mm \\ w_2 + 2\cdot 30mm \end{pmatrix} \cdot t_{beam}$$

Utilisation ratio

$$u_i := \frac{Q_i}{k_{c.90}\cdot f_{c.90.d}\cdot A_{ef_i}} \qquad u =$$

	0
0	0.271
1	0.479
2	0.688
3	0.896
4	0.948
5	1.127
6	1.307
7	1.486
8	1.666
9	1.508
10	1.655
11	1.802
12	1.95

The utilisation ratio is above 1 for all columns below the 10th storey. Hence compression strength of the beams perpendicular to the grains is insufficient and therefore not allowed!

D3.6 Deformation due to shrinkage

Maximum shrinkage parallell to grain $\alpha_f := 0.3\%$

Annual change in RH $\Delta u := 2\%$

Fiber saturation point $u_f := 30\%$

$$\Delta\alpha := \frac{\Delta u}{u_f}\cdot\alpha_f = 2\times 10^{-4}$$

$$\Delta L_i := \Delta\alpha\cdot h_{col_i}$$

Shrinkage deformations

$\delta_{s.tot} := \sum_i \Delta L_i$ $\delta_{s.tot} = 9.67\cdot mm$

$\delta_{s.tot.max} := \sum_j \Delta L_j$ $\delta_{s.tot.max} = 8\cdot mm$

$kj := 11..2$

Shrinkage deformations for each storey

$$\delta_{s.t_k} := \left| \begin{array}{l} \delta_{s.t_{12}} \leftarrow \Delta L_{12} \\ \text{for } k \in 11..0 \\ \quad \delta_{s.t_k} \leftarrow \delta_{s.t_{k+1}} + \Delta L_k \end{array} \right.$$

$$\delta_{s.t.max_k} := \left| \begin{array}{l} \delta_{s.t.max_{12}} \leftarrow \Delta L_{12} \\ \text{for } kj \in 11..2 \\ \quad \delta_{s.t.max_{kj}} \leftarrow \delta_{s.t.max_{kj+1}} + \Delta L_{kj} \end{array} \right.$$

D3.7 Total vertical deformation

Deformation at the 14th storey $\delta_{tot} := \delta_{c.tot} + \delta_{s.tot} = 50.116\cdot mm$

Deformation at the 12th storey $\delta_{tot.max} := \delta_{c.tot.max} + \delta_{s.tot.max} = 45.851\cdot mm$

$\delta_{t.tot} := \delta_{c.t} + \delta_{s.t} =$

	0
0	50.116
1	48.033
2	45.793
3	42.568
4	39.005
5	35.893
6	32.327
7	28.308
8	23.836
9	18.911
10	14.696
11	10.139
12	5.24

$\cdot mm$

$\delta_{t.tot.max} := \delta_{c.t.max} + \delta_{s.t.max} =$

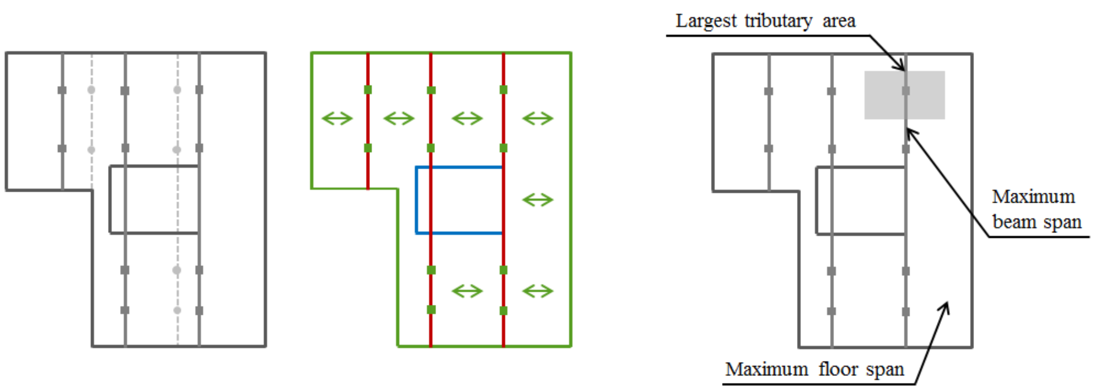
	0
0	45.851
1	45.851
2	45.851
3	43.256
4	40.123
5	37.212
6	33.755
7	29.753
8	25.205
9	20.111
10	15.7
11	10.877
12	5.644

$\cdot mm$

Appendix D4: Vertical deformations in the columns for Concept 4

This Appendix presnts the calculations performed on vertical deformations for the columns in Concept 4. The results from these calculations are presented in Section 7.3.

D4.1 Geometries data



(a) (b) (c)

Tributary area (for part that reaches the 12th storey, up to balcony) $A_{trib.min} := 45.7m^2$

Tributary area, (for columns all the way up to 14th storey) $A_{trib} := 50.1m^2$

D4.2 Loads

D4.2.1 Snow load

Snow load on the roof

$$q_{snow} := 1.2 \frac{kN}{m^2}$$

$$Q_{snow} := q_{snow} \cdot A_{trib} = 60.12 \cdot kN$$

$$q_{snow.bal} := 1.5 \frac{kN}{m^2}$$

$$Q_{snow.max} := q_{snow.bal} \cdot A_{trib.min} = 68.55 \cdot kN$$

D4.2.2 Self-weights

Self-weight of roof $g_{roof} := 4.56 \frac{kN}{m^2}$ $G_{roof} := g_{roof} \cdot A_{trib} = 228.456 \cdot kN$

Self-weight of roof for balcony $g_{b.roof} := 4.06 \frac{kN}{m^2}$

$G_{b.roof} := g_{b.roof} \cdot A_{trib.min} = 185.542 \cdot kN$

Influence length of beams
Concept 4 $l_{inf.4} := 5.95m$

Weight of roof beam, HEA500
Concept 4 $g_{r.b.4} := 155 \frac{kg}{m}$ $G_{r.b} := l_{inf.4} \cdot g_{r.b.4} \cdot g = 9.044 \cdot kN$

Weight of office beam, HEA450
Concept 4 $g_{o.b.4} := 140 \frac{kg}{m}$ $G_{o.b} := l_{inf.4} \cdot g_{o.b.4} \cdot g = 8.169 \cdot kN$

Weight of balcony beam, HEA650
Concept 4 $g_{b.b} := 190 \frac{kg}{m}$ $G_{b.b} := l_{inf.4} \cdot g_{b.b} \cdot g = 11.086 \cdot kN$

Self-weight of the timber slab $g_{slab} := 1 \frac{kN}{m^2}$

$G_{slab} := g_{slab} \cdot A_{trib} = 50.1 \cdot kN$

$G_{slab.min} := g_{slab} \cdot A_{trib.min} = 45.7 \cdot kN$

Self-weight of installations $g_{ins} := 0.3 \frac{kN}{m^2}$

$G_{ins} := g_{ins} \cdot A_{trib} = 15.03 \cdot kN$

$G_{ins.min} := g_{ins} \cdot A_{trib.min} = 13.71 \cdot kN$

D4.2.3 Imposed loads

Office load $q_{office} := 2.5 \frac{kN}{m^2}$

$Q_{office} := q_{office} \cdot A_{trib} = 125.25 \cdot kN$

$Q_{office.min} := q_{office} \cdot A_{trib.min} = 114.25 \cdot kN$

Balcony load

$q_{bal} := 5 \dfrac{kN}{m^2}$ $\qquad$ $Q_{bal} := q_{bal} \cdot A_{trib.min} = 228.5 \cdot kN$

Loads from partition walls

$q_{part} := 0.5 \dfrac{kN}{m^2}$

$Q_{part} := q_{part} \cdot A_{trib} = 25.05 \cdot kN$

$Q_{part.min} := q_{part} \cdot A_{trib.min} = 22.85 \cdot kN$

Coefficients for variable loads

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

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

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

Area of columns

$A_{11} := 0.330m \cdot 0.360m = 0.119\,m^2$

$A_6 := 0.430m \cdot 0.405m = 0.174\,m^2$

$A_2 := 0.430m \cdot 0.540m = 0.232\,m^2$

$$A_{col} := \begin{pmatrix} A_{11} \\ A_{11} \\ A_{11} \\ A_{11} \\ A_6 \\ A_6 \\ A_6 \\ A_6 \\ A_6 \\ A_2 \\ A_2 \\ A_2 \\ A_2 \end{pmatrix} \qquad h_{col} := \begin{pmatrix} 4.75 \\ 3.6 \\ 4 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \\ 3.6 \end{pmatrix} m$$

Density of columns

$$\rho_{mean.gl} := 440 \frac{kg}{m^3}$$

$i := 0..12 \qquad j := 2..12$

Self-weight of columns

$$G_{col_i} := A_{col_i} \cdot \rho_{mean.gl} \cdot h_{col_i} \cdot g \qquad G_{col} = \cdot kN$$

	0
0	2.435
1	1.845
2	2.05
3	1.845
4	2.705
5	2.705
6	2.705
7	2.705
8	2.705
9	3.607
10	3.607
11	3.607
12	3.607

D4.2.4 Accumulating loads and load combination

Loads from self-weight on columns

The load accumumates

$$P_{col_i} := \left| \begin{array}{l} P_{col_0} \leftarrow G_{col_0} \\ \text{for } i \in 1..12 \\ \quad P_{col_i} \leftarrow P_{col_{i-1}} + G_{col_i} \end{array} \right. \qquad P_{col.min_j} := \left| \begin{array}{l} P_{col.min_2} \leftarrow G_{col_2} \\ \text{for } j \in 3..12 \\ \quad P_{col.min_j} \leftarrow P_{col.min_{j-1}} + G_{col_j} \end{array} \right.$$

Imposed loads, office load as main load, snow and partion walls is combinated with their respectively coefficient

Imposed loads

Load combination

$$Q_{imp} := \begin{pmatrix} \psi_{0.snow} \cdot Q_{snow} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \\ Q_{office} + \psi_{0.part} \cdot Q_{part} \end{pmatrix} \qquad Q_{imp.min} := \begin{pmatrix} 0kN \\ 0kN \\ \psi_{0.snow} \cdot Q_{snow.max} + \psi_{0.bal} \cdot Q_{bal} \\ Q_{office.min} + \psi_{0.part} \cdot Q_{part.min} \\ Q_{office.min} + \psi_{0.part} \cdot Q_{part.min} \\ Q_{office.min} + \psi_{0.part} \cdot Q_{part.min} \\ Q_{office.min} + \psi_{0.part} \cdot Q_{part.min} \\ Q_{office.min} + \psi_{0.part} \cdot Q_{part.min} \\ Q_{office.min} + \psi_{0.part} \cdot Q_{part.min} \\ Q_{office.min} + \psi_{0.part} \cdot Q_{part.min} \\ Q_{office.min} + \psi_{0.part} \cdot Q_{part.min} \\ Q_{office.min} + \psi_{0.part} \cdot Q_{part.min} \\ Q_{office.min} + \psi_{0.part} \cdot Q_{part.min} \end{pmatrix}$$

$$P_{imp_i} := \left| \begin{array}{l} P_{imp_0} \leftarrow Q_{imp_0} \\ \text{for } \; i \in 1..12 \\ \quad P_{imp_i} \leftarrow P_{imp_{i-1}} + Q_{imp_i} \end{array} \right. \qquad P_{imp.min_j} := \left| \begin{array}{l} P_{imp.min_2} \leftarrow Q_{imp.min_2} \\ \text{for } \; j \in 3..12 \\ \quad P_{imp.min_j} \leftarrow P_{imp.min_{j-1}} + Q_{imp.mi} \end{array} \right.$$

Loads from roof, floor and beam

$$G_{perm} := \begin{pmatrix} G_{roof} + G_{r.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \\ G_{slab} + G_{ins} + G_{o.b} \end{pmatrix} \qquad G_{perm.min} := \begin{pmatrix} 0kN \\ 0kN \\ G_{b.roof} + G_{b.b} \\ G_{slab.min} + G_{ins.min} + G_{o.b} \\ G_{slab.min} + G_{ins.min} + G_{o.b} \\ G_{slab.min} + G_{ins.min} + G_{o.b} \\ G_{slab.min} + G_{ins.min} + G_{o.b} \\ G_{slab.min} + G_{ins.min} + G_{o.b} \\ G_{slab.min} + G_{ins.min} + G_{o.b} \\ G_{slab.min} + G_{ins.min} + G_{o.b} \\ G_{slab.min} + G_{ins.min} + G_{o.b} \\ G_{slab.min} + G_{ins.min} + G_{o.b} \\ G_{slab.min} + G_{ins.min} + G_{o.b} \end{pmatrix}$$

$$P_{perm_i} := \left\| \begin{array}{l} P_{perm_0} \leftarrow G_{perm_0} \\ \text{for } i \in 1..12 \\ \quad P_{perm_i} \leftarrow P_{perm_{i-1}} + G_{perm} \end{array} \right. \qquad P_{perm.min_j} := \left\| \begin{array}{l} P_{perm.min_2} \leftarrow G_{perm.min_2} \\ \text{for } j \in 3..12 \\ \quad P_{perm.min_j} \leftarrow P_{perm.min_{j-1}} + G_{perm.min_j} \end{array} \right.$$

D4.2.5 Summation of combinated loads

Since the deformations are irreversible the characteristic load combinations is chosen.

Snow load or partition walls as main load will never be the worst case, hence office load is main load.

$$Q_i := P_{col_i} + P_{imp_i} + P_{perm_i} \qquad Q_{min_i} := P_{col.min_i} + P_{imp.min_i} + P_{perm.min_i}$$

$Q =$ (·kN)

	0
0	282.019
1	499.948
2	718.083
3	936.012
4	$1.155 \cdot 10^3$
5	$1.374 \cdot 10^3$
6	$1.592 \cdot 10^3$
7	$1.811 \cdot 10^3$
8	$2.03 \cdot 10^3$
9	$2.25 \cdot 10^3$
10	$2.469 \cdot 10^3$
11	$2.689 \cdot 10^3$
12	$2.909 \cdot 10^3$

$Q_{min} =$ (·kN)

	0
0	0
1	0
2	406.614
3	606.283
4	806.812
5	$1.007 \cdot 10^3$
6	$1.208 \cdot 10^3$
7	$1.408 \cdot 10^3$
8	$1.609 \cdot 10^3$
9	$1.81 \cdot 10^3$
10	$2.012 \cdot 10^3$
11	$2.213 \cdot 10^3$
12	$2.415 \cdot 10^3$

D4.3 Material data

Timber columns of Lc40

Modulus of elasticity, parallel to grains $E_{0.mean.gl} := 13000MPa$

Deformation modification factor
-Service class 1 (indoor environment) $k_{def.gl} := 0.2$

Final mean value modulus for elasticity $E_{mean.fin.gl} := \dfrac{E_{0.mean.gl}}{1 + k_{def.gl}} = 1.083 \times 10^4 \cdot MPa$

D4.4 Calculation of vertical deformation for concept 4

Applied compression stress on each column $\sigma_i := \dfrac{Q_i}{A_{col_i}}$ $\qquad \sigma_{min_i} := \dfrac{Q_{min_i}}{A_{col_i}}$

$\sigma =$ ·MPa

	0
0	2.374
1	4.208
2	6.044
3	7.879
4	6.631
5	7.887
6	9.144
7	10.4
8	11.656
9	9.688
10	10.635
11	11.581
12	12.527

$\sigma_{min} =$ ·MPa

	0
0	0
1	0
2	3.423
3	5.103
4	4.633
5	5.784
6	6.936
7	8.087
8	9.239
9	7.797
10	8.664
11	9.532
12	10.399

Strain on each storey

$$\varepsilon_i := \frac{\sigma_i}{E_{mean.fin.gl}} \qquad \varepsilon_{min_i} := \frac{\sigma_{min_i}}{E_{mean.fin.gl}}$$

Deformations on each storey

$$\delta_i := \varepsilon_i \cdot h_{col_i} \qquad \delta_{min_i} := \varepsilon_{min_i} \cdot h_{col_i}$$

$\delta =$ ·mm

	0
0	1.041
1	1.398
2	2.232
3	2.618
4	2.204
5	2.621
6	3.039
7	3.456
8	3.874
9	3.22
10	3.534
11	3.848
12	4.163

$\delta_{min} =$ ·mm

	0
0	0
1	0
2	1.264
3	1.696
4	1.54
5	1.922
6	2.305
7	2.687
8	3.07
9	2.591
10	2.879
11	3.167
12	3.456

D4.4.1 Creep deformations

$$\delta_{c.tot} := \sum_i \delta_i \qquad \delta_{c.tot.min} := \sum_i \delta_{min_i}$$

$$\delta_{c.tot} = 37.247 \cdot mm \qquad \delta_{c.tot.min} = 26.577 \cdot mm$$

Creep deformations for each storey

$k := 12..0$

$$\delta_{c.t_k} := \left| \begin{array}{l} \delta_{c.t_{12}} \leftarrow \delta_{12} \\ \text{for } k \in 11..0 \\ \quad \delta_{c.t_k} \leftarrow \delta_{c.t_{k+1}} + \delta_k \end{array} \right. \qquad \delta_{c.t.min_k} := \left| \begin{array}{l} \delta_{c.t.min_{12}} \leftarrow \delta_{min_{12}} \\ \text{for } k \in 11..2 \\ \quad \delta_{c.t.min_k} \leftarrow \delta_{c.t.min_{k+1}} + \delta_{min_k} \end{array} \right.$$

D4.5 Deformation due to shrinkage

Maximum shrinkage parallell to grain $\alpha_f := 0.3\%$

Annual change in RH $\Delta u := 2\%$

Fiber saturation point $u_f := 30\%$

$$\Delta\alpha := \frac{\Delta u}{u_f} \cdot \alpha_f = 2 \times 10^{-4}$$

$$\Delta L_i := \Delta\alpha \cdot h_{col_i}$$

Shrinkage deformations

$$\delta_{s.tot} := \sum_i \Delta L_i \qquad \delta_{s.tot} = 9.67 \cdot mm$$

$$\delta_{s.tot.min} := \sum_j \Delta L_j \qquad \delta_{s.tot.min} = 8 \cdot mm$$

Shrinkage deformations for each storey

$kj := 11..2$

$$\delta_{s.t_k} := \left| \begin{array}{l} \delta_{s.t_{12}} \leftarrow \Delta L_{12} \\ \text{for } k \in 11..0 \\ \quad \delta_{s.t_k} \leftarrow \delta_{s.t_{k+1}} + \Delta L_k \end{array} \right.$$

$$\delta_{s.t.min_k} := \left| \begin{array}{l} \delta_{s.t.min_{12}} \leftarrow \Delta L_{12} \\ \text{for } kj \in 11..2 \\ \quad \delta_{s.t.min_{kj}} \leftarrow \delta_{s.t.min_{kj+1}} + \Delta L_{kj} \end{array} \right.$$

D4.6 Total vertical deformation

Deformation at the 14th storey $\delta_{tot} := \delta_{c.tot} + \delta_{s.tot} = 46.917 \cdot mm$

Deformation at the 12th storey $\delta_{tot.min} := \delta_{c.tot.min} + \delta_{s.tot.min} = 34.577 \cdot mm$

$\delta_{t.tot} := \delta_{s.t} + \delta_{c.t} =$

	0
0	46.917
1	44.926
2	42.807
3	39.775
4	36.437
5	33.514
6	30.173
7	26.414
8	22.238
9	17.645
10	13.705
11	9.451
12	4.883

$\cdot mm$

$\delta_{t.tot.min} := \delta_{c.t.min} + \delta_{s.t.min} =$

	0
0	34.577
1	34.577
2	34.577
3	32.513
4	30.097
5	27.838
6	25.195
7	22.171
8	18.763
9	14.973
10	11.662
11	8.063
12	4.176

$\cdot mm$

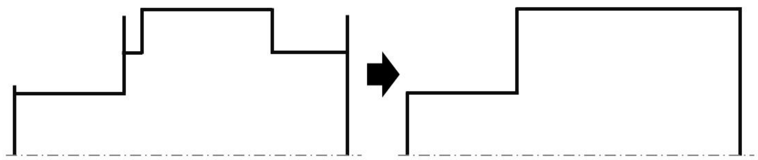
Appendix D5: Total weight of Concept 3, Concept 4 and the reference building

In this Appendix the calculations of the total weight of Concept 3, Concept 4 and the reference building is presented. The results are presented in Section 7.1.

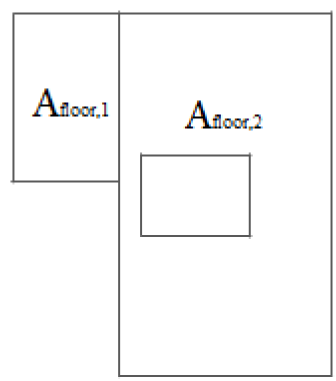
D5.1 Geometrical data

When calculating the weight of the building a simplified model of the roof is used.

$\boxed{ton := 1000kg}$



Height of building including basement	$h_{building} := 51.02m$
Total area of one storey	$A_{floor} := 865m^2$
Area of part 1	$A_{floor1} := 181m^2$
Area of part 2	$A_{floor2} := 684m^2$
Number of storeys for part 1	$n_{floor.1} := 12$
Number of storeys for part 2	$n_{floor.2} := 14$



Cross-sectional area of the concrete core	$A_{core} := 0.25m \cdot (9 \cdot 3m + 3 \cdot 2.8m + 4m + 6m + 4.4m) = 12.45\,m$
Area of the floor inside the core (concrete)	$A_{core.floor} := 26.4m^2$
Height of steel columns	$h_{col} := 3.6m$
Weight of installations	$\rho_{ins} := 30 \dfrac{kg}{m^2}$

D5.2 Weight of the reference building

Weight of concrete, C45/55 $\rho_c := 2500\frac{kg}{m^3}$

D5.2.1 Weight of columns in the reference building

The following columns are used in the reference building. SP stands for steel column and BP stands for concrete column

SP1	K-VKR 150x150
SP2	K-VKR 200x200
SP3	K-VKR 250x250
SP4	K-VKR 300x300
SP5	K-VKR 400x400
SP6	K-CKR 193.7
SP7	K-CKR 355,6
SP8	K-CKR 470

BP1	K-150x500 BTG
BP2	K-300x500 BTG
BP3	K-500x500 BTG
BP4	K-600x600 BTG
BP5	K-300x500 PREFAB
BP6	K-600x600 PREFAB

Number of columns $n_{sp2} := 10$ $n_{sp4} := 25$ $n_{sp5} := 42$

$n_{sp7} := 6$ $n_{sp8} := 6$ $n_{bp} := 6$

Weight of steel columns, VKR and KCKR. Since thicknesses are unknown, mean values are taken.

$\rho_{sp2} := 65\frac{kg}{m}$ $m_{sp2} := \rho_{sp2}\cdot h_{col}\cdot n_{sp2} = 2.34\times 10^3\,kg$

$\rho_{sp4} := 120\frac{kg}{m}$ $m_{sp4} := \rho_{sp4}\cdot h_{col}\cdot n_{sp4} = 1.08\times 10^4\,kg$

$\rho_{sp5} := 150\frac{kg}{m}$ $m_{sp5} := \rho_{sp5}\cdot h_{col}\cdot n_{sp5} = 2.268\times 10^4\,kg$

$\rho_{sp7} := 140\frac{kg}{m}$ $m_{sp7} := \rho_{sp7}\cdot h_{col}\cdot n_{sp7} = 3.024\times 10^3\,kg$

$\rho_{sp8} := 220\frac{kg}{m}$ $m_{sp8} := \rho_{sp8}\cdot h_{col}\cdot n_{sp8} = 4.752\times 10^3\,kg$

Total weight of steel columns $m_{sp.tot} := m_{sp2} + m_{sp4} + m_{sp5} + m_{sp7} + m_{sp8} = 43.596\cdot ton$

Weight of concrete columns $m_{bp.6} := 0.6m\cdot 0.6m\cdot h_{col}\cdot\rho_c = 3.24\times 10^3\,kg$

$m_{bp.tot} := m_{bp.6}\cdot n_{bp} = 19.44\cdot ton$

Total weight of the columns $m_{ref.col} := m_{sp.tot} + m_{bp.tot} = 63.036\cdot ton$

D5.2.2 Weight of floors in the reference building

Thickness of the concerete floor $t_{core.floor} := 200mm$

Thickness of the topping $t_{topping} := 30mm$

Thickness of the foundation $t_{found} := 600mm$

Weight of concrete floor $\rho_{c.floor} := 400\frac{kg}{m^2} + t_{topping} \cdot \rho_c = 475\frac{kg}{m^2}$

Weight of floors in part 1 $m_{f.1} := (\rho_{c.floor} + \rho_{ins}) \cdot A_{floor1} \cdot n_{floor.1} = 1.097 \times 10^3 \cdot ton$

Weight of floors in part 2 $m_{f.2} := (\rho_{c.floor} + \rho_{ins}) \cdot A_{floor2} \cdot n_{floor.2} = 4.836 \times 10^3 \cdot ton$

Weight of foundation slab (garage floor) $m_{found} := t_{found} \cdot \rho_c \cdot A_{floor} = 1.298 \times 10^3 \cdot ton$

Total weight of floors $m_{ref.floor} := m_{f.1} + m_{f.2} + m_{found} = 7.23 \times 10^3 \cdot ton$

D5.2.3 Weight of beams in the reference building

According to the drawings there are small differences in the dimensions of the beams and since the thickness of the material is unknown the same beam is used for the whole building in order to simplify the calculations.

Avarage weight of HSQ-beam $\rho_{HSQ} := 120\frac{kg}{m}$

Total lenght of beams in part 1 $l_{beam1} := 16.8m$

Total lenght of beams in part 2 $l_{beam2} := 27.625m$

Total weight of beams

$$m_{ref.beam} := l_{beam1} \cdot n_{floor.1} \cdot \rho_{HSQ} \cdots + l_{beam2} \cdot n_{floor.2} \cdot \rho_{HSQ} = 70.602 \cdot ton$$

D5.2.4 Weight of walls in the reference building

Height of windows $h_{window} := 1.7m$

Thickness of concrete, 2-14 th storey $t_{w.1} := 230mm$

Thickness of concrete, 1st storey $t_{w.2} := 280mm$

Thickness of concrete, basement $t_{w.3} := 330mm$

D5.2.4.1 Weight of walls on 2-11 th storey

Total length of walls on 2-11th storey and the basement $l_{wall.tot} := 37.225m + 33m + 17.6m + 10.8m \ldots + 20.025m + 22.2m = 140.85\,m$

Height of walls on 2-11 storey $h_{wall.2} := 3.6m$

Length of wall without windows $l_{wall.2} := 57.6m$

Length of wall with windows $l_{window.2} := l_{wall.tot} - l_{wall.2} = 83.25\,m$

Weight of walls on the 2-11 storey (9 storeys)

$$m_{wall.2} := \rho_c \cdot t_{w.1} \cdot 9 \cdot \left[l_{wall.2} \cdot h_{wall.2} + l_{window.2} \cdot \left(h_{wall.2} - h_{window}\right)\right] = 1.892 \times 10^3 \cdot ton$$

D5.2.4.2 Weight of walls on 12-14 th storey

Total length of walls on 12th storey $l_{wall.tot} = 140.85\,m$

Height of walls on 12th storey $h_{wall.12} := 4m$

Length of wall without windows, 12 $l_{wall.2} = 57.6\,m$

Length of wall with windows, 12 $l_{window.2} = 83.25\,m$

Total length of walls on 13-14th storey $l_{wall.top} := 118.85m$

Height of walls on 13th storey $h_{wall.13} := 3.6m$

Height of walls on 14th storey $h_{wall.14} := 4.75m$

Length of wall without windows on 13th floor $l_{wall.13} := 48.6m$

Length of wall with windows on 13th floor $l_{window.13} := l_{wall.top} - l_{wall.13} = 70.25\,m$

Length of wall without windows on 14th floor $l_{wall.14} := l_{wall.top} = 118.85\,m$

Weight of walls on the 12-14 storey

$$m_{wall.13} := \rho_c \cdot t_{w.1} \cdot \left[\begin{array}{l} l_{wall.2} \cdot h_{wall.12} + l_{window.2} \cdot \left(h_{wall.12} - h_{window}\right) \ldots \\ + l_{wall.13} \cdot h_{wall.13} + l_{window.13} \cdot \left(h_{wall.13} - h_{window}\right) + l_{wall.14} \cdot h_{wall.14} \end{array}\right]$$

$$m_{wall.13} = 744.537 \cdot ton$$

D5.2.4.3 Weight of walls on 1st storey

Total length of walls on 1 storey	$l_{wall.bottom} := l_{wall.tot} - 48.425m = 92.425\,m$
Height of walls on 1st storey	$h_{wall.1} := 4.3m$
Length of wall without windows on 1st floor	$l_{wall.1} := 37m$
Length of wall with windows on 1st floor	$l_{window.1} := l_{wall.bottom} - l_{wall.1} = 55.425\,m$

Weight of walls on the 1st storey

$$m_{wall.1} := \rho_c \cdot t_{w.2} \cdot \left[l_{wall.1} \cdot h_{wall.1} + l_{window.1} \cdot \left(h_{wall.1} - h_{window} \right) \right] = 212.243 \cdot ton$$

D5.2.4.4 Weight of walls in the basement

Total length of walls on basement	$l_{wall.tot} = 140.85\,m$
Height of walls on basement	$h_{wall.0} := 3.15m$
Length of wall without windows on basement	$l_{wall.0} := l_{wall.tot} = 140.85\,m$

Weight of walls, basement

$$m_{wall.0} := \rho_c \cdot t_{w.3} \cdot \left(l_{wall.0} \cdot h_{wall.0} \right) = 366.034 \cdot ton$$

D5.2.4.5 Total weight of walls in the reference building

Total weight of the concrete walls

$$m_{ref.wall} := m_{wall.2} + m_{wall.13} + m_{wall.1} + m_{wall.0} = 3.214 \times 10^3 \cdot ton$$

D5.2.5 Weight of roof of the reference buildling

Weight of roof (over part 2)	$\rho_{roof} := 456 \frac{kg}{m^2}$
Weight of balcony roof (over part 1)	$\rho_{b.roof} := 406 \frac{kg}{m^2}$

$$m_{roof} := \rho_{roof} \cdot A_{floor2} + \rho_{b.roof} \cdot A_{floor1} = 385.39 \cdot ton$$

D5.2.6 Weight of the core in the reference building

Weight of the core $m_{core.wall} := A_{core} \cdot \rho_c \cdot h_{building} = 1.588 \times 10^3 \cdot ton$

Weight of the floor in the core $m_{core.floor} := A_{core.floor} \cdot t_{core.floor} \cdot 14 \cdot \rho_c = 184.8 \cdot ton$

Total weight of the core $m_{core} := m_{core.wall} + m_{core.floor} = 1.773 \times 10^3 \cdot ton$

D5.2.7 Total weight of the reference building

$$m_{ref.tot} := m_{ref.col} + m_{ref.floor} + m_{ref.beam} + m_{ref.wall} + m_{roof} + m_{core} = 1.274 \times 10^4 \cdot ton$$

$$g_{ref.tot} := m_{ref.tot} \cdot g = 124.903 \cdot MN$$

D5.2.8 Average weight of one storey in the reference building

$$m_{ref.storey} := \frac{m_{ref.tot}}{15} = 849.102 \cdot ton \qquad g_{ref.storey} := m_{ref.storey} \cdot g = 8.327 \cdot MN$$

D5.3 Weight of Concept 3 and Concept 4

Number of floors between 2nd and 5th floor $n_2 := 4$

Number of floors between 6nd and 10th floor $n_6 := 5$

Number of floors between 11nd and 14th floor $n_{11} := 4$

D5.3.1 Weight of columns for Concept 3 and Concept 4

The amount and cross-section of the columns are the same for both concepts.

Area of timber columns

2nd - 5th storey $A_{t.col.2} := 0.43 \cdot 0.54m^2 = 0.232\,m^2$

6th-10th storey $A_{t.col.6} := 0.43 \cdot 0.405m^2 = 0.174\,m^2$

11th-14th storey $A_{t.col.11} := 0.33 \cdot 0.36m^2 = 0.119\,m^2$

Weight of glulam, Lc40 $\rho_{gl} := 440 \frac{kg}{m^3}$

Number of timber columns per floor $n_{t.col} := 10$

Weight of timber columns

2-5th storey $m_{t.col.2} := \rho_{gl} \cdot A_{t.col.2} \cdot h_{col} \cdot n_{t.col} \cdot n_2 = 14.712 \cdot ton$

6-10th storey $m_{t.col.6} := \rho_{gl} \cdot A_{t.col.6} \cdot h_{col} \cdot n_{t.col} \cdot n_6 = 13.793 \cdot ton$

11-14th storey $m_{t.col.11} := \rho_{gl} \cdot A_{t.col.11} \cdot h_{col} \cdot n_{t.col} \cdot n_{11} = 7.527 \cdot ton$

Total weight of timber columns $m_{t.col} := m_{t.col.2} + m_{t.col.6} + m_{t.col.11} = 36.032 \cdot ton$

Weight of steel columns (the same as in the reference building on entrance storey) $m_{sp7} = 3.024 \cdot ton$

$m_{sp8} = 4.752 \cdot ton$

$m_{sp.tot3} := m_{sp7} + m_{sp8} = 7.776 \cdot ton$

Weight of concrete columns (the same as in the reference building) $m_{bp.tot} = 19.44 \cdot ton$

Total weight of the columns $m_{3.col} := m_{sp.tot3} + m_{bp.tot} + m_{t.col} = 63.248 \cdot ton$

D5.3.2 Weight of floors for Concept 3 and Concept 4

The same floor is used for both concepts.

Weight of timber cassette floor $\rho_{t.floor} := 100 \frac{kg}{m^2}$

Weight of floors in part 1 $m_{t.f.1} := \left(\rho_{t.floor} + \rho_{ins}\right) \cdot A_{floor1} \cdot n_{floor.1} = 282.36 \cdot ton$

Weight of floors in part 2 $m_{t.f.2} := \left(\rho_{t.floor} + \rho_{ins}\right) \cdot A_{floor2} \cdot n_{floor.2} = 1.245 \times 10^3 \cdot ton$

Weight of foundation slab (same as for reference buildilng) $m_{found} = 1.298 \times 10^3 \cdot ton$

Total weight of floors $m_{t.floor} := m_{t.f.1} + m_{t.f.2} + m_{found} = 2.825 \times 10^3 \cdot ton$

D5.3.3 Weight of walls for Concept 3 and Concept 4

The same CLT walls are used for both concepts.

Weight of CLT $\rho_{CLT} := 400\frac{kg}{m^3}$

Height of windows $h_{window} = 1.7\,m$

For the thickness an additional thickness of 50 mm has been added since the load bearing part of the wall has an outer layer of non load bearing CLT. The weight of the insulation between is neglected.

Thickness of timber walls

2nd-5th storey $t_{tw.2} := 309mm$

6th-10th storey $t_{tw.6} := 271mm$

11th-14th storey $t_{tw.11} := 208mm$

Thickness of concrete, 1st storey $t_{w.2} = 280\cdot mm$

Thickness of concrete, basement $t_{w.3} = 330\cdot mm$

D5.3.3.1 Weight of walls on the 2-5 th storey

Total length of walls on 2-5th storey and the basement $l_{wall.tot} = 140.85\,m$

Height of walls on 2-5 storey $h_{wall.2} = 3.6\,m$

Length of wall without windows $l_{wall.2} = 57.6\,m$

Length of wall with windows $l_{window.2} = 83.25\,m$

Weight of walls, 2-5th storey

$$m_{t.wall.2} := \rho_{CLT}\cdot t_{tw.2}\cdot n_2\cdot\left[l_{wall.2}\cdot h_{wall.2} + l_{window.2}\cdot\left(h_{wall.2} - h_{window}\right)\right] = 180.721\cdot ton$$

D5.3.3.2 Weight of walls on the 6-10 th storey

Total length of walls on 6-10th storey and the basement $l_{wall.tot} = 140.85\,m$

Height of walls on 6-10 storey $h_{wall.2} = 3.6\,m$

Length of wall without windows $l_{wall.2} = 57.6\,m$

Length of wall with windows $l_{window.2} = 83.25\,m$

Weight of walls, 6-10th storey

$$m_{t.wall.6} := \rho_{CLT} \cdot t_{tw.6} \cdot n_6 \cdot \left[l_{wall.2} \cdot h_{wall.2} + l_{window.2} \cdot \left(h_{wall.2} - h_{window} \right) \right] = 198.12 \cdot ton$$

D5.3.3.3 Weight of walls on the 11-14 th storey

Total length of walls on 11-12th storey and the basement $l_{wall.tot} = 140.85\,m$

Height of walls on 11th and 13 th storey $h_{wall.2} = 3.6\,m$

Height of walls on 12th storey $h_{wall.12} = 4\,m$

Height of walls on 14th storey $h_{wall.14} = 4.75\,m$

Length of wall without windows $l_{wall.2} = 57.6\,m$

Length of wall with windows $l_{window.2} = 83.25\,m$

Total length of walls on 11-12th storey $l_{wall.tot} = 140.85\,m$

Length of wall without windows, 11-12 $l_{wall.2} = 57.6\,m$

Length of wall with windows, 11-12 $l_{window.2} = 83.25\,m$

Total length of walls on 13-14th storey $l_{wall.top} = 118.85\,m$

Length of wall without windows on 13th floor $l_{wall.13} = 48.6\,m$

Length of wall with windows on 13th floor $l_{window.13} = 70.25\,m$

Length of wall without windows on 14th floor $l_{wall.14} = 118.85\,m$

Weight of walls, 11-14 storey

$$m_{twall.11} := \rho_{CLT} \cdot t_{tw.11} \cdot \left[l_{wall.2} \cdot h_{wall.2} + l_{window.2} \cdot \left(h_{wall.2} - h_{window} \right) \right] = 30.413 \cdot ton$$

$$m_{twall.12} := \rho_{CLT} \cdot t_{tw.11} \cdot \left[l_{wall.2} \cdot h_{wall.12} + l_{window.2} \cdot \left(h_{wall.12} - h_{window} \right) \right] = 35.1 \cdot ton$$

$$m_{twall.13} := \rho_{CLT} \cdot t_{tw.11} \cdot \left[l_{wall.13} \cdot h_{wall.2} + l_{window.13} \cdot \left(h_{wall.2} - h_{window} \right) \right] = 25.662 \cdot ton$$

$$m_{twall.14} := \rho_{CLT} \cdot t_{tw.11} \cdot \left(l_{wall.14} \cdot h_{wall.14} \right) = 46.97 \cdot ton$$

D5.3.3.4 Total weight of timber walls

$$m_{t.wall.tot} := m_{t.wall.2} + m_{t.wall.6} + m_{twall.11} + m_{twall.12} + m_{twall.13} + m_{twall.14} = 516.984 \cdot ton$$

D5.3.3.5 Weight of 1st storey and the basement

Entrance floor (same as reference buidling) $m_{wall.1} = 212.243 \cdot ton$

Basement floor (same as reference buidling) $m_{wall.0} = 366.034 \cdot ton$

D5.3.3.6 Total weight of walls for Concept 3 and Concept 4

Total weight of walls for concept 3 and 4

$$m_{t.wall} := m_{t.wall.tot} + m_{wall.1} + m_{wall.0} = 1.095 \times 10^{3} \cdot ton$$

D5.3.4 Weight of the roof

Weight of roof structure (same as for reference building) $m_{roof} = 385.39 \cdot ton$

D5.3.5 Weight of the core

Total weight of the core (same as for reference buliding) $m_{core} = 1.773 \times 10^{3} \cdot ton$

D5.3.6 Weight of beams in Concept 3

Total length of steel beams per floor $l_{s.beam} := 16.8m$

Total length of timber beams per floor $l_{t.beam} := 55.25m$

Weight of LVL, Kerto-S $\rho_{lvl} := 510\frac{kg}{m^3}$

Cross-section area of timber roof beam $A_{r.beam} := 0.8 \cdot 0.225m^2 = 0.18\,m^2$

Cross-section area of timber office beam $A_{o.beam} := 0.65 \cdot 0.225m^2 = 0.146\,m^2$

Weight of steel columns $\rho_{HEA650} := 190\frac{kg}{m}$

$\rho_{HEA500} := 155\frac{kg}{m}$

Total weight of beams

$$m_{beam.3} := \rho_{lvl} \cdot l_{t.beam} \cdot \left[\left(n_{floor.2} - 1\right) \cdot A_{o.beam} + A_{r.beam}\right] \ldots + l_{s.beam} \cdot \left[\rho_{HEA500} \cdot \left(n_{floor.1} - 1\right) + \rho_{HEA650}\right] = 90.48 \cdot ton$$

D5.3.7 Weight of beams in concept 4

Total lenght of steel beams in part 1 $l_{s.beam1} := 16.8m$

Total lenght of steel beams in part 2 $l_{s.beam2} := 55.25m$

Weight of steel columns

Balcony beam $\rho_{HEA650} = 190\frac{kg}{m}$

Roof beam $\rho_{HEA500} = 155\frac{kg}{m}$

Office beam $\rho_{HEA450} := 140\frac{kg}{m}$

Total weight of beams

$$m_{beam.4} := l_{s.beam1} \cdot \left[\rho_{HEA450} \cdot \left(n_{floor.1} - 1\right) + \rho_{HEA650}\right] \ldots + l_{s.beam2} \cdot \left[\rho_{HEA450} \cdot \left(n_{floor.2} - 1\right) + \rho_{HEA500}\right] = 138.183 \cdot ton$$

D5.3.8 Total weight concept 3

$m_{3.tot} := m_{t.col} + m_{t.floor} + m_{beam.3} + m_{t.wall} + m_{roof} + m_{core} = 6.205 \times 10^{3} \cdot ton$

$g_{3.tot} := m_{3.tot} \cdot g = 60.847 \cdot MN$

D5.3.9 Average weight of one storey for concept 3

$m_{3.storey} := \dfrac{m_{3.tot}}{15} = 413.647 \cdot ton$ $\qquad g_{3.storey} := m_{3.storey} \cdot g = 4.056 \cdot MN$

D5.3.10 Total weight concept 4

$m_{4.tot} := m_{t.col} + m_{t.floor} + m_{beam.4} + m_{t.wall} + m_{roof} + m_{core} = 6.252 \times 10^{3} \cdot ton$

$g_{4.tot} := m_{4.tot} \cdot g = 61.315 \cdot MN$

D5.3.11 Average weight of one storey for concept 4

$m_{4.storey} := \dfrac{m_{4.tot}}{15} = 416.827 \cdot ton$ $\qquad g_{4.storey} := m_{4.storey} \cdot g = 4.088 \cdot MN$

D5.4 Summary

Total weight of referenece building	$m_{ref.tot} = 1.274 \times 10^{4} \cdot ton$	$g_{ref.tot} = 124.903 \cdot MN$
Total weight of Concept 3	$m_{3.tot} = 6.205 \times 10^{3} \cdot ton$	$g_{3.tot} = 60.847 \cdot MN$
Total weight of Concept 4	$m_{4.tot} = 6.252 \times 10^{3} \cdot ton$	$g_{4.tot} = 61.315 \cdot MN$
Average weight of referenece building	$m_{ref.storey} = 849.102 \cdot ton$	$g_{ref.storey} = 8.327 \cdot MN$
Average weight of Concept 3	$m_{3.storey} = 413.647 \cdot ton$	$g_{3.storey} = 4.056 \cdot MN$
Average weight of Concept 4	$m_{4.storey} = 416.827 \cdot ton$	$g_{4.storey} = 4.088 \cdot MN$

Appendix D6: Design of floor with regard to lateral loads, from east

$kNm := kN \cdot m$

This Appendix shows the performed calculations when checking the floors load bearing capacity with regard to lateral loads from east, the results are presented in section 7.4.

D6.1 Geometric conditions and material properties for the model

The floor is modelled as a beam subjected to a uniformly distributed load. The beam model is supported by two supports.

Beam length (=total length of the floor) $l_{beam} := 37.225m$

Beam width (=total width of the floor) $w_{beam} := 21.4m$

Length of the first cantiliver $l_{1.beam} := 14.2m$

Span between the supports $l_f := 8.7m$

Thickness of floor plate $t_{floor} := 73mm$

Width of one floor element $w_{floor} := 2.4m$

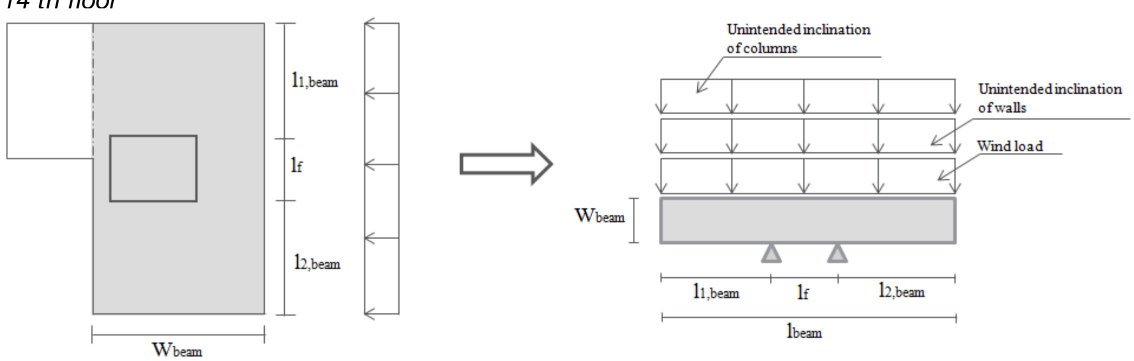
Lenght of stabilising part of core $l_{core} := 6m$

Lengt of the total core $l_{core.tot} := 11m$

Modulus of elasticity
(from Massivträ Handboken) $E_{floor.y} := 6700MPa$

Second moment of inertia
14th floor $I_{floor.y} := \frac{t_{floor} \cdot w_{beam}^3}{12} = 59.619\,m^4$

Stiffness of the floor around y
14 th floor $EI_y := E_{floor.y} \cdot I_{floor.y} = 3.994 \times 10^{11} \cdot N \cdot m^2$



Charachteristic density of CLT $\rho_k := 400 \frac{kg}{m^3}$

D6.2 Horizontal loads

D6.2.1 Wind load

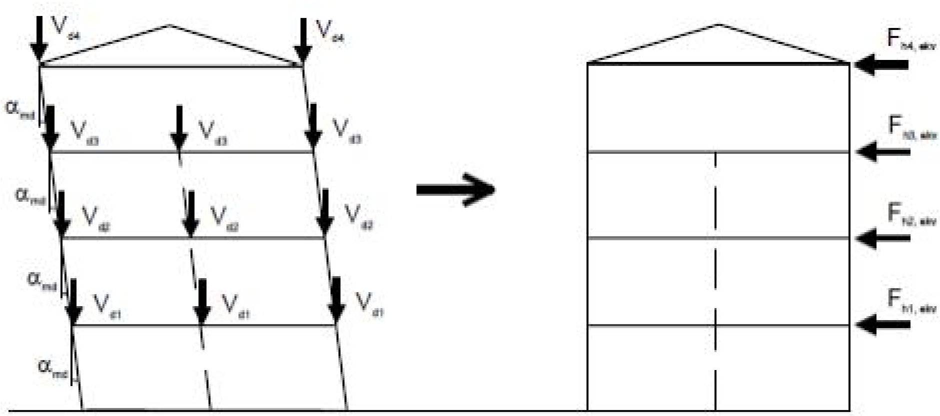
The value for the wind load is taken from Appendix C1

Total wind pressure when wind from east, zone 1 $w_e := 1.515 \cdot 10^3 Pa$

Influence height for floor 14 $h_{14} := 4.175m$

Wind load on the 14th floor slab $H_{wind.14} := w_e \cdot h_{14} = 6.325 \cdot \frac{kN}{m}$

D6.2.2 Forces due to unintended inclination



$H_u := V_d \cdot n \cdot \alpha_{md}$ $\alpha_{md} := \alpha_0 + \frac{\alpha_d}{\sqrt{n}}$

Number of supporting walls/columns subjected to vertical load on each floor

Walls $n_{wall} := 2$

Columns $n_{col} := 8$

	$n := n_{wall} + n_{col} = 10$
Systemetic part of the angle	$\alpha_0 := 0.003$
Random part of the angle	$\alpha_d := 0.012$
Unintended inclination angle	$\alpha_{md} := \alpha_0 + \frac{\alpha_d}{\sqrt{n}} = 6.795 \times 10^{-3}$
Height of 14th storey	$h_{floor} := 4.75m$

D6.2.2.1 Self-weight of floors that are taken by columns and walls

Mean tributrary are for concept 3	$A_{mean.trib.3} := 35.3m^2$
Mean tributrary are for concept 4	$A_{mean.trib.4} := 37.2m^2$
Mean influence lenght for walls	$l_{infl.mean.wall} := 3.24m$
Self-weight of floor structure	$g_{floor} := 1\frac{kN}{m^2}$
Self-weight of installations	$g_{ins} := 0.3\frac{kN}{m^2}$

D6.2.2.2 Self-weight of beams that are taken by columns

Self-weight of beam, concept 3 *Kerto-S, roof beam*	$g_{lvl} := 510\frac{kg}{m^3}\cdot 0.8m\cdot 0.225m$
Average influence length	$l_{infl.3} := 4.75m$
Average self weight of beams	$G_{beam.3} := l_{infl.3}\cdot g\cdot g_{lvl} = 4.276\cdot kN$
Self-weight of beam, concept 4 *HEA500, roof beam*	$g_{HEA} := 140\frac{kg}{m}\cdot g = 1.373\cdot\frac{kN}{m}$
Average influence length	$l_{infl.4} := 5.01m$
Average self weight of beams	$G_{beam.4} := g_{HEA}\cdot l_{infl.4} = 6.878\cdot kN$

Self-weight of columns $\rho_{glulam} := 440\,\frac{kg}{m^3}$

$G_{col} := \rho_{glulam} \cdot 0.36m \cdot 0.33m \cdot h_{floor} \cdot g = 2.435 \cdot kN$

Self-weight of walls $\rho_{CLT} := 4\,\frac{kN}{m^3}$

$t_{wall} := 158mm$

$G_{wall} := \rho_{CLT} \cdot h_{floor} \cdot t_{wall} = 3.002 \cdot \frac{kN}{m}$

Office load $q_{office} := 2.5\,\frac{kN}{m^2}$

Loads from partition walls $q_{part} := 0.5\,\frac{kN}{m^2}$

D6.2.2.3 Load combination, ULS

Load on columns in concept 3, wind load as main load

6.10a

$$V_{d.3.c.a} := 1.35 \cdot \left[\left(g_{floor} + g_{ins}\right) \cdot A_{mean.trib.3} + G_{beam.3} + G_{col}\right] \dots + 1.5 \cdot 0.7 \cdot \left(q_{office} + q_{part}\right) \cdot A_{mean.trib.3} = 182.206 \cdot kN$$

6.10b

$$V_{d.3.c.b} := 1.35 \cdot 0.89 \cdot \left[\left(g_{floor} + g_{ins}\right) \cdot A_{mean.trib.3} + G_{beam.3} + G_{col}\right] \dots + 1.5 \cdot 0.7 \cdot \left(q_{office} + q_{part}\right) \cdot A_{mean.trib.3} = 174.395 \cdot kN$$

$$V_{d.3.c} := \max\left(V_{d.3.c.a}, V_{d.3.c.b}\right) = 182.206 \cdot kN$$

Load on columns in concept 4, wind load as main load

6.10a

$$V_{d.4.c.a} := 1.35 \cdot \left[\left(g_{floor} + g_{ins}\right) \cdot A_{mean.trib.4} + G_{beam.4} + G_{col}\right] \dots + 1.5 \cdot 0.7 \cdot \left(q_{office} + q_{part}\right) \cdot A_{mean.trib.4} = 195.039 \cdot kN$$

6.10b

$$V_{d.4.c.b} := 1.35 \cdot 0.89 \cdot \left[\left(g_{floor} + g_{ins}\right) \cdot A_{mean.trib.4} + G_{beam.4} + G_{col}\right] \dots + 1.5 \cdot 0.7 \cdot \left(q_{office} + q_{part}\right) \cdot A_{mean.trib.4} = 186.474 \cdot kN$$

$$V_{d.4.c} := \max\left(V_{d.4.c.a}, V_{d.4.c.b}\right) = 195.039 \cdot kN$$

Load on walls in concept 3 and 4, wind load as main load

6.10a

$$V_{d.w.a} := 1.35 \cdot \left[\left(g_{floor} + g_{ins}\right) \cdot l_{infl.mean.wall} + G_{wall}\right] \dots + 1.5 \cdot 0.7 \cdot \left(q_{office} + q_{part}\right) \cdot l_{infl.mean.wall} = 19.945 \cdot \frac{kN}{m}$$

6.10b

$$V_{d.w.b} := 1.35 \cdot 0.89 \cdot \left[\left(g_{floor} + g_{ins}\right) \cdot l_{infl.mean.wall} + G_{wall}\right] \dots + 1.5 \cdot .07 \cdot \left(q_{office} + q_{part}\right) \cdot l_{infl.mean.wall} = 9.688 \cdot \frac{kN}{m}$$

$$V_{d.w} := \max\left(V_{d.w.a}, V_{d.w.b}\right) = 19.945 \cdot \frac{kN}{m}$$

D6.2.3 Horizontal loads

Horizontal loads due to unintended inclination

Columns concept 3 $$H_{u.3} := \frac{V_{d.3.c} \cdot n_{col} \cdot \alpha_{md}}{l_{beam}} = 0.266 \cdot \frac{kN}{m}$$

Columns concept 4 $$H_{u.4} := \frac{V_{d.4.c} \cdot n_{col} \cdot \alpha_{md}}{l_{beam}} = 0.285 \cdot \frac{kN}{m}$$

Walls concept 3 and 4 $$H_{u.w} := V_{d.w} \cdot n_{wall} \cdot \alpha_{md} = 0.271 \cdot \frac{kN}{m}$$

Wind load,
design value (wind as main load) $$H_{w.14} := 1.5 \cdot H_{wind.14} = 9.488 \cdot \frac{kN}{m}$$

D6.3 Moment and shear forces the "floor beam" should resist and transfere between two elements

D6.3.1 Maximum moments in the beam

D6.3.1.1 Support moments

The support moments are statically determined due to the cantilivers on each side of the supports.

Support moment for the 14th floor

$$M_{s.3} := \frac{\left(H_{w.14} + H_{u.w} + H_{u.3}\right) \cdot l_{1.beam}^{2}}{2} = 1.011 \times 10^{3} \cdot kNm$$

$$M_{s.4} := \frac{\left(H_{w.14} + H_{u.w} + H_{u.4}\right) \cdot l_{1.beam}^{2}}{2} = 1.013 \times 10^{3} \cdot kNm$$

The concepts is resulting in almoast the same load effects, the worst concept is used.

The first support moment $\quad M_s := M_{s.4} = 1.013 \times 10^{3} \cdot kNm$

D6.3.1.2 Field moment

Support moment almoast the same, therefore calcualting the field moment in the middle of the span by using supoerposition method. The field moment from a simply supported beam minus the support moment.

$$M_f := \frac{\left(H_{w.14} + H_{u.w} + H_{u.4}\right) \cdot l_f^{2}}{8} - M_s = -917.565 \cdot kNm$$

D6.3.2 Resisting force couple for the moments

$$F_{MEd.s} := \frac{M_s}{0.8 w_{beam}} = 59.147 \cdot kN$$

$$F_{MEd.f} := \left| \frac{M_f}{0.8 w_{beam}} \right| = 53.596 \cdot kN$$

These forces are the tension and compression forces at the top and bottom of the beam model that resists the applied moment. Because of the fact that the wind can blow from both sides the signs of the forces are not of interest, it is understood that they can be both compression and tension.

D6.3.2.1 Design of steel edge-beam

Steel quality $f_{yk} := 355MPa$ $\gamma_{M1} := 1$

$$f_{yd} := \frac{f_{yk}}{\gamma_{M1}} = 355 \cdot MPa$$

Elastic modulus $E_{steel} := 210GPa$

Dimensions $w_{steel} := 6mm$

$$h_{steel} := 65mm$$

$$A_{steel} := w_{steel} \cdot h_{steel} = 3.9 \times 10^{-4} m^2$$

Capacity $N_{Rd} := A_{steel} \cdot f_{yd} = 138.45 \cdot kN$

D6.3.3 Strain in the edge beam

Applied stress to the steel $\sigma_{steel} := \frac{F_{MEd.s}}{A_{steel}} = 151.658 \cdot MPa$

Elastic strain in the steel $\varepsilon_{steel} := \frac{\sigma_{steel}}{E_{steel}} = 7.222 \times 10^{-4}$

Elongation of the steel edge beam $\delta_{steel} := \varepsilon_{steel} \cdot l_{beam} = 26.883 \cdot mm$

D6.3.4 Utilisation ratio for edge beam

Utilisation of tension capacity $u_t := \frac{F_{MEd.s}}{N_{Rd}} = 42.721 \cdot \%$

D6.3.5 Shear forces in joints between floor elements

D6.3.5.1 Johansens's equations

Length of nail $l_{nail} := 75mm$

Diameter of nail $d := 5mm$

Penetration lengths $t_1 := 45mm$

$t_2 := l_{nail} - t_1 = 0.03\,m$ > $8 \cdot d = 0.04\,m$ OK!

Ultimate strength $f_{u.nail} := 600MPa$

Need of pre-drilling $t_{drill} := \max\left[7 \cdot \frac{d}{mm}, \left(13\frac{d}{mm} - 30\right) \cdot \frac{\rho_k}{400\frac{kg}{m^3}}\right] = 35$

$$\begin{vmatrix} \text{"Pre-drill"} \quad \text{if } \frac{t_1}{mm} < t_{drill} \\ \text{"No need for pre-drilling"} \quad \text{otherwise} \end{vmatrix} = \text{"No need for pre-drilling"}$$

Embedment strength for first web (t1) $f_{h.1.k} := 0.082 \cdot \frac{\rho_k}{\frac{kg}{m^3}} \cdot \left(\frac{d}{mm}\right)^{-0.3} \cdot MPa = 20.239 \cdot MPa$

Embedment strength for second web (t2) $f_{h.2.k} := 0.082 \cdot \frac{\rho_k}{\frac{kg}{m^3}} \cdot \left(\frac{d}{mm}\right)^{-0.3} \cdot MPa = 20.239 \cdot MPa$

Ratio between embedment strengths $\beta := \frac{f_{h.2.k}}{f_{h.1.k}} = 1$

Characteristic yield moment for the nail $M_{y.Rk} := 0.3 \cdot f_{u.nail} \cdot d^{2.6} \cdot m^{0.4} = 1.873 \times 10^5 \cdot N \cdot mm$

$F_{v.Rk.1} := f_{h.1.k} \cdot t_1 \cdot d = 4.554 \cdot kN$

$F_{v.Rk.2} := f_{h.2.k} \cdot t_2 \cdot d = 3.036 \cdot kN$

$$F_{v.Rk.3} := \frac{f_{h.1.k} \cdot t_1 \cdot d}{1 + \beta} \cdot \left[\sqrt{\beta + 2 \cdot \beta^2 \left[1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1} \right)^2 \right] + \beta^3 \cdot \left(\frac{t_2}{t_1} \right)^2} - \beta \cdot \left(1 + \frac{t_2}{t_1} \right) \right] = 1.625 \cdot kN$$

$$F_{v.Rk.4} := 1.05 \cdot \frac{f_{h.1.k} \cdot t_1 \cdot d}{2 + \beta} \cdot \left[\sqrt{2 \cdot \beta \cdot (1 + \beta) + \frac{4 \cdot \beta \cdot (2 + \beta) \cdot M_{y.Rk}}{f_{h.1.k} \cdot d \cdot t_1^2}} - \beta \right] = 4.573 \cdot kN$$

$$F_{v.Rk.5} := 1.05 \cdot \frac{f_{h.1.k} \cdot t_2 \cdot d}{1 + 2\beta} \cdot \left[\sqrt{2 \cdot \beta^2 \cdot (1 + \beta) + \frac{4 \cdot \beta \cdot (1 + 2\beta) \cdot M_{y.Rk}}{f_{h.1.k} \cdot d \cdot t_2^2}} - \beta \right] = 4.628 \cdot kN$$

$$F_{v.Rk.6} := 1.15 \cdot \sqrt{\frac{2 \cdot \beta}{1 + \beta}} \cdot \sqrt{2 \cdot M_{y.Rk} \cdot f_{h.1.k} \cdot d} = 7.081 \cdot kN$$

Lateral capacity of one fastener

$$F_{v.Rk} := \min\left(F_{v.Rk.1}, F_{v.Rk.2}, F_{v.Rk.3}, F_{v.Rk.4}, F_{v.Rk.5}, F_{v.Rk.6}\right) = 1.625 \cdot kN$$

Strength modification factor, short term load $\quad k_{mod} := 0.9$

Partial factor for solid wood $\quad \gamma_M := 1.3$

$$F_{v.Rd} := k_{mod} \cdot \frac{F_{v.Rk}}{\gamma_M} = 1.125 \cdot kN$$

D6.3.5.2 Minimum spacing

Angle between force and grain direction $\quad \alpha_f := 0$

For a ρ.k < 420 kg/m^3

Spacing parallel to grain $\quad a_1 := \left(5 + 5 \cdot \left|\cos\left(\alpha_f\right)\right|\right) \cdot d = 50 \cdot mm$

Spacing perpendicular to grain $\quad a_2 := 5 \cdot d = 25 \cdot mm$

Distance to loaded end $\quad a_{3.t} := \left(10 + 5 \cdot \cos\left(\alpha_f\right)\right) \cdot d = 75 \cdot mm$

Distance to unloaded end $\quad a_{3.c} := 10d = 50 \cdot mm$

Distance to loaded edge $\quad a_{4.t} := \left(5 + 2 \cdot \sin\left(\alpha_f\right)\right) \cdot d = 25 \cdot mm$

Distance to unloaded edge $\quad a_{4.c} := 5d = 25 \cdot mm$

D6.3.5.3 Shear force in one connector and utilisation ratio

Shear force in the most loaded floor joint

$$F_{joint} := \frac{\left(H_{w.14} + H_{u.w} + H_{u.4}\right)}{w_{beam}} l_{1.beam} - \frac{\left(H_{w.14} + H_{u.w} + H_{u.4}\right)}{w_{beam}} \cdot w_{floor} = 5.538 \cdot \frac{kN}{m}$$

Spacing between connectors (nails) $s_{nail} := 200mm$

Applied force on each connector $F_{v.Ed} := F_{joint} \cdot s_{nail} = 1.108 \cdot kN$

Utilisation ratio
(having two rows of nails) $u_{nail} := \frac{F_{v.Ed}}{2F_{v.Rd}} = 0.492$

D6.4 Shear forces the floor beam should transfer to the core

Shear force between the floor and the concrete core $F_{core} := \frac{\left(H_{w.14} + H_{u.w} + H_{u.4}\right)}{l_{core.tot}} l_{1.beam} = 12.965 \cdot \frac{kN}{m}$

Capacity of the connector (assumed capacity from screw FBS 10 A4 from Fischer) $F_{Rd} := 13.3kN$

http://www.fischersverige.se/PortalData/10/Resources/fischer_se/katalog_pdf/stal_infastning/_dokument/Betongskruv_FBS.pdf

Spacing between the connectors $s := 250mm$

Applied force on each conncetor $F_{Ed} := F_{core} \cdot s = 3.241 \cdot kN$

Utilisation ratio $u_{screw} := \frac{F_{core} \cdot s}{F_{Rd}} = 24.371 \cdot \%$

Spacing between the timber connectors $s_{timb} := 150mm$

Applied force on each timber conncetor $F_{Ed.timb} := F_{core} \cdot s_{timb} = 1.945 \cdot kN$

Utilisation ratio $u_{nails} := \frac{F_{core} \cdot s_{timb}}{2 \cdot F_{v.Rd}} = 86.422 \cdot \%$

D6.7 Additional calculation of unintended inclination for the reference building and the concepts

In this Chapter, the equivalent force of the unintended inclination for the reference building and the concepts are calculated. The forces are not used in this Appendix but in Appendix D8 to calculate the moment in the core and the horisontal deflection of the core from lateral forces.

These calculations were performed in this Appendix because all the facts about loads, influence lengths and tributary areas and so on are defined in this Appendix.

Number of columns, and total number of vertical loaded members $n_{col.ref} := 6$ $\quad n_{ref} := n_{wall} + n_{col.ref} = 8$

Unintended inclination angle $\alpha_{md.ref} := \alpha_0 + \dfrac{\alpha_d}{\sqrt{n_{ref}}} = 7.243 \times 10^{-3}$

Self-weight of the floor $g_{floor.ref} := 4.75 \dfrac{kN}{m^2}$

Self-weight of the wall $G_{wall.ref} := 14.634 \dfrac{kN}{m}$

Self-weight of the columns $G_{col.ref} := g \cdot 154 \dfrac{kg}{m} \cdot 3.6m = 5.437 \cdot kN$

Tributary area for the columns $A_{mean.trib.ref} := 60m^2$

Influence length for the walls $l_{infl.ref.wall} := 5.375m$

D6.7.1 Horizontal load from unintended inclination for self-weight as unfavourable for the Reference building

Load on columns, wind load as main load

$$V_{un.unf.ref.col} := 1.1 \cdot \left[\left(g_{floor.ref} + g_{ins}\right) \cdot A_{mean.trib.ref} + G_{beam.4} + G_{col.ref} \right] \ldots + 1.5 \cdot 0.7 \cdot \left(q_{office} + q_{part}\right) \cdot A_{mean.trib.ref} = 535.847 \cdot kN$$

Load on walls, wind load as main load

$$V_{un.unf.ref.wall} := 1.1 \cdot \left[\left(g_{floor.ref} + g_{ins}\right) \cdot l_{infl.ref.wall} + G_{wall.ref} \right] \ldots + 1.5 \cdot 0.7 \cdot \left(q_{office} + q_{part}\right) \cdot l_{infl.ref.wall} = 62.887 \cdot \frac{kN}{m}$$

Equvivalent force from the columns

$$H_{ref.unf.col} := \frac{V_{un.unf.ref.col} \cdot n_{col.ref} \cdot \alpha_{md.ref}}{l_{beam}} = 0.626 \cdot \frac{kN}{m}$$

Equvivalent force from the walls

$$H_{ref.unf.wall} := V_{un.unf.ref.wall} \cdot n_{wall} \cdot \alpha_{md.ref} = 0.911 \cdot \frac{kN}{m}$$

Total horisontal force from unintended inclination effects

$$H_{ref.unf} := H_{ref.unf.col} + H_{ref.unf.wall} = 1.536 \cdot \frac{kN}{m}$$

D6.7.2 Horizontal load from unintended inclination for self-weight as favourable for the Reference building

Load on columns, wind load as main load

$$V_{d.ref.c.fav} := 0.9 \cdot \left[\left(g_{floor.ref} + g_{ins}\right) \cdot A_{mean.trib.ref} + G_{beam.4} + G_{col.ref}\right] \dots = 283.784 \cdot kN$$
$$+ 0 \cdot \left(q_{office} + q_{part}\right) \cdot A_{mean.trib.ref}$$

Load on walls, wind load as main load

$$V_{d.w.fav} := 0.9 \cdot \left[\left(g_{floor.ref} + g_{ins}\right) \cdot l_{infl.ref.wall} + G_{wall.ref}\right] \dots = 37.6 \cdot \frac{kN}{m}$$
$$+ 0 \cdot \left(q_{office} + q_{part}\right) \cdot l_{infl.ref.wall}$$

Equvivalent force from the columns

$$H_{ref.fav.col} := \frac{V_{d.ref.c.fav} \cdot n_{col.ref} \cdot \alpha_{md.ref}}{l_{beam}} = 0.331 \cdot \frac{kN}{m}$$

Equvivalent force from the walls

$$H_{ref.fav.wall} := V_{d.w.fav} \cdot n_{wall} \cdot \alpha_{md.ref} = 0.545 \cdot \frac{kN}{m}$$

Total horisontal force from unintended inclination effects

$$H_{ref.fav} := H_{ref.fav.col} + H_{ref.fav.wall} = 0.876 \cdot \frac{kN}{m}$$

D6.7.3 Horizontal load from unintended inclination for self-weight as unfavourable for the Concepts

$$V_{un.unf.3.col} := 1.1\cdot\left[\left(g_{floor}+g_{ins}\right)\cdot A_{mean.trib.3}+G_{beam.3}+G_{col}\right] \ldots = 169.056\cdot kN$$
$$+\,1.5\cdot 0.7\cdot\left(q_{office}+q_{part}\right)\cdot A_{mean.trib.3}$$

$$V_{un.unf.4.col} := 1.1\cdot\left[\left(g_{floor}+g_{ins}\right)\cdot A_{mean.trib.4}+G_{beam.4}+G_{col}\right] \ldots = 180.621\cdot kN$$
$$+\,1.5\cdot 0.7\cdot\left(q_{office}+q_{part}\right)\cdot A_{mean.trib.4}$$

$$V_{un.unf.wall} := 1.1\cdot\left[\left(g_{floor}+g_{ins}\right)\cdot l_{infl.mean.wall}+G_{wall}\right] \ldots = 18.141\cdot\frac{kN}{m}$$
$$+\,1.5\cdot 0.7\cdot\left(q_{office}+q_{part}\right)\cdot l_{infl.mean.wall}$$

Horizontal loads due to unintended inclination

Columns concept 3
$$H_{un.unf.3.col} := \frac{V_{un.unf.3.col}\cdot n_{col}\cdot \alpha_{md}}{l_{beam}} = 0.247\cdot\frac{kN}{m}$$

Columns concept 4
$$H_{un.unf.4.col} := \frac{V_{un.unf.4.col}\cdot n_{col}\cdot \alpha_{md}}{l_{beam}} = 0.264\cdot\frac{kN}{m}$$

Walls concept 3 and 4
$$H_{un.unf.wall} := V_{un.unf.wall}\cdot n_{wall}\cdot \alpha_{md} = 0.247\cdot\frac{kN}{m}$$

Total horizontal load due to unintended inclination
$$H_{un.unf.3} := H_{un.unf.3.col}+H_{un.unf.wall} = 0.493\cdot\frac{kN}{m}$$
$$H_{un.unf.4} := H_{un.unf.4.col}+H_{un.unf.wall} = 0.51\cdot\frac{kN}{m}$$

D6.7.3 Horizontal load from unintended inclination for self-weight as favourable for the Concepts

$$V_{un.fav.3.col} := 0.9\cdot\left[\left(g_{floor}+g_{ins}\right)\cdot A_{mean.trib.3}+G_{beam.3}+G_{col}\right] \ldots = 47.341\cdot kN$$
$$+\,0\cdot\left(q_{office}+q_{part}\right)\cdot A_{mean.trib.3}$$

$$V_{un.fav.4.col} := 0.9\cdot\left[\left(g_{floor}+g_{ins}\right)\cdot A_{mean.trib.4}+G_{beam.4}+G_{col}\right] \ldots = 51.906\cdot kN$$
$$+\,0\cdot\left(q_{office}+q_{part}\right)\cdot A_{mean.trib.4}$$

$$V_{un.fav.wall} := 0.9\cdot\left[\left(g_{floor}+g_{ins}\right)\cdot l_{infl.mean.wall}+G_{wall}\right] \ldots = 6.493\cdot\frac{kN}{m}$$
$$+\,0\cdot\left(q_{office}+q_{part}\right)\cdot l_{infl.mean.wall}$$

Horizontal loads due to unintended inclination

Columns concept 3

$$H_{un.fav.3.col} := \frac{V_{un.fav.3.col} \cdot n_{col} \cdot \alpha_{md}}{l_{beam}} = 0.069 \cdot \frac{kN}{m}$$

Columns concept 4

$$H_{un.fav.4.col} := \frac{V_{un.fav.4.col} \cdot n_{col} \cdot \alpha_{md}}{l_{beam}} = 0.076 \cdot \frac{kN}{m}$$

Walls concept 3 and 4

$$H_{un.fav.wall} := V_{un.fav.wall} \cdot n_{wall} \cdot \alpha_{md} = 0.088 \cdot \frac{kN}{m}$$

Total horizontal load due to unintended inclination

$$H_{un.fav.3} := H_{un.fav.3.col} + H_{un.fav.wall} = 0.157 \cdot \frac{kN}{m}$$

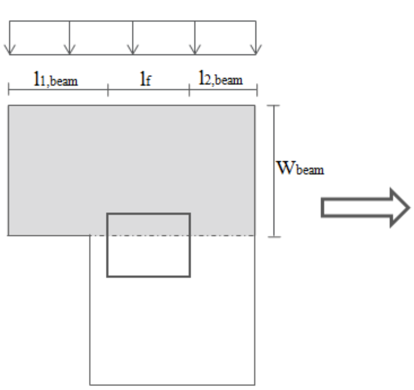
$$H_{un.fav.4} := H_{un.fav.4.col} + H_{un.fav.wall} = 0.164 \cdot \frac{kN}{m}$$

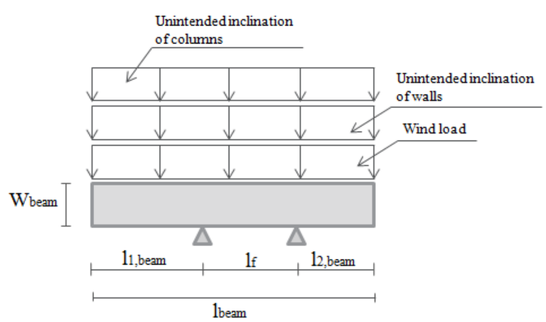
Appendix D7: Design of floor with regard to lateral loads, from north

This Appendix shows the performed calculations when checking the floors load bearing capacity with regard to lateral loads from north, the results are presented in Section 7.4.

D7.1 Geometric conditions and material properties for the model

Beam length (=total length of the floor)	$l_{beam} := 33m$
Beam width (=total width of the floor)	$w_{beam} := 17.6m$
Length of the first cantiliver	$l_{1.beam} := 12.9m$
Length of the second cantiliver	$l_{2.beam} := 8.1m$
Span between the supports	$l_f := 11m$
Thickness of floor plate	$t_{floor} := 73mm$
Width of one floor element	$w_{floor} := 2.4m$
Modulus of elasticity (from Massivträ Handboken)	$E_{floor.y} := 6700MPa$
Lenght of stabilising part of core	$l_{core} := 3.2m$
Total length of the core	$l_{core.tot} := 8.6m$
Second moment of inertia 12th floor	$I_{floor.y} := \frac{t_{floor} \cdot w_{beam}^3}{12} = 33.165\,m^4$
Stiffness of the floor around y 12 th floor	$EI_y := E_{floor.y} \cdot I_{floor.y} = 2.222 \times 10^{11} \cdot N \cdot m^2$





Charachteristic density of CLT $\rho_k := 400 \frac{kg}{m^3}$

D7.2 Horizontal loads

D7.2.1 Wind load

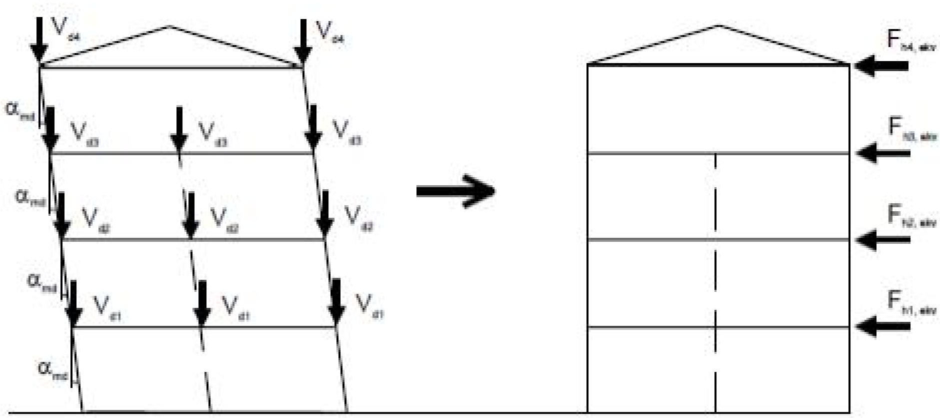
The value for the wind load is taken from Appendix C1

Total wind pressure when wind from north, zone 1 $w_n := 1.505 \times 10^3 \, Pa$

Influence height for floor 12 $h_{12} := 3.8m$

Wind load on the 12th floor slab $H_{wind} := w_n \cdot h_{12} = 5.719 \cdot \frac{kN}{m}$

D7.2.2 Forces due to unintended inclination



$H_u := V_d \cdot n \cdot \alpha_{md}$ $\alpha_{md} := \alpha_0 + \frac{\alpha_d}{\sqrt{n}}$

Number of supporting walls/columns subjected to vertical load on each floor

Walls $n_{wall} := 2$

Columns $n_{col.office} := 8$ $n_{col.bal} := 2$

$n := n_{wall} + n_{col.office} + n_{col.bal} = 12$

Systemetic part of the angle $\alpha_0 := 0.003$

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

Unintended inclination angle $\alpha_{md} := \alpha_0 + \frac{\alpha_d}{\sqrt{n}} = 6.464 \times 10^{-3}$

Height of 12h storey $h_{floor} := 4m$

D7.2.2.1 Self-weight of floors that are taken by columns

(In this case the walls are not loaded)

Mean tributrary are for concept 3 $A_{mean.trib.3} := 37.15m^2$

Mean tributrary are for concept 4 $A_{mean.trib.4} := 38.54m^2$

Self-weight of floor structure $g_{floor} := 1\frac{kN}{m^2}$

Self-weight of installations $g_{ins} := 0.3\frac{kN}{m^2}$

D7.2.2.2 Self-weight of beams that are taken by columns

Weight of beam, concept 3
Kerto-S, office beam $g_{lvl} := 510\frac{kg}{m^3} \cdot 0.65m \cdot 0.225m$

Weight of beam, concept 3
Steel, balcony beam $g_{HEA650} := 190\frac{kg}{m}$

Average influence length, timber $l_{infl.t.3} := 4.75m$

Average influence length, steel $l_{infl.s.3} := 6.35m$

Self-weight of office beams concept 3 $G_{t.beam.3} := l_{infl.t.3} \cdot g \cdot g_{lvl} = 3.474 \cdot kN$

Self-weight of balcony beams concept 3 $G_{s.beam.3} := l_{infl.s.3} \cdot g_{HEA650} \cdot g = 11.832 \cdot kN$

Weight of beam, concept 4
HEA450, office beam

$$g_{HEA450} := 140 \frac{kg}{m}$$

Weight of beam, concept 4
HEA650, balcony beam

$$g_{HEA650} = 190 \frac{kg}{m}$$

Average influence length of office beams to the columns

$$l_{infl.4.o} := 5.01m$$

Average influence length of balcony beams to the columns

$$l_{infl.4.b} := 6.15m$$

Self-weight of steel beams concept 3 office beam

$$G_{beam.4.o} := g_{HEA450} \cdot l_{infl.4.o} \cdot g = 6.878 \cdot kN$$

Self-weight of steel beams concept 3 balcony beam

$$G_{beam.4.b} := g_{HEA650} \cdot l_{infl.4.b} \cdot g = 11.459 \cdot kN$$

Self-weight of columns

$$\rho_{glulam} := 440 \frac{kg}{m^3}$$

$$G_{col} := \rho_{glulam} \cdot 0.36m \cdot 0.33m \cdot h_{floor} \cdot g = 2.05 \cdot kN$$

Self-weight of walls

$$\rho_{CLT} := 4 \frac{kN}{m^3}$$

$$t_{wall} := 158mm$$

$$G_{wall} := \rho_{CLT} \cdot h_{floor} \cdot t_{wall} = 2.528 \cdot \frac{kN}{m}$$

Office load

$$q_{office} := 2.5 \frac{kN}{m^2}$$

Loads from partition walls

$$q_{part} := 0.5 \frac{kN}{m^2}$$

D7.2.2.3 Load combination, ULS

Load on columns subjected to balcony beam in concept 3, wind load as main load

6.10a

$$V_{d.3.c.a.bal} := 1.35 \cdot \left[\left(g_{floor} + g_{ins} \right) \cdot A_{mean.trib.3} + G_{s.beam.3} + G_{col} \right] \ldots + 1.5 \cdot 0.7 \cdot \left(q_{office} + q_{part} \right) \cdot A_{mean.trib.3} = 200.962 \cdot kN$$

6.10b

$$V_{d.3.c.b.bal} := 1.35 \cdot 0.89 \cdot \left[\left(g_{floor} + g_{ins} \right) \cdot A_{mean.trib.3} + G_{s.beam.3} + G_{col} \right] \ldots + 1.5 \cdot 0.7 \cdot \left(q_{office} + q_{part} \right) \cdot A_{mean.trib.3} = 191.728 \cdot kN$$

$$V_{d.3.c.bal} := \max\left(V_{d.3.c.a.bal}, V_{d.3.c.b.bal} \right) = 200.962 \cdot kN$$

Load on the columns subjected to office beam in concept 3, wind load as main loac

6.10a

$$V_{d.3.c.a.office} := 1.35 \cdot \left[\left(g_{floor} + g_{ins} \right) \cdot A_{mean.trib.3} + G_{t.beam.3} + G_{col} \right] \ldots + 1.5 \cdot 0.7 \cdot \left(q_{office} + q_{part} \right) \cdot A_{mean.trib.3} = 189.679 \cdot kN$$

6.10b

$$V_{d.3.c.b.office} := 1.35 \cdot 0.89 \cdot \left[\left(g_{floor} + g_{ins} \right) \cdot A_{mean.trib.3} + G_{t.beam.3} + G_{col} \right] \ldots + 1.5 \cdot 0.7 \cdot \left(q_{office} + q_{part} \right) \cdot A_{mean.trib.3} = 181.687 \cdot kN$$

$$V_{d.3.c.office} := \max\left(V_{d.3.c.a.office}, V_{d.3.c.b.office} \right) = 189.679 \cdot kN$$

Load on columns subjected to balcony beam in concept 4, wind load as main load

6.10a

$$V_{d.4.c.a.bal} := 1.35 \cdot \left[\left(g_{floor} + g_{ins} \right) \cdot A_{mean.trib.4} + G_{beam.4.b} + G_{col} \right] \ldots + 1.5 \cdot 0.7 \cdot \left(q_{office} + q_{part} \right) \cdot A_{mean.trib.4} = 207.277 \cdot kN$$

6.10b

$$V_{d.4.c.b.bal} := 1.35 \cdot 0.89 \cdot \left[\left(g_{floor} + g_{ins} \right) \cdot A_{mean.trib.4} + G_{beam.4.b} + G_{col} \right] \ldots + 1.5 \cdot 0.7 \cdot \left(q_{office} + q_{part} \right) \cdot A_{mean.trib.4} = 197.83 \cdot kN$$

$$V_{d.4.c.bal} := \max\left(V_{d.4.c.a.bal}, V_{d.4.c.b.bal} \right) = 207.277 \cdot kN$$

Load on columns subjected to office beam in concept 4, wind load as main load

6.10a

$$V_{d.4.c.a.office} := 1.35 \cdot \left[\left(g_{floor} + g_{ins}\right) \cdot A_{mean.trib.4} + G_{beam.4.o} + G_{col}\right] \ldots = 201.093 \cdot kN$$
$$+ 1.5 \cdot 0.7 \cdot \left(q_{office} + q_{part}\right) \cdot A_{mean.trib.4}$$

6.10b

$$V_{d.4.c.b.office} := 1.35 \cdot 0.89 \cdot \left[\left(g_{floor} + g_{ins}\right) \cdot A_{mean.trib.4} + G_{beam.4.o} + G_{col}\right] \ldots = 192.327 \cdot kN$$
$$+ 1.5 \cdot 0.7 \cdot \left(q_{office} + q_{part}\right) \cdot A_{mean.trib.4}$$

$$V_{d.4.c.office} := \max\left(V_{d.4.c.a.office}, V_{d.4.c.b.office}\right) = 201.093 \cdot kN$$

D7.2.3 Horizontal loads

Horizontal loads due to unintended inclination

Columns subjected to office beam,concept 3

$$H_{u.3.office} := \frac{V_{d.3.c.office} \cdot n_{col.office} \cdot \alpha_{md}}{l_{beam}} = 0.297 \cdot \frac{kN}{m}$$

Columns subjected to balcony beam,concept 3

$$H_{u.3.bal} := \frac{V_{d.3.c.bal} \cdot n_{col.bal} \cdot \alpha_{md}}{l_{beam}} = 0.079 \cdot \frac{kN}{m}$$

Horizontal load from unintended inclination, concept 3

$$H_{u.3} := H_{u.3.office} + H_{u.3.bal} = 0.376 \cdot \frac{kN}{m}$$

Columns subjected to office beam,concept 4

$$H_{u.4.office} := \frac{V_{d.4.c.office} \cdot n_{col.office} \cdot \alpha_{md}}{l_{beam}} = 0.315 \cdot \frac{kN}{m}$$

Columns subjected to balcony beam,concept 4

$$H_{u.4.bal} := \frac{V_{d.4.c.bal} \cdot n_{col.bal} \cdot \alpha_{md}}{l_{beam}} = 0.081 \cdot \frac{kN}{m}$$

Horizontal load from unintended inclination, concept 4

$$H_{u.4} := H_{u.4.office} + H_{u.4.bal} = 0.396 \cdot \frac{kN}{m}$$

Wind load,
design value (wind as main load)

$$H_w := 1.5 \cdot H_{wind} = 8.579 \cdot \frac{kN}{m}$$

D7.3 Moment and shear forces the floor beam should resist and transfere between two elements

D7.3.1 Maximum moments in the beam

D7.3.1.1 Support moments

The support moments are statically determined due to the cantilivers on each side of the supports.

Support moment for the 14th floor, the moment is only calculated for the longest cantilever of 12.9m

$$M_{s.3} := \frac{\left(H_w + H_{u.3}\right)\cdot l_{1.beam}^{2}}{2} = 745.056\cdot kN\cdot m$$

$$M_{s.4} := \frac{\left(H_w + H_{u.4}\right)\cdot l_{1.beam}^{2}}{2} = 746.75\cdot kN\cdot m$$

The concepts is resulting in almoast the same load effects, the worst concept is used.

The first support moment $M_s := M_{s.4} = 746.75\cdot kN\cdot m$

D7.3.1.2 Field moment

Support moment almoast the same, therefore calcualting the field moment in the middle of the span by using supoerposition method. The field moment from a simply supported beam minus the support moment.

$$M_f := \frac{\left(H_w + H_{u.4}\right)\cdot l_f^{2}}{8} - M_s = -611.006\cdot kN\cdot m$$

D7.3.2 Resisting force couple for the moments

$$F_{MEd.s} := \frac{M_s}{0.8 w_{beam}} = 53.036\cdot kN$$

$$F_{MEd.f} := \left|\frac{M_f}{0.8 w_{beam}}\right| = 43.395\cdot kN$$

These forces are the tension and compression forces at the top and bottom of the beam model that resists the applied moment. Because of the fact that the wind can blow from both sides the signs of the forces are not of interest, it is understood that they can be both compression and tension.

D7.3.2.1 Design of steel edge-beam

Steel quality $f_{yk} := 355MPa$ $\gamma_{M1} := 1$

$$f_{yd} := \frac{f_{yk}}{\gamma_{M1}} = 355 \cdot MPa$$

Elastic modulus $E_{steel} := 210GPa$

Dimensions $w_{steel} := 6mm$

$h_{steel} := 65mm$

$A_{steel} := w_{steel} \cdot h_{steel} = 3.9 \times 10^{-4} m^2$

Capacity $N_{Rd} := A_{steel} \cdot f_{yd} = 138.45 \cdot kN$

D7.3.3 Strain in the edge beam

Applied stress to the steel $\sigma_{steel} := \dfrac{F_{MEd.s}}{A_{steel}} = 135.99 \cdot MPa$

Elastic strain in the steel $\varepsilon_{steel} := \dfrac{\sigma_{steel}}{E_{steel}} = 6.476 \times 10^{-4}$

Elongation of the steel edge beam $\delta_{steel} := \varepsilon_{steel} \cdot l_{beam} = 21.37 \cdot mm$

D7.3.4 Utilisation ratio for edge beam

Utilisation of tension capacity $u_t := \dfrac{F_{MEd.s}}{N_{Rd}} = 38.307 \cdot \%$

D7.4 Shear forces the floor beam should transfer to the core

Shear force between the floor and the concrete core $F_{core} := \dfrac{\left(H_w + H_{u.3} + H_{u.4}\right)}{l_{core.tot}} l_{1.beam} = 14.026 \cdot \dfrac{kN}{m}$

Capacity of the connector (assumed capacity from screw FBS 10 A4 from Fischer) $F_{Rd} := 13.3kN$

Capacity of timber nail $F_{v.Rd} := 1.125kN$

http://www.fischersverige.se/PortalData/10/Resources/fischer_se/katalog_pdf/stal_infastning/_dokument/Betongskruv_FBS.pdf

Spacing between the concrete connectors $s_{conc} := 250mm$

Applied force on each conncetor $F_{Ed} := F_{core} \cdot s_{conc} = 3.507 \cdot kN$

Utilisation ratio $u_{screw} := \dfrac{F_{core} \cdot s_{conc}}{F_{Rd}} = 26.365 \cdot \%$

Spacing between the timber connectors $s_{timb} := 150mm$

Applied force on each conncetor $F_{Ed.timb} := F_{core} \cdot s_{timb} = 2.104 \cdot kN$

Utilisation ratio $u_{nail} := \dfrac{F_{core} \cdot s_{timb}}{2F_{v.Rd}} = 93.508 \cdot \%$

D7.5 Additional calculation of unintended inclination for the reference building and the concept

In this Chapter, the equivalent force of the unintended inclination for the reference building and the concepts are calculated. The forces are not used in this Appendix but in Appendix D8 to calculate the moment in the core and the horisontal deflection of the core from lateral forces.

These calculations were performed in this Appendix because all the facts about loads, influence lengths and tributary areas and so on are defined in this Appendix.

Number of columns, and total number of vertical loaded members $n_{col.ref} := 6$ $n_{ref} := n_{wall} + n_{col.ref} = 8$

Unintended inclination angle $\alpha_{md.ref} := \alpha_0 + \dfrac{\alpha_d}{\sqrt{n_{ref}}} = 7.243 \times 10^{-3}$

Self-weight of the floor $g_{floor.ref} := 4.75 \dfrac{kN}{m^2}$

Self-weight of the wall $G_{wall.ref} := 14.634 \dfrac{kN}{m}$

Self-weight of the columns $G_{col.ref} := g \cdot 154 \dfrac{kg}{m} \cdot 3.6m = 5.437 \cdot kN$

Tributary area for the columns $A_{mean.trib.ref} := 60m^2$

Influence length for the walls $l_{infl.ref.wall} := 5.375m$

D7.5.1 Horizontal load from unintended inclination for self-weight as unfavourable for the Reference building

Load on columns, wind load as main load

$$V_{un.unf.ref.col} := 1.1 \cdot \left[\left(g_{floor.ref} + g_{ins} \right) \cdot A_{mean.trib.ref} + G_{beam.4.b} + G_{col.ref} \right] \ldots = 540.885 \cdot kN$$
$$+ 1.5 \cdot 0.7 \cdot \left(q_{office} + q_{part} \right) \cdot A_{mean.trib.ref}$$

Equvivalent force from the columns $$H_{ref.unf.col} := \frac{V_{un.unf.ref.col} \cdot n_{col.ref} \cdot \alpha_{md.ref}}{l_{beam}} = 0.712 \cdot \frac{kN}{m}$$

Total horisontal force from unintended inclination effects $$H_{ref.unf} := H_{ref.unf.col} = 0.712 \cdot \frac{kN}{m}$$

D7.5.2 Horizontal load from unintended inclination for self-weight as favourable for the Reference building

Load on columns, wind load as main load

$$V_{un.fav.ref.col} := 0.9 \cdot \left[\left(g_{floor.ref} + g_{ins} \right) \cdot A_{mean.trib.ref} + G_{beam.4.b} + G_{col.ref} \right] \ldots = 287.906 \cdot kN$$
$$+ 0 \cdot \left(q_{office} + q_{part} \right) \cdot A_{mean.trib.ref}$$

Equvivalent force from the columns $$H_{ref.fav.col} := \frac{V_{un.fav.ref.col} \cdot n_{col.ref} \cdot \alpha_{md.ref}}{l_{beam}} = 0.379 \cdot \frac{kN}{m}$$

Total horisontal force from unintended inclination effects $$H_{ref.fav} := H_{ref.fav.col} = 0.379 \cdot \frac{kN}{m}$$

D7.5.3 Horizontal load from unintended inclination for self-weight as unfavourable for the Concepts

Concept 3

$$V_{unf.3.bal.col} := 1.1 \cdot \left[\left(g_{floor} + g_{ins} \right) \cdot A_{mean.trib.3} + G_{s.beam.3} + G_{col} \right] \ldots = 185.417 \cdot kN$$
$$+ 1.5 \cdot 0.7 \cdot \left(q_{office} + q_{part} \right) \cdot A_{mean.trib.3}$$

$$V_{unf.3.office.col} := 1.1 \cdot \left[\left(g_{floor} + g_{ins} \right) \cdot A_{mean.trib.3} + G_{t.beam.3} + G_{col} \right] \ldots = 176.224 \cdot kN$$
$$+ 1.5 \cdot 0.7 \cdot \left(q_{office} + q_{part} \right) \cdot A_{mean.trib.3}$$

Concept 4

$$V_{unf.4.bal.col} := 1.1\cdot\left[\left(g_{floor} + g_{ins}\right)\cdot A_{mean.trib.4} + G_{beam.4.b} + G_{col}\right] \ldots = 191.374\cdot kN$$
$$+ 1.5\cdot 0.7\cdot\left(q_{office} + q_{part}\right)\cdot A_{mean.trib.4}$$

$$V_{unf.4.office.col} := 1.1\cdot\left[\left(g_{floor} + g_{ins}\right)\cdot A_{mean.trib.4} + G_{beam.4.o} + G_{col}\right] \ldots = 186.335\cdot kN$$
$$+ 1.5\cdot 0.7\cdot\left(q_{office} + q_{part}\right)\cdot A_{mean.trib.4}$$

Horizontal loads due to unintended inclination

Columns concept 3

$$H_{unf.3.bal.col} := \frac{V_{unf.3.bal.col}\cdot n_{col.bal}\cdot \alpha_{md}}{l_{beam}} = 0.073\cdot\frac{kN}{m}$$

$$H_{unf.3.office.col} := \frac{V_{unf.3.office.col}\cdot n_{col.office}\cdot \alpha_{md}}{l_{beam}} = 0.276\cdot\frac{kN}{m}$$

Total horizontal load due to unintended inclination

$$H_{unf.3} := H_{unf.3.bal.col} + H_{unf.3.office.col} = 0.349\cdot\frac{kN}{m}$$

Columns concept 4

$$H_{unf.4.bal.col} := \frac{V_{unf.4.bal.col}\cdot n_{col.bal}\cdot \alpha_{md}}{l_{beam}} = 0.075\cdot\frac{kN}{m}$$

$$H_{unf.4.office.col} := \frac{V_{unf.4.office.col}\cdot n_{col.office}\cdot \alpha_{md}}{l_{beam}} = 0.292\cdot\frac{kN}{m}$$

Total horizontal load due to unintended inclination

$$H_{unf.4} := H_{unf.4.bal.col} + H_{unf.4.office.col} = 0.367\cdot\frac{kN}{m}$$

D7.5.3 Horizontal load from unintended inclination for self-weight as favourable for the Concepts

$$V_{fav.3.bal.col} := 0.9\cdot\left[\left(g_{floor} + g_{ins}\right)\cdot A_{mean.trib.3} + G_{s.beam.3} + G_{col}\right] \ldots = 55.959\cdot kN$$
$$+ 0\cdot\left(q_{office} + q_{part}\right)\cdot A_{mean.trib.3}$$

$$V_{fav.3.office.col} := 0.9\cdot\left[\left(g_{floor} + g_{ins}\right)\cdot A_{mean.trib.3} + G_{t.beam.3} + G_{col}\right] \ldots = 48.438\cdot kN$$
$$+ 0\cdot\left(q_{office} + q_{part}\right)\cdot A_{mean.trib.3}$$

$$V_{fav.4.bal.col} := 0.9 \cdot \left[\left(g_{floor} + g_{ins} \right) \cdot A_{mean.trib.4} + G_{beam.4.b} + G_{col} \right] \ldots = 57.25 \cdot kN$$
$$+ 0 \cdot \left(q_{office} + q_{part} \right) \cdot A_{mean.trib.4}$$

$$V_{fav.4.office.col} := 0.9 \cdot \left[\left(g_{floor} + g_{ins} \right) \cdot A_{mean.trib.4} + G_{beam.4.o} + G_{col} \right] \ldots = 53.128 \cdot kN$$
$$+ 0 \cdot \left(q_{office} + q_{part} \right) \cdot A_{mean.trib.4}$$

Horizontal loads due to unintended inclination

Columns concept 3

$$H_{fav.3.bal.col} := \frac{V_{fav.3.bal.col} \cdot n_{col.bal} \cdot \alpha_{md}}{l_{beam}} = 0.022 \cdot \frac{kN}{m}$$

$$H_{fav.3.office.col} := \frac{V_{fav.3.office.col} \cdot n_{col.office} \cdot \alpha_{md}}{l_{beam}} = 0.076 \cdot \frac{kN}{m}$$

Total horizontal load due to unintended inclination

$$H_{fav.3} := H_{fav.3.bal.col} + H_{fav.3.office.col} = 0.098 \cdot \frac{kN}{m}$$

Columns concept 4

$$H_{fav.4.bal.col} := \frac{V_{fav.4.bal.col} \cdot n_{col.bal} \cdot \alpha_{md}}{l_{beam}} = 0.022 \cdot \frac{kN}{m}$$

$$H_{fav.4.office.col} := \frac{V_{fav.4.office.col} \cdot n_{col.office} \cdot \alpha_{md}}{l_{beam}} = 0.083 \cdot \frac{kN}{m}$$

Total horizontal load due to unintended inclination

$$H_{fav.4} := H_{fav.4.bal.col} + H_{fav.4.office.col} = 0.106 \cdot \frac{kN}{m}$$

Appendix D8: Lateral deflection, natural frequency and lateral acceleration

In this appendix the calculations of the lateral deflections, natural frequency and the lateral acceleration for the reference building and the concepts are presented. The results are presented in Section 7.5. Since the weight of the Concept 3 and Concept 4 are apprioximately the same only Concept 4 has been evaluated.

$kNm := kN \cdot m$

D8.1 Lateral deflection

The lateral deflection of the building is calculated with the equation below. The structure is assumed to behave mainly in flexure and behave as a cantilever beam.

$$u(x_1) := \int_0^{x_1} \frac{M(x_1)}{E \cdot I(x_1)} \cdot (x_1 - x)\, dx$$

Elastic modulus of concrete (C45/55) $E_{cm} := 36GPa$

Total length of core in y-direction $l_y := 11m$

Total length of core in z-direction $l_z := 8.8m$

$t := 250mm$

$l_1 := 3.2m$

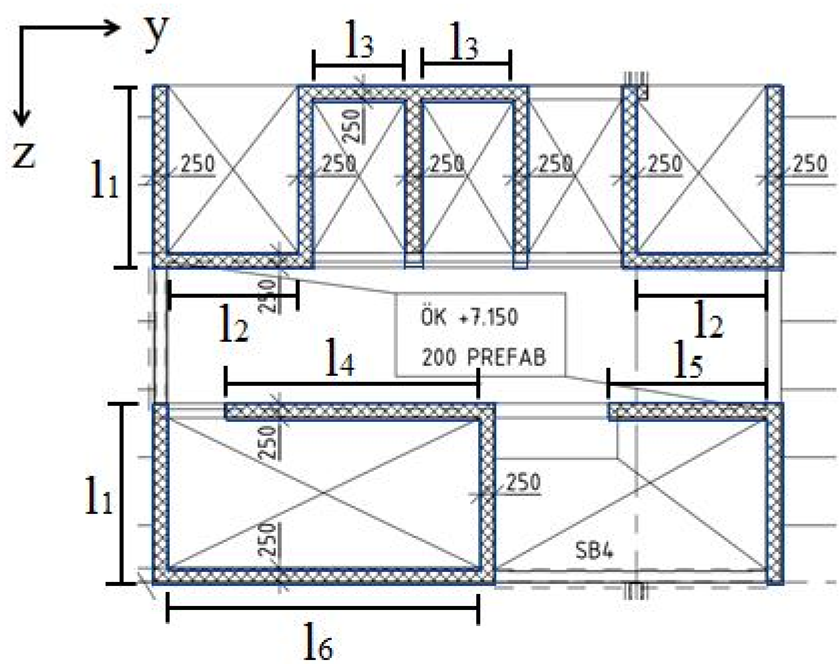
$l_2 := 2.2m$

$l_3 := 1.6m$

$l_4 := 4.4m$

$l_5 := 2.8m$

$l_6 := 5.4m$



Area of concrete walls

$A_1 := t \cdot l_1 = 0.8\,m^2$ $\quad A_3 := t \cdot l_3 = 0.4\,m^2$ $\quad A_5 := t \cdot l_5 = 0.7\,m^2$

$A_2 := t \cdot l_2 = 0.55\,m^2$ $\quad A_4 := t \cdot l_4 = 1.1\,m^2$ $\quad A_6 := t \cdot l_6 = 1.35\,m^2$

D8.1.1 Second moment of inertia around y (wind from north)

Centre of gravity for the different parts in the concrete core

$$z_1 := \frac{l_1}{2} = 1.6\,m$$

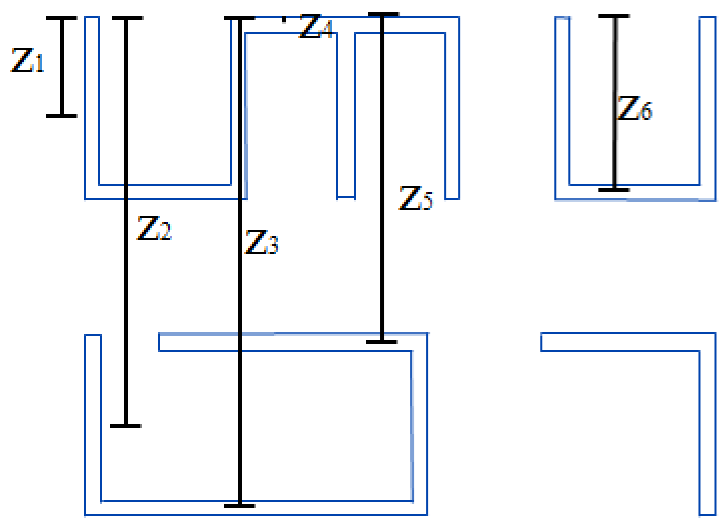
$$z_2 := l_z - \frac{l_1}{2} = 7.2\,m$$

$$z_3 := l_z - \frac{t}{2} = 8.675\,m$$

$$z_4 := \frac{t}{2} = 0.125\,m$$

$$z_5 := l_z - l_1 + \frac{t}{2} = 5.725\,m$$

$$z_6 := l_1 - \frac{t}{2} = 3.075\,m$$



Centre of gravity of the concrete core

$$z := \frac{6 \cdot A_1 \cdot z_1 + 2 \cdot A_2 \cdot z_6 + 2 \cdot A_3 \cdot z_4 + A_4 \cdot z_5 + A_5 \cdot z_5 + 3 \cdot A_1 \cdot z_2 + A_6 \cdot z_3}{9 \cdot A_1 + 2 \cdot A_2 + 2 \cdot A_3 + A_4 + A_5 + A_6} = 4.119\,m$$

Second moment of inertia for the different parts in the concrete core

$$I_{y.1} := \frac{t \cdot l_1^3}{12} = 0.683 \cdot m^4 \qquad I_{y.4} := \frac{l_4 \cdot t^3}{12} = 5.729 \times 10^{-3}\,m^4$$

$$I_{y.2} := \frac{l_2 \cdot t^3}{12} = 2.865 \times 10^{-3}\,m^4 \qquad I_{y.5} := \frac{l_5 \cdot t^3}{12} = 3.646 \times 10^{-3}\,m^4$$

$$I_{y.3} := \frac{l_3 \cdot t^3}{12} = 2.083 \times 10^{-3}\,m^4 \qquad I_{y.6} := \frac{l_6 \cdot t^3}{12} = 7.031 \times 10^{-3}\,m^4$$

Second moment of inertia of the concrete core

$$\begin{aligned} I_y := {} & 6 \cdot \left[I_{y.1} + A_1 \cdot (z - z_1)^2\right] + 3 \cdot \left[I_{y.1} + A_1 \cdot (z - z_2)^2\right] + 2 \cdot \left[I_{y.2} + A_2 \cdot (z - z_6)^2\right] \ldots = 106.035\,m^4 \\ & + 2 \cdot \left[I_{y.3} + A_3 \cdot (z - z_4)^2\right] + \left[I_{y.4} + A_4 \cdot (z - z_5)^2\right] + \left[I_{y.5} + A_5 \cdot (z - z_5)^2\right] \ldots \\ & + \left[I_{y.6} + A_6 \cdot (z - z_3)^2\right] \end{aligned}$$

Stiffness of the concrete core (y-direction)

$EI_y := E_{cm} \cdot I_y = 3.817 \times 10^{12} \cdot N \cdot m^2$

D8.1.2 Second moment of inertia around z (wind from east)

Centre of gravity for the different parts in the concrete core

$y_1 := l_y - \frac{t}{2} = 10.875\,m$

$y_2 := l_y - 1.5 \cdot t - l_2 = 8.425\,m$

$y_3 := 2t + l_2 + \frac{l_3}{2} = 3.5\,m$

$y_4 := 3 \cdot t + l_2 + 1.5 \cdot l_3 = 5.35\,m$

$y_5 := 3.5 \cdot t + l_2 + 2 \cdot l_3 = 6.275\,m$

$y_6 := 2.5 \cdot t + l_2 + l_3 = 4.425\,m$

$y_7 := 1.5 \cdot t + l_2 = 2.575\,m$

$y_8 := t + \frac{l_2}{2} = 1.35\,m$

$y_9 := l_y - t - \frac{l_2}{2} = 9.65\,m$

$y_{10} := l_y - \frac{t}{2} = 10.875\,m$

$y_{11} := l_y - t - \frac{l_5}{2} = 9.35\,m$

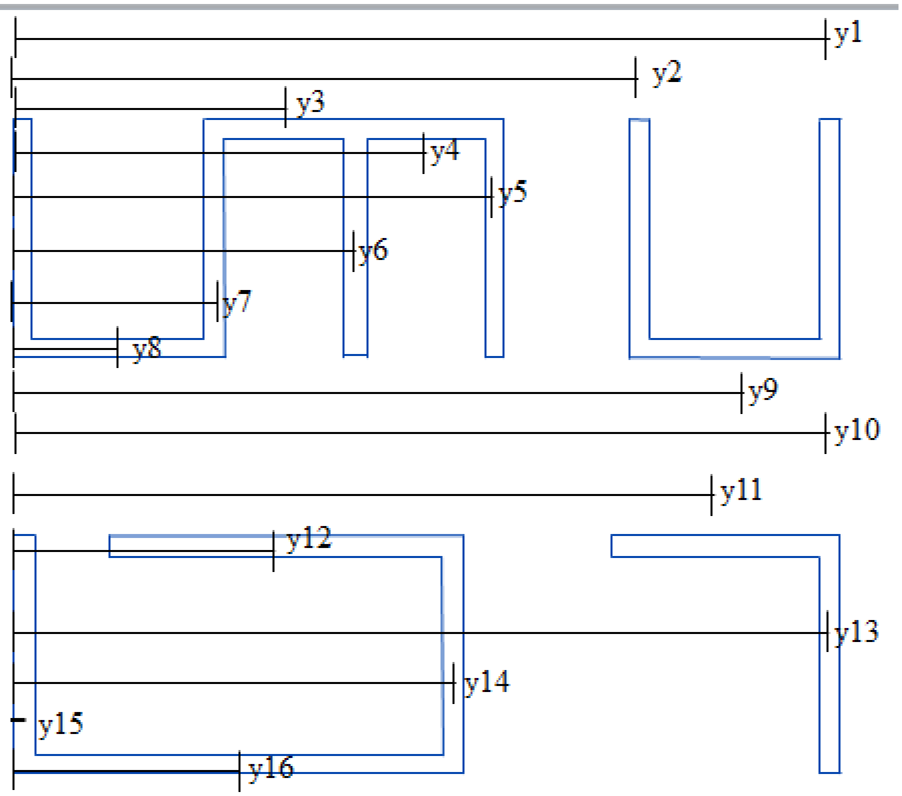
$y_{12} := t + 1m + \frac{l_4}{2} = 3.45\,m$

$y_{13} := l_y - \frac{t}{2} = 10.875\,m$

$y_{14} := 1.5t + l_6 = 5.775\,m$

$y_{15} := \frac{t}{2} = 0.125\,m$

$y_{16} := t + \frac{l_6}{2} = 2.95\,m$



Centre of gravity of the concrete core

$$y := \frac{A_1 \cdot (2y_{15} + y_7 + y_6 + y_5 + y_2 + 2 \cdot y_1 + y_{14}) + A_2 \cdot (y_8 + y_9) + A_3 \cdot (y_3 + y_4) + A_4 \cdot y_{12} \dots + A_5 \cdot y_{11} + A_6 \cdot y_{16}}{9 \cdot A_1 + 2 \cdot A_2 + 2 \cdot A_3 + A_4 + A_5 + A_6} = 5.183\,\text{n}$$

Second moment of inertia for the different parts in the concrete core

$$I_{z.1} := \frac{l_1 \cdot t^3}{12} = 4.167 \times 10^{-3}\,m^4 \qquad I_{z.4} := \frac{t \cdot l_4^{\,3}}{12} = 1.775\,m^4$$

$$I_{z.2} := \frac{t \cdot l_2^{\,3}}{12} = 0.222\,m^4 \qquad I_{z.5} := \frac{t \cdot l_5^{\,3}}{12} = 0.457\,m^4$$

$$I_{z.3} := \frac{t \cdot l_3^{\,3}}{12} = 0.085\,m^4 \qquad I_{z.6} := \frac{t \cdot l_6^{\,3}}{12} = 3.281\,m^4$$

Second moment of inertia of the concrete core

$$I_z := \left[I_{z.1} \cdot 9 + A_1 \cdot \left(2 \cdot y_{15}^{\,2} + y_7^{\,2} + y_6^{\,2} + y_5^{\,2} + y_2^{\,2} + 2 \cdot y_1^{\,2} + y_{14}^{\,2}\right)\right] \dots = 485.954\,m^4$$
$$+ \left[I_{z.2} \cdot 2 + A_2 \cdot \left(y_8^{\,2} + y_9^{\,2}\right)\right] + \left[I_{z.3} \cdot 2 + A_3 \cdot \left(y_3^{\,2} + y_4^{\,2}\right)\right] \dots$$
$$+ \left(I_{z.4} + A_4 \cdot y_{12}^{\,2}\right) + \left(I_{z.5} + A_5 \cdot y_{11}^{\,2}\right) + \left(I_{z.6} + A_6 \cdot y_{16}^{\,2}\right)$$

Stiffness of the concrete core (z-direction)

$$EI_z := E_{cm} \cdot I_z = 1.749 \times 10^{13} \cdot N \cdot m^2$$

D8.1.3 Applied moment on each floor

Table below shows different horisontal loads from unintended inclinations. These values were caluclated in Appendix D6 and D7.

Unintended inclination	When self-weight is unfavourable	When self-weight is favourable
Wind from east, reference building	1.536 kN/m	0.876 kN/m
Wind from east, Concept 3	0.493 kN/m	0.157 kN/m
Wind from east, Concept 4	0.510 kN/m	0.164 kN/m
Wind from north, reference building	0.712 kN/m	0.379 kN/m
Wind from north, Concept 3	0.349 kN/m	0.098 kN/m
Wind from north, Concept 4	0.367 kN/m	0.106 kN/m

D8.1.3.1 Applied moment for concepts when wind from north

When considering wind from north a simplification made was to neglect the fact that the two storeys in the top of the building is smaller due to the balcony. This simplification is on the safe side becuase the building will be subjected to a larger wind load.

Wind pressure from north (values taken from Appendix C1)

$w_{1.n} := 1505\text{Pa}$ (Zone 1, the upper zone)

$w_{2.n} := 1317\text{Pa}$ (Zone 2, the lower zone)

Lenght of north facade

$l_n := 33.06\text{m}$

Characterisitic equivalent force from unintended inclination (value taken from table above)

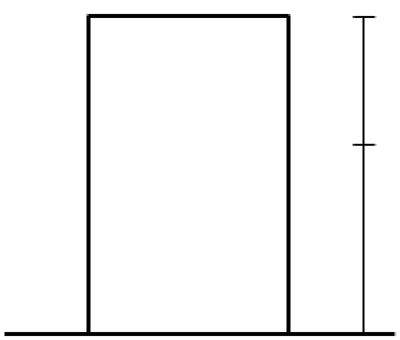
$$f_{un.k} := 0.367\frac{\text{kN}}{\text{m}}\cdot\frac{l_n}{3.6\text{m}} = 3.37\cdot\frac{\text{kN}}{\text{m}}$$

Wind expressed as a distributed load along the core (wind load as main load and therefore increased by a facotr of 1.5)

$$H_{1.n} := 1.5 l_n \cdot w_{1.n} + f_{un.k} = 78.003\cdot\frac{\text{kN}}{\text{m}}$$

$$H_{2.n} := 1.5 l_n \cdot w_{2.n} + f_{un.k} = 68.68\cdot\frac{\text{kN}}{\text{m}}$$

Height of zone 1 and zone 2



$zone_1 := 52.65\text{m} - 37.285\text{m} = 15.365\,\text{m}$

$zone_2 := 37.285\text{m}$

Support moment

$$M_{a.n} := H_{1.n}\cdot zone_1\cdot\left(zone_2 + \frac{zone_1}{2}\right) + H_{2.n}\cdot\frac{zone_2^{\,2}}{2} = 1.016\times 10^5\cdot\text{kNm}$$

Reaction force

$$V_{a.n} := H_{1.n}\cdot zone_1 + H_{2.n}\cdot zone_2 = 3.759\times 10^3\cdot\text{kN}$$

Moment distribution for zone 2

$x2 := 0\text{m}, 0.01\text{m}..37.29\text{m}$

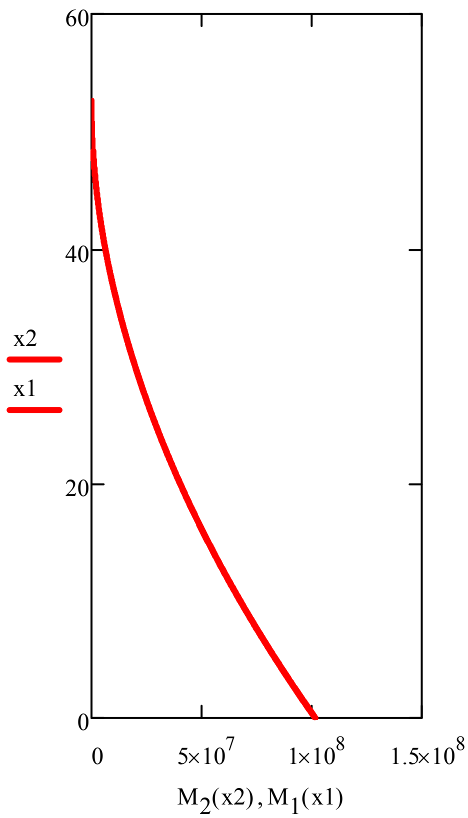
$$M_2(x2) := M_{a.n} + H_{2.n}\cdot\frac{x2^2}{2} - V_{a.n}\cdot x2$$

Moment distribution for zone 1

$$x1 := 37.29m, 37.30m..52.65m$$

$$M_1(x1) := M_{a.n} + H_{2.n}\cdot zone_2\cdot\left(x1 - \frac{zone_2}{2}\right) + H_{1.n}\cdot\frac{(x1 - zone_2)^2}{2} - V_{a.n}\cdot x1$$

Moment distribution when the concepts are subjected to wind from north



$$M_n := \begin{pmatrix} M_1(52.65m) \\ M_1(47.9m) \\ M_1(44.3m) \\ M_1(40.3m) \\ M_2(36.7m) \\ M_2(33.1m) \\ M_2(29.5m) \\ M_2(25.9m) \\ M_2(22.3m) \\ M_2(18.7m) \\ M_2(15.1m) \\ M_2(11.5m) \\ M_2(7.9m) \\ M_2(4.3m) \\ M_2(0m) \end{pmatrix} = \cdot kNm$$

	0
0	$2.98\cdot10^{-11}$
1	879.974
2	$2.719\cdot10^{3}$
3	$5.949\cdot10^{3}$
4	$9.921\cdot10^{3}$
5	$1.482\cdot10^{4}$
6	$2.062\cdot10^{4}$
7	$2.73\cdot10^{4}$
8	$3.488\cdot10^{4}$
9	$4.334\cdot10^{4}$
10	$5.27\cdot10^{4}$
11	$6.294\cdot10^{4}$
12	$7.408\cdot10^{4}$
13	$8.61\cdot10^{4}$
14	$1.016\cdot10^{5}$

D8.1.3.2 Applied moment for reference building when wind from north

Characterisitic equivalent force from unintended inclination (value taken from table above)

$$f_{un.k.ref} := 0.712\frac{kN}{m}\cdot\frac{l_n}{3.6m} = 6.539\cdot\frac{kN}{m}$$

Wind expressed as a distributed load along the core (wind load as main load and therefore increased by a facotr of 1.5)

$$H_{1.n.ref} := 1.5 l_n\cdot w_{1.n} + f_{un.k.ref} = 81.171\cdot\frac{kN}{m}$$

$$H_{2.n.ref} := 1.5 l_n\cdot w_{2.n} + f_{un.k.ref} = 71.849\cdot\frac{kN}{m}$$

Support moment $M_{a.n.ref} := H_{1.n.ref} \cdot zone_1 \cdot \left(zone_2 + \frac{zone_1}{2} \right) + H_{2.n.ref} \cdot \frac{{zone_2}^2}{2} = 1.06 \times 10^5 \cdot kNm$

Reaction force $V_{a.n.ref} := H_{1.n.ref} \cdot zone_1 + H_{2.n.ref} \cdot zone_2 = 3.926 \times 10^3 \cdot kN$

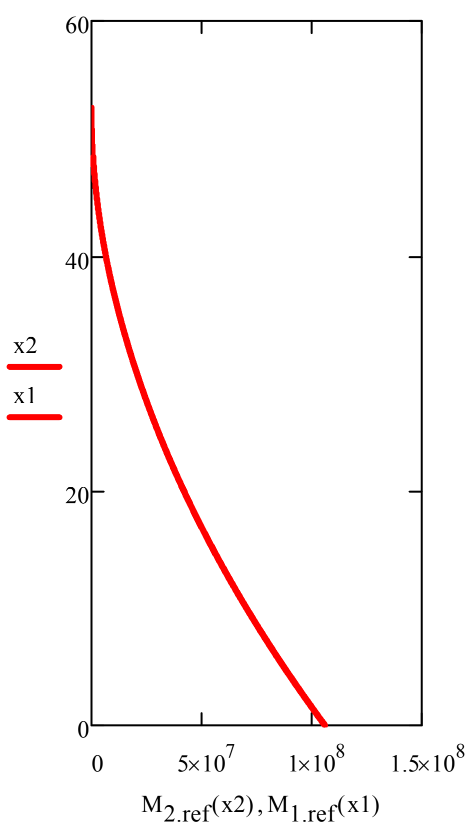
Moment distribution for zone 2

$$M_{2.ref}(x2) := M_{a.n.ref} + H_{2.n.ref} \cdot \frac{x2^2}{2} - V_{a.n.ref} \cdot x2$$

Moment distribution for zone 1

$$M_{1.ref}(x1) := M_{a.n.ref} + H_{2.n.ref} \cdot zone_2 \cdot \left(x1 - \frac{zone_2}{2} \right) + H_{1.n.ref} \cdot \frac{(x1 - zone_2)^2}{2} - V_{a.n.ref} \cdot x1$$

Moment distribution when the reference building is subjected to wind from north



$$M_{n.ref} := \begin{pmatrix} M_{1.ref}(52.65m) \\ M_{1.ref}(47.9m) \\ M_{1.ref}(44.3m) \\ M_{1.ref}(40.3m) \\ M_{2.ref}(36.7m) \\ M_{2.ref}(33.1m) \\ M_{2.ref}(29.5m) \\ M_{2.ref}(25.9m) \\ M_{2.ref}(22.3m) \\ M_{2.ref}(18.7m) \\ M_{2.ref}(15.1m) \\ M_{2.ref}(11.5m) \\ M_{2.ref}(7.9m) \\ M_{2.ref}(4.3m) \\ M_{2.ref}(0m) \end{pmatrix} = \cdot kNm$$

	0
0	0
1	915.716
2	$2.83 \cdot 10^3$
3	$6.19 \cdot 10^3$
4	$1.032 \cdot 10^4$
5	$1.543 \cdot 10^4$
6	$2.147 \cdot 10^4$
7	$2.844 \cdot 10^4$
8	$3.634 \cdot 10^4$
9	$4.517 \cdot 10^4$
10	$5.493 \cdot 10^4$
11	$6.563 \cdot 10^4$
12	$7.725 \cdot 10^4$
13	$8.981 \cdot 10^4$
14	$1.06 \cdot 10^5$

D8.1.3.3 Applied moment for concepts when wind from east

Wind pressure from north (values taken from Appendix C1)

$w_{1.e} := 1515Pa$ (Zone 1, the upper zone)

$w_{2.e} := 1374Pa$ (Zone 2, the lower zone)

Lenght of east facade

$l_e := 37.285m$

Characterisitic equivalent force from unintended inclination (value taken from table above)

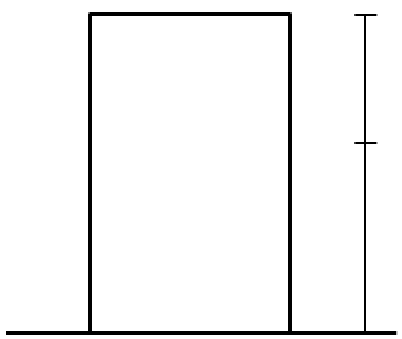
$$f_{ue.k} := 0.510\frac{kN}{m}\cdot\frac{l_e}{3.6m} = 5.282\cdot\frac{kN}{m}$$

Wind expressed as a distributed load along the core (wind load as main load and therefore increased by a facotr of 1.5)

$$H_{1.e} := 1.5l_e\cdot w_{1.e} + f_{ue.k} = 90.012\cdot\frac{kN}{m}$$

$$H_{2.e} := 1.5l_e\cdot w_{2.e} + f_{ue.k} = 82.126\cdot\frac{kN}{m}$$

Height of zone 1 and zone 2



$zone_{1.e} := 52.65m - 33.06m = 19.59\,m$

$zone_{2.e} := 33.06m$

Support moment

$$M_{a.e} := H_{1.e}\cdot zone_{1.e}\cdot\left(zone_{2.e} + \frac{zone_{1.e}}{2}\right) + H_{2.e}\cdot\frac{zone_{2.e}^{\,2}}{2} = 1.204\times10^{5}\cdot kNm$$

Reaction force

$$V_{a.e} := H_{1.e}\cdot zone_{1.e} + H_{2.e}\cdot zone_{2.e} = 4.478\times10^{3}\cdot kN$$

Moment distribution for zone 2

$x2_e := 0m, 0.01m..33.06m$

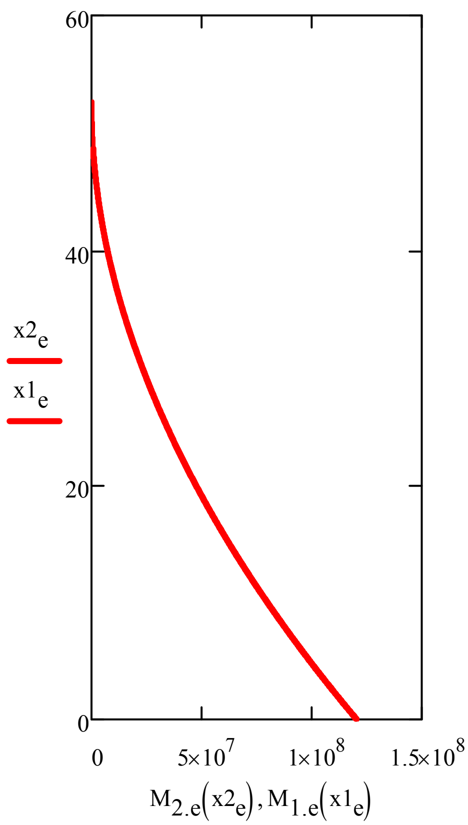
$$M_{2.e}(x2_e) := M_{a.e} + H_{2.e}\cdot\frac{x2_e^{\,2}}{2} - V_{a.e}\cdot x2_e$$

Moment distribution for zone 1

$x1_e := 33.06m, 33.07m..52.65m$

$$M_{1.e}(x1_e) := M_{a.e} + H_{2.e}\cdot zone_{2.e}\cdot\left(x1_e - \frac{zone_{2.e}}{2}\right) + H_{1.e}\cdot\frac{\left(x1_e - zone_{2.e}\right)^2}{2} - V_{a.e}\cdot x1_e$$

Moment distribution when the concepts are subjected to wind from east



$$M_e := \begin{pmatrix} M_{1.e}(52.65m) \\ M_{1.e}(47.9m) \\ M_{1.e}(44.3m) \\ M_{1.e}(40.3m) \\ M_{1.e}(36.7m) \\ M_{1.e}(33.1m) \\ M_{2.e}(29.5m) \\ M_{2.e}(25.9m) \\ M_{2.e}(22.3m) \\ M_{2.e}(18.7m) \\ M_{2.e}(15.1m) \\ M_{2.e}(11.5m) \\ M_{2.e}(7.9m) \\ M_{2.e}(4.3m) \\ M_{2.e}(0m) \end{pmatrix} = \cdot kNm$$

	0
0	$2.98 \cdot 10^{-11}$
1	$1.015 \cdot 10^{3}$
2	$3.138 \cdot 10^{3}$
3	$6.864 \cdot 10^{3}$
4	$1.145 \cdot 10^{4}$
5	$1.72 \cdot 10^{4}$
6	$2.407 \cdot 10^{4}$
7	$3.2 \cdot 10^{4}$
8	$4.1 \cdot 10^{4}$
9	$5.106 \cdot 10^{4}$
10	$6.219 \cdot 10^{4}$
11	$7.438 \cdot 10^{4}$
12	$8.763 \cdot 10^{4}$
13	$1.02 \cdot 10^{5}$
14	$1.204 \cdot 10^{5}$

D8.1.3.4 Applied moment for reference building when wind from east

Characterisitic equivalent force from unintended inclination (value taken from table above)

$$f_{ue.k.ref} := 1.536 \frac{kN}{m} \cdot \frac{l_e}{3.6m} = 15.908 \cdot \frac{kN}{m}$$

Wind expressed as a distributed load along the core

$$H_{1.e.ref} := 1.5 l_e \cdot w_{1.e} + f_{ue.k.ref} = 100.638 \cdot \frac{kN}{m}$$

$$H_{2.e.ref} := 1.5 l_e \cdot w_{2.e} + f_{ue.k.ref} = 92.753 \cdot \frac{kN}{m}$$

Support moment

$$M_{a.e.ref} := H_{1.e.ref} \cdot zone_{1.e} \cdot \left(zone_{2.e} + \frac{zone_{1.e}}{2} \right) + H_{2.e.ref} \cdot \frac{zone_{2.e}^{\;2}}{2} = 1.352 \times 10^{5} \cdot kNm$$

Reaction force

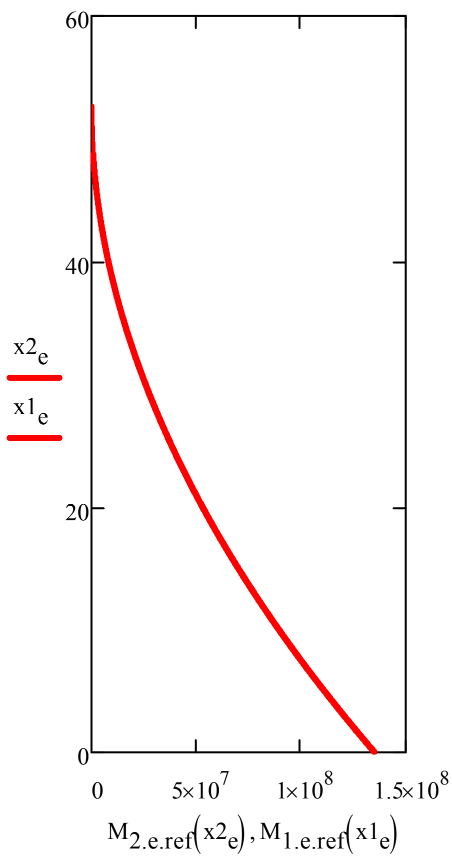
$$V_{a.e.ref} := H_{1.e.ref} \cdot zone_{1.e} + H_{2.e.ref} \cdot zone_{2.e} = 5.038 \times 10^{3} \cdot kN$$

Moment distribution for the lower part

$$M_{2.e.ref}(x2_e) := M_{a.e.ref} + H_{2.e.ref} \cdot \frac{x2_e^{\,2}}{2} - V_{a.e.ref} \cdot x2_e$$

$$M_{1.e.ref}(x1_e) := M_{a.e.ref} + H_{2.e.ref} \cdot zone_{2.e} \cdot \left(x1_e - \frac{zone_{2.e}}{2} \right) + H_{1.e.ref} \cdot \frac{(x1_e - zone_{2.e})^2}{2} - V_{a.e.ref} \cdot x1_e$$

Moment distribution when the reference building is subjected to wind from east



$$M_{e.ref} := \begin{pmatrix} M_{1.e.ref}(52.65m) \\ M_{1.e.ref}(47.9m) \\ M_{1.e.ref}(44.3m) \\ M_{1.e.ref}(40.3m) \\ M_{1.e.ref}(36.7m) \\ M_{1.e.ref}(33.1m) \\ M_{2.e.ref}(29.5m) \\ M_{2.e.ref}(25.9m) \\ M_{2.e.ref}(22.3m) \\ M_{2.e.ref}(18.7m) \\ M_{2.e.ref}(15.1m) \\ M_{2.e.ref}(11.5m) \\ M_{2.e.ref}(7.9m) \\ M_{2.e.ref}(4.3m) \\ M_{2.e.ref}(0m) \end{pmatrix} = \cdot kNm$$

	0
0	$2.98 \cdot 10^{-11}$
1	$1.135 \cdot 10^{3}$
2	$3.508 \cdot 10^{3}$
3	$7.675 \cdot 10^{3}$
4	$1.28 \cdot 10^{4}$
5	$1.923 \cdot 10^{4}$
6	$2.692 \cdot 10^{4}$
7	$3.58 \cdot 10^{4}$
8	$4.589 \cdot 10^{4}$
9	$5.718 \cdot 10^{4}$
10	$6.968 \cdot 10^{4}$
11	$8.337 \cdot 10^{4}$
12	$9.827 \cdot 10^{4}$
13	$1.144 \cdot 10^{5}$
14	$1.352 \cdot 10^{5}$

D8.1.4 Lateral deflection

Below the lateral deflections of each floor are calculated according to the equation that was described earlier.

Concepts (wind from north)

$$u_n := \begin{bmatrix} \int_{0m}^{4.3m} \frac{M_{n_1}}{EI_y}\cdot(4.3m - x)\,dx \\ \int_{0m}^{7.9m} \frac{M_{n_2}}{EI_y}\cdot(7.9m - x)\,dx \\ \int_{0m}^{11.5m} \frac{M_{n_3}}{EI_y}\cdot(11.5m - x)\,dx \\ \int_{0m}^{15.1m} \frac{M_{n_4}}{EI_y}\cdot(15.1m - x)\,dx \\ \int_{0m}^{18.7m} \frac{M_{n_5}}{EI_y}\cdot(18.7m - x)\,dx \\ \int_{0m}^{22.3m} \frac{M_{n_6}}{EI_y}\cdot(22.3m - x)\,dx \\ \int_{0m}^{25.9m} \frac{M_{n_7}}{EI_y}\cdot(25.9m - x)\,dx \\ \int_{0m}^{29.5m} \frac{M_{n_8}}{EI_y}\cdot(29.5m - x)\,dx \\ \int_{0m}^{33.1m} \frac{M_{n_9}}{EI_y}\cdot(33.1m - x)\,dx \end{bmatrix}$$

Concepts (wind from east)

$$u_e := \begin{bmatrix} \int_{0m}^{4.3m} \frac{M_{e_1}}{EI_z}\cdot(4.3m - x)\,dx \\ \int_{0m}^{7.9m} \frac{M_{e_2}}{EI_z}\cdot(7.9m - x)\,dx \\ \int_{0m}^{11.5m} \frac{M_{e_3}}{EI_z}\cdot(11.5m - x)\,dx \\ \int_{0m}^{15.1m} \frac{M_{e_4}}{EI_z}\cdot(15.1m - x)\,dx \\ \int_{0m}^{18.7m} \frac{M_{e_5}}{EI_z}\cdot(18.7m - x)\,dx \\ \int_{0m}^{22.3m} \frac{M_{e_6}}{EI_z}\cdot(22.3m - x)\,dx \\ \int_{0m}^{25.9m} \frac{M_{e_7}}{EI_z}\cdot(25.9m - x)\,dx \\ \int_{0m}^{29.5m} \frac{M_{e_8}}{EI_z}\cdot(29.5m - x)\,dx \\ \int_{0m}^{33.1m} \frac{M_{e_9}}{EI_z}\cdot(33.1m - x)\,dx \end{bmatrix}$$

$$\begin{bmatrix} \vdots \\ \displaystyle\int_{0m}^{36.7m} \frac{M_{n_{10}}}{EI_y}\cdot(36.7m - x)\,dx \\ \displaystyle\int_{0m}^{40.3m} \frac{M_{n_{11}}}{EI_y}\cdot(40.3m - x)\,dx \\ \displaystyle\int_{0m}^{44.3m} \frac{M_{n_{12}}}{EI_y}\cdot(44.3m - x)\,dx \\ \displaystyle\int_{0m}^{47.9m} \frac{M_{n_{13}}}{EI_y}\cdot(47.9m - x)\,dx \\ \displaystyle\int_{0m}^{52.65m} \frac{M_{n_{14}}}{EI_y}\cdot(52.65m - x)\,dx \end{bmatrix} \qquad \begin{bmatrix} \vdots \\ \displaystyle\int_{0m}^{36.7m} \frac{M_{e_{10}}}{EI_z}\cdot(36.7m - x)\,dx \\ \displaystyle\int_{0m}^{40.3m} \frac{M_{e_{11}}}{EI_z}\cdot(40.3m - x)\,dx \\ \displaystyle\int_{0m}^{44.3m} \frac{M_{e_{12}}}{EI_z}\cdot(44.3m - x)\,dx \\ \displaystyle\int_{0m}^{47.9m} \frac{M_{e_{13}}}{EI_z}\cdot(47.9m - x)\,dx \\ \displaystyle\int_{0m}^{52.65m} \frac{M_{e_{14}}}{EI_z}\cdot(52.65m - x)\,dx \end{bmatrix}$$

Reference building (wind from north)

$$\begin{bmatrix} \displaystyle\int_{0m}^{4.3m} \frac{M_{n.ref_1}}{EI_y}\cdot(4.3m - x)\,dx \\ \displaystyle\int_{0m}^{7.9m} \frac{M_{n.ref_2}}{EI_y}\cdot(7.9m - x)\,dx \\ \displaystyle\int_{0m}^{11.5m} \frac{M_{n.ref_3}}{EI_y}\cdot(11.5m - x)\,dx \\ \displaystyle\int_{0m}^{15.1m} \frac{M_{n.ref_4}}{EI_y}\cdot(15.1m - x)\,dx \\ \vdots \end{bmatrix}$$

Reference building (wind from east)

$$\begin{bmatrix} \displaystyle\int_{0m}^{4.3m} \frac{M_{e.ref_1}}{EI_z}\cdot(4.3m - x)\,dx \\ \displaystyle\int_{0m}^{7.9m} \frac{M_{e.ref_2}}{EI_z}\cdot(7.9m - x)\,dx \\ \displaystyle\int_{0m}^{11.5m} \frac{M_{e.ref_3}}{EI_z}\cdot(11.5m - x)\,dx \\ \displaystyle\int_{0m}^{15.1m} \frac{M_{e.ref_4}}{EI_z}\cdot(15.1m - x)\,dx \\ \vdots \end{bmatrix}$$

$$
u_{n.ref} := \begin{pmatrix}
\displaystyle\int_{0m}^{18.7m} \frac{M_{n.ref_5}}{EI_y}\cdot(18.7m - x)\,dx \\
\displaystyle\int_{0m}^{22.3m} \frac{M_{n.ref_6}}{EI_y}\cdot(22.3m - x)\,dx \\
\displaystyle\int_{0m}^{25.9m} \frac{M_{n.ref_7}}{EI_y}\cdot(25.9m - x)\,dx \\
\displaystyle\int_{0m}^{29.5m} \frac{M_{n.ref_8}}{EI_y}\cdot(29.5m - x)\,dx \\
\displaystyle\int_{0m}^{33.1m} \frac{M_{n.ref_9}}{EI_y}\cdot(33.1m - x)\,dx \\
\displaystyle\int_{0m}^{36.7m} \frac{M_{n.ref_{10}}}{EI_y}\cdot(36.7m - x)\,dx \\
\displaystyle\int_{0m}^{40.3m} \frac{M_{n.ref_{11}}}{EI_y}\cdot(40.3m - x)\,dx \\
\displaystyle\int_{0m}^{44.3m} \frac{M_{n.ref_{12}}}{EI_y}\cdot(44.3m - x)\,dx \\
\displaystyle\int_{0m}^{47.9m} \frac{M_{n.ref_{13}}}{EI_y}\cdot(47.9m - x)\,dx
\end{pmatrix}
\qquad
u_{e.ref} := \begin{pmatrix}
\displaystyle\int_{0m}^{18.7m} \frac{M_{e.ref_5}}{EI_z}\cdot(18.7m - x)\,dx \\
\displaystyle\int_{0m}^{22.3m} \frac{M_{e.ref_6}}{EI_z}\cdot(22.3m - x)\,dx \\
\displaystyle\int_{0m}^{25.9m} \frac{M_{e.ref_7}}{EI_z}\cdot(25.9m - x)\,dx \\
\displaystyle\int_{0m}^{29.5m} \frac{M_{e.ref_8}}{EI_z}\cdot(29.5m - x)\,dx \\
\displaystyle\int_{0m}^{33.1m} \frac{M_{e.ref_9}}{EI_z}\cdot(33.1m - x)\,dx \\
\displaystyle\int_{0m}^{36.7m} \frac{M_{e.ref_{10}}}{EI_z}\cdot(36.7m - x)\,dx \\
\displaystyle\int_{0m}^{40.3m} \frac{M_{e.ref_{11}}}{EI_z}\cdot(40.3m - x)\,dx \\
\displaystyle\int_{0m}^{44.3m} \frac{M_{e.ref_{12}}}{EI_z}\cdot(44.3m - x)\,dx \\
\displaystyle\int_{0m}^{47.9m} \frac{M_{e.ref_{13}}}{EI_z}\cdot(47.9m - x)\,dx
\end{pmatrix}
$$

$$\left[\int_{0m}^{52.65m} \frac{M_{n.ref_{14}}}{EI_y}\cdot(52.65m - x)\,dx\right] \qquad \left[\int_{0m}^{52.65m} \frac{M_{e.ref_{14}}}{EI_z}\cdot(52.65m - x)\,dx\right]$$

Lateral deflection for the Concepts

$u_n =$ (·mm)

	0
0	$2.131\cdot10^{-3}$
1	0.022
2	0.103
3	0.296
4	0.679
5	1.343
6	2.399
7	3.976
8	6.22
9	9.297
10	13.39
11	19.042
12	25.877
13	36.902

$u_e =$ (·mm)

	0
0	$5.366\cdot10^{-4}$
1	$5.597\cdot10^{-3}$
2	0.026
3	0.075
4	0.172
5	0.342
6	0.614
7	1.02
8	1.599
9	2.394
10	3.452
11	4.915
12	6.685
13	9.543

Lateral deflection for the Reference building

$u_{n.ref} =$ (·mm)

	0
0	$2.218\cdot10^{-3}$
1	0.023
2	0.107
3	0.308
4	0.707
5	1.398
6	2.499
7	4.142
8	6.482
9	9.691
10	13.96
11	19.857
12	26.99
13	38.496

$u_{e.ref} =$ (·mm)

	0
0	$6\cdot10^{-4}$
1	$6.258\cdot10^{-3}$
2	0.029
3	0.083
4	0.192
5	0.383
6	0.686
7	1.141
8	1.791
9	2.682
10	3.87
11	5.512
12	7.5
13	10.71

D8.2 Natural frequency

An approximation of the natural frequency of a building can be calculated with the equation below.

$$f_n := \frac{1}{2\cdot\pi}\cdot\sqrt{g\cdot\left[\frac{\sum_i \left(F_i\cdot u_i\right)}{\left[\sum_i \left(W_i\cdot u_i\right)^2\right]}\right]}$$

D8.2.1 Equivalent lateral load at each floor

Values for the wind load are taken from Appendix C1. The wind load is unfavourable, hence it is multiplied with 1.5 in the load combination.

$$F_{wind.n} := 1.5\begin{pmatrix} 3.574 \\ 6.283 \\ 5.718 \\ 5.718 \\ 5.417 \\ 5.098 \\ 4.741 \\ 4.741 \\ 4.741 \\ 4.741 \\ 4.741 \\ 4.741 \\ 4.741 \\ 5.202 \end{pmatrix}\frac{kN}{m} \qquad F_{wind.e} := 1.5\begin{pmatrix} 3.598 \\ 6.325 \\ 5.757 \\ 5.757 \\ 5.454 \\ 5.2 \\ 4.946 \\ 4.946 \\ 4.946 \\ 4.946 \\ 4.946 \\ 4.946 \\ 4.946 \\ 5.427 \end{pmatrix}\frac{kN}{m}$$

Equivalent load (wind from north)

$$F_n := l_n\cdot\left(F_{wind.n}\right)$$

Equivalent load (wind from east)

$$F_e := l_e\cdot F_{wind.e}$$

D8.2.2 Weight of each floor

$ton := 1000kg$

In this section the weight per storey in the buildings are calculated. Values for the weight of different members in the buildings are taken from Appendix D3.

D8.2.2.1 Weight of the reference building

Weight of floor and roof per storey

Weight of roof $w_{roof.2} := 311.904ton$

Weight of balcony roof $w_{roof.1} := 73.486ton$

Weight of floor, part 1 $w_{floor.1.ref} := 91.405ton$

Weight of floor, part 2 $w_{floor.2.ref} := 345.42ton$

$$w_{floor.ref} := \begin{pmatrix} w_{roof.2} \\ w_{floor.1.ref} \\ w_{floor.2.ref} + w_{roof.1} \\ w_{floor.1.ref} + w_{floor.2.ref} \\ w_{floor.1.ref} + w_{floor.2.ref} \\ w_{floor.1.ref} + w_{floor.2.ref} \\ w_{floor.1.ref} + w_{floor.2.ref} \\ w_{floor.1.ref} + w_{floor.2.ref} \\ w_{floor.1.ref} + w_{floor.2.ref} \\ w_{floor.1.ref} + w_{floor.2.ref} \\ w_{floor.1.ref} + w_{floor.2.ref} \\ w_{floor.1.ref} + w_{floor.2.ref} \\ w_{floor.1.ref} + w_{floor.2.ref} \end{pmatrix} = \cdot ton$$

	0
0	311.904
1	91.405
2	418.906
3	436.825
4	436.825
5	436.825
6	436.825
7	436.825
8	436.825
9	436.825
10	436.825
11	436.825
12	436.825

Weight of walls and core per storey

Weight of walls on 14th storey $w_{wall.14.ref} := 324.609ton$

Weight of walls on 13th storey $w_{wall.13.ref} := 177.35ton$

Weight of walls on 12th storey $w_{wall.12.ref} := 242.578ton$

Weight of walls on 11th - 1st storey $w_{wall.ref.} := 210.183ton$

Weight of concrete core $w_{core} := 128.105ton$

$$w_{wall.ref} := \begin{pmatrix} w_{wall.14.ref} \\ w_{wall.13.ref} \\ w_{wall.12.ref} \\ w_{wall.ref.} \\ w_{wall.ref.} \\ w_{wall.ref.} \\ w_{wall.ref.} \\ w_{wall.ref.} \\ w_{wall.ref.} \\ w_{wall.ref.} \\ w_{wall.ref.} \\ w_{wall.ref.} \\ w_{wall.ref.} \end{pmatrix} \qquad w_{core} := \begin{pmatrix} w_{core} \\ w_{core} \\ w_{core} \\ w_{core} \\ w_{core} \\ w_{core} \\ w_{core} \\ w_{core} \\ w_{core} \\ w_{core} \\ w_{core} \\ w_{core} \\ w_{core} \end{pmatrix}$$

Weight of each floor in the reference building

$$w_{ref} := g \cdot \left(w_{floor.ref} + w_{wall.ref} + w_{core}\right) =$$

	0
0	7.498
1	3.892
2	7.743
3	7.601
4	7.601
5	7.601
6	7.601
7	7.601
8	7.601
9	7.601
10	7.601
11	7.601
12	7.601

$\cdot MN$

D8.2.2.2 Weight of Concept building

Weight of floor and roof per storey

Weight of roof $w_{roof.2} = 311.904 \cdot ton$

Weight of balcony roof $w_{roof.1} = 73.486 \cdot ton$

Weight of floor, part 1 $w_{floor.1} := 23.53ton$

Weight of floor, part 2 $w_{floor.2} := 88.92ton$

Weight of walls and core per storey

Weight of walls on 14th storey $w_{wall.14} := 46.97ton$

Weight of walls on 13th storey $w_{wall.13} := 25.662ton$

Weight of walls on 12th storey $w_{wall.12} := 35.1ton$

Weight of walls on 11th storey $w_{wall.11} := 30.413ton$

Weight of walls on 6th - 10th storey $w_{wall.6} := 39.624ton$

Weight of walls on 2nd - 5th storey $w_{wall.2} := 45.18ton$

Weight of concrete core $w_{core} := 128.105ton$

$$w_{floor} := \begin{pmatrix} w_{roof.2} \\ w_{floor.1} \\ w_{roof.1} + w_{floor.2} \\ w_{floor.1} + w_{floor.2} \\ w_{floor.1} + w_{floor.2} \\ w_{floor.1} + w_{floor.2} \\ w_{floor.1} + w_{floor.2} \\ w_{floor.1} + w_{floor.2} \\ w_{floor.1} + w_{floor.2} \\ w_{floor.1} + w_{floor.2} \\ w_{floor.1} + w_{floor.2} \\ w_{floor.1} + w_{floor.2} \\ w_{floor.1} + w_{floor.2} \end{pmatrix} \qquad w_{wall} := \begin{pmatrix} w_{wall.14} \\ w_{wall.13} \\ w_{wall.12} \\ w_{wall.11} \\ w_{wall.6} \\ w_{wall.6} \\ w_{wall.6} \\ w_{wall.6} \\ w_{wall.6} \\ w_{wall.2} \\ w_{wall.2} \\ w_{wall.2} \\ w_{wall.2} \end{pmatrix}$$

Weight of each floor in the concepts

$$w := g\cdot\left(w_{floor} + w_{wall} + w_{core}\right) = \begin{array}{|c|c|}\hline & 0 \\ \hline 0 & 4.776 \\ \hline 1 & 1.739 \\ \hline 2 & 3.193 \\ \hline 3 & 2.657 \\ \hline 4 & 2.748 \\ \hline 5 & 2.748 \\ \hline 6 & 2.748 \\ \hline 7 & 2.748 \\ \hline 8 & 2.748 \\ \hline 9 & 2.802 \\ \hline 10 & 2.802 \\ \hline 11 & 2.802 \\ \hline 12 & 2.802 \\ \hline \end{array} \cdot MN$$

D8.2.3 Natural frequency

The natural frequency is calculated according to the equation extressed above.

D8.3.2.1 Natural frequency of reference building

$i := 0..12$

Wind from north

$$Fu_{ref.n} := \sum_{i=0}^{12}\left(F_{n_i}\cdot u_{n.ref_i}\right) = 2.033\times10^{4}\,J$$

$$Wu_{ref.n} := \sum_{i=0}^{12}\left[w_{ref_i}\cdot\left(u_{n.ref_i}\right)^2\right] = 1.125\times10^{4}\,\frac{m^3\cdot kg}{s^2}$$

Natrual frequency

$$f_{ref.n} := \frac{1}{2\cdot\pi}\cdot\sqrt{\frac{g\cdot Fu_{ref.n}}{Wu_{ref.n}}} = 0.67\cdot Hz$$

Wind from east

$$Fu_{ref.e} := \sum_{i=0}^{12}\left(F_{e_i}\cdot u_{e.ref_i}\right) = 6.621\times10^{3}\,J$$

$$Wu_{ref.e} := \sum_{i=0}^{12}\left[w_{ref_i}\cdot\left(u_{e.ref_i}\right)^2\right] = 866.35\,\frac{m^3\cdot kg}{s^2}$$

Natrual frequency

$$f_{ref.e} := \frac{1}{2\cdot\pi}\cdot\sqrt{\frac{g\cdot Fu_{ref.e}}{Wu_{ref.e}}} = 1.378\cdot Hz$$

D8.3.2.2 Natural frequency of concept

Wind from north

$$Fu_n := \sum_{i=0}^{12} \left(F_{n_i} \cdot u_{n_i}\right) = 1.95 \times 10^4\ J$$

$$Wu_n := \sum_{i=0}^{12} \left[w_i \cdot \left(u_{n_i}\right)^2\right] = 3.809 \times 10^3\ \frac{m^3 \cdot kg}{s^2}$$

Natrual frequency

$$f_n := \frac{1}{2 \cdot \pi} \cdot \sqrt{\frac{g \cdot Fu_n}{Wu_n}} = 1.128 \cdot Hz$$

Wind from east

$$Fu_e := \sum_{i=0}^{12} \left(F_{e_i} \cdot u_{e_i}\right) = 5.907 \times 10^3\ J$$

$$Wu_e := \sum_{i=0}^{12} \left[w_i \cdot \left(u_{e_i}\right)^2\right] = 253.73\ \frac{m^3 \cdot kg}{s^2}$$

Natrual frequency

$$f_e := \frac{1}{2 \cdot \pi} \cdot \sqrt{\frac{g \cdot Fu_e}{Wu_e}} = 2.405 \cdot Hz$$

D8.3 Along-wind induced acceleration

An approximation of the along-wind induced acceleration of a building is calculated with the equation below which can be found in SS-EN 1991-1-4 and the Swedish National Annex.

Along-wind induced acceleration

$$a := k_p \cdot \frac{3\, I_v(H) \cdot R \cdot q_m(H) \cdot b \cdot c_f \cdot \Phi_{1.x}(z)}{m_0}$$

The variables in the equation are stated below.

Peak factor

$$k_p := \sqrt{2 \cdot \ln(v \cdot T)} + \frac{0.6}{\sqrt{2 \cdot \ln(v \cdot T)}}$$

Up-crossing frequency

$$v := f_n \cdot \frac{R}{\sqrt{B^2 + R^2}}$$

Resonance part of response

$$R^2 := \frac{2\pi \cdot F \cdot \Phi_b \cdot \Phi_h}{\delta_s + \delta_a}$$

Size effect of the building

$$\Phi_b := \frac{1}{1 + \frac{3.2 \cdot f_n \cdot b}{v_m(H)}}$$

Size effect of the height of the building

$$\Phi_h := \frac{1}{1 + \frac{2 \cdot f_n \cdot h}{v_m(H)}}$$

Background excitiation

$$B^2 := \exp\left[-0.05 \cdot \frac{H}{h_{ref}} + \left(1 - \frac{b}{H}\right) \cdot \left(0.04 + 0.01 \cdot \frac{H}{h_{ref}}\right)\right]$$

Wind energy spectrum

$$F := \frac{4 \cdot y_c}{\left(1 + 70.8 \cdot y_c^{\,2}\right)^{\frac{5}{6}}}$$

$$y_c := \frac{150 \cdot f_n}{v_m(H)}$$

Mean wind velocity at the top of the building $v_m(H)$

Height of the building $H := 52.65m$

Reference height $h_{ref} := 52.65m$

D8.3.1 Acceleration of concepts when wind from north

Width of the building $b_n := 33.06m$

Referencevelocity for the wind $v_b := 25 \frac{m}{s}$

Factor to change the wind velocity to a velcoty that has a return period of 5 years $\xi_{5year} := 0.855$

Factor to change the wind velocity to a velcoty that has a return period of 1 year (taken from Bjertnaes & Malo (2014))

$$\xi_{1year} := \frac{19.13 \frac{m}{s}}{26 \frac{m}{s}} = 0.736$$

Wind velocity for the dynamic analysis, use the 5 year return value or the yearly return value.

$$v_{m.n} := \xi_{5year} \cdot v_b = 21.375 \frac{m}{s}$$

D8.3.1.1 Deflected mode shape and equivalent building mass

Deflected mode shape (Section F.3 in SS-EN 1991-1-4)

Central reinforced concrete cores $\xi := 1.5$

$$z := 0m, 0.05m .. 52.65m$$

Deflected mode shape $\Phi_1(z) := \left(\frac{z}{H}\right)^{\xi}$

Equivalent building mass (Section F.4 in SS-EN 1991-1-4)

Total weight of Concept 3 (value taken from Appendix D3) $m_3 := 6.205 \cdot 10^6 kg$

Weight per unit length $m_{3.s} := \frac{m_3}{H} = 1.179 \times 10^5 \frac{kg}{m}$

Equivalent buildling mass

$$m_0 := \frac{\int_{0m}^{H} m_{3.s} \Phi_1(z)^2 \, dz}{\int_{0m}^{H} \Phi_1(z)^2 \, dz}$$

D8.3.1.2 Peak factor

Background exitiation

$$B_n := \sqrt{\exp\left[-0.05 \cdot \frac{H}{h_{ref}} + \left(1 - \frac{b_n}{H}\right) \cdot \left(0.04 + 0.01 \frac{H}{h_{ref}}\right)\right]} = 0.984$$

$$B_n^2 = 0.969$$

Resonance part of response
(Section F5 in SS-EN 1991-1-4 and the Swedish National Annex, EKS 9 chapter 1.1.4)

$$y_{c.n} := \frac{150m \cdot f_n}{v_{m.n}} = 7.913$$

Wind energy spectrum

$$F_{es.n} := \frac{4 \cdot y_{c.n}}{\left(1 + 70.8 \cdot y_{c.n}^2\right)^{\frac{5}{6}}} = 0.029$$

Size effect of the building

$$\Phi_{b.n} := \frac{1}{1 + \frac{3.2 \cdot f_n \cdot b_n}{v_{m.n}}} = 0.152$$

Size effect of the height of the building $\Phi_{h.n} := \dfrac{1}{1 + \dfrac{2 \cdot f_n \cdot H}{v_{m.n}}} = 0.153$

Structural damping factor $\delta_s := 0.075$ assumed from table Table F.2 in SS-EN 1991-1-4

Aerodynamic damping factor $\delta_a := \dfrac{c_f \cdot \rho \cdot b \cdot v_{m.n}}{2 \cdot f_n \cdot m_0}$

Air density $\rho := 1.25 \dfrac{kg}{m^3}$

Force coefficient $c_{f.0} := 2.06$ $\psi_r := 1$

$\lambda_n := 1.4 \cdot \dfrac{H}{b_n} = 2.23$

$A_{c.n} := H \cdot b_n = 1.741 \times 10^3 \, m^2$ $A_n := 6m \cdot H = 315.9 \, m^2$

$\varphi_n := \dfrac{A_n}{A_{c.n}} = 0.181$ $\psi_\lambda := 0.96$

$c_{f.n} := c_{f.0} \cdot \psi_r \cdot \psi_\lambda = 1.978$

$\delta_{a.n} := \dfrac{c_{f.n} \cdot \rho \cdot b_n \cdot v_{m.n}}{2 \cdot f_n \cdot m_0} = 6.572 \times 10^{-3}$

Resonance part of response $R_n := \sqrt{\dfrac{2 \cdot \pi \cdot F_{es.n} \cdot \Phi_{b.n} \cdot \Phi_{h.n}}{\delta_s + \delta_{a.n}}} = 0.227$

$R_n^2 = 0.052$

Up-crossing frequency $v_n := f_n \cdot \dfrac{R_n}{\sqrt{(.B)^2 + R^2}}$

$v_n := f_n \cdot \dfrac{R_n}{\sqrt{B_n^2 + R_n^2}} = 0.254 \dfrac{1}{s}$

Averaging time for mean wind velocity $T := 600s$

Peak factor $k_{p.n} := \sqrt{2 \cdot \ln(v_n \cdot T)} + \dfrac{0.6}{\sqrt{2 \cdot \ln(v_n \cdot T)}} = 3.36$

D8.3.1.3 Acceleration

(From the Swedish National Annex, EKS9 chapter 1.1.4).

Turbulence intensity $I_{v.n} := 0.194$

Wind peak velocity pressure $q_{m.n} := \frac{1}{2} \cdot \rho \cdot v_{m.n}^{2} = 285.557\,\text{Pa}$

Acceleration when wind from north $a_n := k_{p.n} \cdot \frac{3 \cdot I_{v.n} \cdot R_n \cdot q_{m.n} \cdot b_n \cdot c_{f.n} \cdot \Phi_1(H)}{m_0} = 0.07\,\frac{\text{m}}{\text{s}^2}$

D8.3.2 Wind from east for Concept buildings

Width of the building $b_e := 37.285\text{m}$

Wind velocity for the dynamic analysis $v_{m.e} := \xi_{5year} \cdot v_b = 21.375\,\frac{\text{m}}{\text{s}}$

D8.3.2.1 Deflected mode shape and equivalent building mass

Deflected mode shape

Central reinforced concrete cores $\xi = 1.5$

$z := 0\text{m}, 0.05\text{m} .. 52.65\text{m}$

Deflected mode shape $\Phi_{1.e}(z) := \left(\frac{z}{H}\right)^{\xi}$

Equivalent building mass $m_3 = 6.205 \times 10^{6}\,\text{kg}$

$m_{3.s} = 1.179 \times 10^{5}\,\frac{\text{kg}}{\text{m}}$

$m_0 = 1.179 \times 10^{5}\,\frac{\text{kg}}{\text{m}}$

D8.3.2.2 Peak factor

Background exitiation $B_e := \sqrt{\exp\left[-0.05 \cdot \frac{H}{h_{ref}} + \left(1 - \frac{b_e}{H}\right) \cdot \left(0.04 + 0.01\,\frac{H}{h_{ref}}\right)\right]} = 0.982$

$B_e^{2} = 0.965$

Resonance part of response

(Section F5 in SS-EN 1991-1-4 and the Swedish National Annex, EKS 9 chapter 1.1.4)

$$y_{c.e} := \frac{150m \cdot f_e}{v_{m.e}} = 16.875$$

Wind energy spectrum

$$F_{es.e} := \frac{4 \cdot y_{c.e}}{\left(1 + 70.8 \cdot y_{c.e}^{2}\right)^{\frac{5}{6}}} = 0.017$$

Size effect of the building

$$\Phi_{b.e} := \frac{1}{1 + \frac{3.2 \cdot f_e \cdot b_e}{v_{m.e}}} = 0.069$$

Size effect of the height of the building

$$\Phi_{h.e} := \frac{1}{1 + \frac{2 \cdot f_e \cdot H}{v_{m.e}}} = 0.078$$

Structural damping factor

$\delta_s = 0.075$ <u>assumed from table Table F.2 in SS-EN 1991-1-4</u>

Air density

$$\rho = 1.25 \frac{kg}{m^3}$$

Force coefficient

$c_{f.0.e} := 2.12$ $\psi_r = 1$

$$\lambda_e := 1.4 \cdot \frac{H}{b_e} = 1.977$$

$A_{c.e} := H \cdot b_e = 1.963 \times 10^{3}\, m^{2}$ $A_e := 3.2m \cdot H = 168.48\, m^{2}$

$\varphi_e := \frac{A_e}{A_{c.e}} = 0.086$ $\psi_{\lambda.e} := 0.99$

$$c_{f.e} := c_{f.0.e} \cdot \psi_r \cdot \psi_{\lambda.e} = 2.099$$

Aerodynamic damping factor

$$\delta_{a.e} := \frac{c_{f.e} \cdot \rho \cdot b_e \cdot v_{m.e}}{2 \cdot f_e \cdot m_0} = 3.689 \times 10^{-3}$$

Resonance part of response

$$R_e := \sqrt{\frac{2 \cdot \pi \cdot F_{es.e} \cdot \Phi_{b.e} \cdot \Phi_{h.e}}{\delta_s + \delta_{a.e}}} = 0.087$$

$$R_e^{2} = 7.527 \times 10^{-3}$$

Up-crossing frequency $v_e := f_e \cdot \frac{R_e}{\sqrt{B_e^2 + R_e^2}} = 0.212 \frac{1}{s}$

Averaging time for mean wind velocity $T := 600s$

Peak factor $k_{p.e} := \sqrt{2 \cdot \ln(v_e \cdot T)} + \frac{0.6}{\sqrt{2 \cdot \ln(v_e \cdot T)}} = 3.305$

D8.3.2.3 Acceleration

Turbulence intensity $I_{v.e} := 0.194$

Wind peak velocity pressure $q_{m.e} := \frac{1}{2} \cdot \rho \cdot v_{m.e}^2 = 285.557\,Pa$

Acceleration when wind from north $a_e := k_{p.e} \cdot \frac{3 \cdot I_{v.e} \cdot R_e \cdot q_{m.e} \cdot b_e \cdot c_{f.e} \cdot \Phi_{1.e}(H)}{m_0} = 0.032 \frac{m}{s^2}$

D8.3.3 Wind from north for Reference building

Width of the building $b_n = 33.06\,m$

Wind velocity for the dynamic analysis $v_{m.n} = 21.375 \frac{m}{s}$

D8.3.3.1 Deflected mode shape and equivalent building mass

Equivalent building mass $m_{ref} := 1.274 \cdot 10^7 kg$

$$m_{ref.s} := \frac{m_{ref}}{H} = 2.42 \times 10^5 \frac{kg}{m}$$

$$m_{0.ref} := \frac{\int_{0m}^{H} m_{ref.s}\, \Phi_1(z)^2\, dz}{\int_{0m}^{H} \Phi_1(z)^2\, dz}$$

D8.3.3.2 Peak factor

Background exitiation $B_n^2 = 0.969$

Resonance part of response

$$y_{c.ref.n} := \frac{150m \cdot f_{ref.n}}{v_{m.n}} = 4.702$$

Wind energy spectrum

$$F_{es.ref.n} := \frac{4 \cdot y_{c.ref.n}}{\left(1 + 70.8 \cdot y_{c.ref.n}^{2}\right)^{\frac{5}{6}}} = 0.041$$

Size effect of the building

$$\Phi_{b.ref.n} := \frac{1}{1 + \frac{3.2 \cdot f_{ref.n} \cdot b_n}{v_{m.n}}} = 0.232$$

Size effect of the height of the building

$$\Phi_{h.ref.n} := \frac{1}{1 + \frac{2 \cdot f_{ref.n} \cdot H}{v_{m.n}}} = 0.233$$

Structural damping factor

$\delta_{s.ref} := 0.1$ assumed from table Table F.2 in SS-EN 1991-1-4

Air density

$$\rho = 1.25 \frac{kg}{m^3}$$

Force coefficient

$c_{f.0} = 2.06$

$\psi_r = 1$ (The same as for the concept)

$\psi_\lambda = 0.96$

$c_{f.ref.n} := c_{f.0} \cdot \psi_r \cdot \psi_\lambda = 1.978$

Aerodynamic damping factor

$$\delta_{a.ref.n} := \frac{c_{f.ref.n} \cdot \rho \cdot b_n \cdot v_{m.n}}{2 \cdot f_{ref.n} \cdot m_{0.ref}} = 5.387 \times 10^{-3}$$

Resonance part of response

$$R_{ref.n} := \sqrt{\frac{2 \cdot \pi \cdot F_{es.ref.n} \cdot \Phi_{b.ref.n} \cdot \Phi_{h.ref.n}}{\delta_{s.ref} + \delta_{a.ref.n}}} = 0.363$$

$R_{ref.n}^{2} = 0.131$

Up-crossing frequency

$$\nu_{ref.n} := f_{ref.n} \cdot \frac{R_{ref.n}}{\sqrt{B_n^{2} + R_{ref.n}^{2}}} = 0.232 \frac{1}{s}$$

Averaging time for mean wind velocity $T = 600\,s$

Peak factor $k_{p.ref.n} := \sqrt{2\cdot\ln(v_{ref.n}\cdot T)} + \frac{0.6}{\sqrt{2\cdot\ln(v_{ref.n}\cdot T)}} = 3.332$

D8.3.3.3 Acceleration

Turbulence intensity $I_{v.n} = 0.194$

Wind peak velocity pressure $q_{m.n} = 285.557\,Pa$

Acceleration when wind from north $a_{ref.n} := k_{p.ref.n}\cdot\frac{3\cdot I_{v.n}\cdot R_{ref.n}\cdot q_{m.n}\cdot b_n\cdot c_{f.ref.n}\cdot\Phi_1(H)}{m_{0.ref}} = 0.054\,\frac{m}{s^2}$

D8.3.4 Wind from east for Reference building

Width of the building $b_e = 37.285\,m$

Wind velocity for the dynamic analysis $v_{m.e} = 21.375\,\frac{m}{s}$

D8.3.4.1 Deflected mode shape and equivalent building mass

Equivalent building mass $m_{ref} = 1.274\times10^7\,kg$

$m_{ref.s} = 2.42\times10^5\,\frac{kg}{m}$

$m_{0.ref} = 2.42\times10^5\,\frac{kg}{m}$

D8.3.4.2 Peak factor

Background exitiation $B_e^{\,2} = 0.965$

Resonance part of response

$$y_{c.ref.e} := \frac{150m\cdot f_{ref.e}}{v_{m.e}} = 9.669$$

Wind energy spectrum $F_{es.ref.e} := \frac{4\cdot y_{c.ref.e}}{\left(1 + 70.8\cdot y_{c.ref.e}^{\,2}\right)^{\frac{5}{6}}} = 0.025$

Size effect of the building $\Phi_{b.ref.e} := \frac{1}{1 + \frac{3.2\cdot f_{ref.e}\cdot b_e}{v_{m.e}}} = 0.115$

Size effect of the height of the building $\Phi_{h.ref.e} := \dfrac{1}{1 + \dfrac{2 \cdot f_{ref.e} \cdot H}{v_{m.e}}} = 0.128$

Structural damping factor $\delta_{s.ref} = 0.1$ assumed from table Table F.2 in SS-EN 1991-1-4

Force coefficient $c_{f.0.ref.e} := 2.12$

$\psi_r = 1$

$\psi_{\lambda.e} = 0.99$

$c_{f.ref.e} := c_{f.0.ref.e} \cdot \psi_r \cdot \psi_{\lambda.e} = 2.099$

Aerodynamic damping factor $\delta_{a.ref.e} := \dfrac{c_{f.ref.e} \cdot \rho \cdot b_e \cdot v_{m.e}}{2 \cdot f_{ref.e} \cdot m_{0.ref}} = 3.136 \times 10^{-3}$

Resonance part of response $R_{ref.e} := \sqrt{\dfrac{2 \cdot \pi \cdot F_{es.ref.e} \cdot \Phi_{b.ref.e} \cdot \Phi_{h.ref.e}}{\delta_{s.ref} + \delta_{a.ref.e}}} = 0.151$

$R_{ref.e}^{\;2} = 0.023$

Up-crossing frequency $v_{ref.e} := f_{ref.e} \cdot \dfrac{R_{ref.e}}{\sqrt{B_e^{\;2} + R_{ref.e}^{\;2}}} = 0.209 \dfrac{1}{s}$

Averaging time for mean wind velocity $T = 600\,s$

Peak factor $k_{p.ref.e} := \sqrt{2 \cdot \ln(v_{ref.e} \cdot T)} + \dfrac{0.6}{\sqrt{2 \cdot \ln(v_{ref.e} \cdot T)}} = 3.302$

D8.3.4.3 Acceleration

Acceleration when wind from north $a_{ref.e} := k_{p.ref.e} \cdot \dfrac{3 \cdot I_{v.e} \cdot R_{ref.e} \cdot q_{m.e} \cdot b_e \cdot c_{f.ref.e} \cdot \Phi_{1.e}(H)}{m_{0.ref}} = 0.027 \dfrac{m}{s^2}$

D8.4 Summary of the results

	Wind from North	**Wind from East**
Acceleration for the Concepts	$a_n = 0.07 \frac{m}{s^2}$	$a_e = 0.032 \frac{m}{s^2}$
Acceleration for the Reference building	$a_{ref.n} = 0.054 \frac{m}{s^2}$	$a_{ref.e} = 0.027 \frac{m}{s^2}$
Natural frequencies for the Concepts	$f_n = 1.128 \cdot Hz$	$f_e = 2.405 \cdot Hz$
Natural frequencies for the Reference building	$f_{ref.n} = 0.67 \cdot Hz$	$f_{ref.e} = 1.378 \cdot Hz$
Top lateral deflection for the Concepts	$u_{n_{13}} = 36.902 \cdot mm$	$u_{e_{13}} = 9.543 \cdot mm$
Top lateral deflection for the Reference building	$u_{n.ref_{13}} = 38.496 \cdot mm$	$u_{e.ref_{13}} = 10.71 \cdot mm$

Appendix D9: Sectional forces in the core

Load combinations in these calculations have been performed according to Eurcode 0 and the Swedish National Annex, EKS 9. The results are presented in Section 7.2.

$kNm := kN \cdot m \quad ton := 1000kg$

Number of floors $n := 14$

$l_y := 11m$

$l_z := 8.8m$

Second moment of inertia when wind from north $I_n := 106.035m^4$ (Values taken from Appendix D8)

Second moment of inertia when wind from east $I_e := 485.954\ m^4$

Centre of gravity of the core when wind from north $z_n := 4.119m$

Centre of gravity of the core when wind from east $z_e := 5.183m$

Cross-sectional area of the concrete core $A_{core} := 12.45m^2$ (Value taken from Appendix D7)

D9.1 Moment due to lateral loads

The moment in the bottom of the building is taken from Appendix D8.

Applied moment in the bottom of the building	When self-weight is unfavourable	When self-weight is favourable
Wind from east, reference building	1.352×10^5 kNm	1.257×10^5 kNm
Wind from east, Concept 3	1.202×10^5 kNm	1.154×10^5 kNm
Wind from east, Concept 4	1.204×10^5 kNm	1.155×10^5 kNm
Wind from north, reference building	1.06×10^5 kNm	1.018×10^5 kNm
Wind from north, Concept 3	1.014×10^5 kNm	9.821×10^4 kNm
Wind from north, Concept 4	1.016×10^5 kNm	9.831×10^4 kNm

Moment when wind from east $M_e := 1.257 \cdot 10^5 kNm$

Moment when wind from north $M_n := 1.018 \cdot 10^5 \cdot kNm$

D9.2 Self-weight of the concrete core

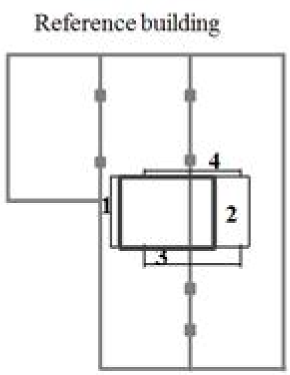
The weight of the concrete core is taken from Appendix D7.

Total weight of the core $m_{core} := 1.922 \cdot 10^{3} \text{ton}$

Load from core $q_{core} := g \cdot m_{core} = 1.885 \times 10^{4} \cdot \text{kN}$

D9.3 Sectional forces in the reference building

The areas are defined as in the figure below.



$A_{floor.1} := 8.8\text{m} \cdot 1.1\text{m} = 9.68 \cdot \text{m}^{2}$

$A_{floor.2.ref} := 8.8\text{m} \cdot 4\text{m} = 35.2\,\text{m}^{2}$

$A_{floor.3.ref} := 10.7\text{m} \cdot 2.15\text{m} = 23.005\,\text{m}^{2}$

$A_{floor.4.ref} := 10.7\text{m} \cdot 1.0\text{m} = 10.7\,\text{m}^{2}$

$$A_{floor.tot.ref} := A_{floor.1} + A_{floor.2.ref} + A_{floor.3.ref} + A_{floor.4.ref} = 78.585\,\text{m}^{2}$$

Self-weight of floor $\rho_{floor.ref} := 475 \frac{\text{kg}}{\text{m}^{2}}$ (Value taken from Appendix D7)

Load from floor $q_{floor.ref} := n \cdot g \cdot \rho_{floor.ref} \cdot A_{floor.tot.ref} = 5.125 \times 10^{3} \cdot \text{kN}$

D9.3.1 Load combination for the reference building

When self-weight and imposed loads are taken as favourable the self-weight should be multiplied with 0.9 and imposed loads should not be considered. If unfavourable self-weight shopuld be multiplied with 1.1 and imposed load with 1.5. The worst case has been found to be when they are considered as favourable. Therefore the imposed load are not included in the expression below.

$$N_{ref} := 0.9 \cdot \left(q_{core} + q_{floor.ref}\right) = 2.158 \times 10^{4} \cdot \text{kN}$$

D9.3.2 Check of tension forces in the concrete core

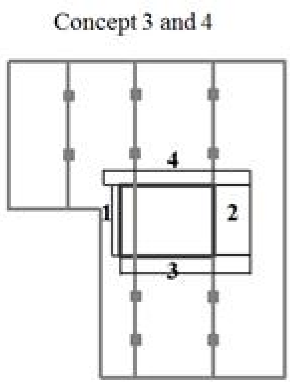
Wind from north

$$\sigma_{ref.n} := \frac{-N_{ref}}{A_{core}} + \frac{M_n}{I_n}\cdot(z_n) = 2.221\cdot MPa$$

Wind from east

$$\sigma_{ref.e} := \frac{-N_{ref}}{A_{core}} + \frac{M_e}{I_e}\cdot(l_y - z_e) = -0.228\cdot MPa$$

D9.4 Sectional forces in Concept 3



$A_{floor.3.1} := 9.68m^2$

$A_{floor.3.2} := 37.1m^2$

$A_{floor.3.3} := 33.0m^2$

$A_{floor.3.4} := 23.3m^2$

$$A_{floor.tot.3} := A_{floor.3.1} + A_{floor.3.2} + A_{floor.3.3} + A_{floor.3.4} = 103.08\,m^2$$

Self-weight of timber floor $\rho_{floor} := 100\frac{kg}{m^2}$

Load from floor $q_{floor.3} := n\cdot\rho_{floor}\cdot g\cdot A_{floor.tot.3} = 1.415\times10^3\cdot kN$

D9.4.1 Load combination for Concept 3

When self-weight and imposed loads are taken as favourable the self-weight should be multiplied with 0.9 and imposed loads should not be considered. If unfavourable self-weight shopuld be multiplied with 1.1 and imposed load with 1.5. The worst case has been found to be when they are considered as favourable. Therefore the imposed load are not included in the expression below.

$$N_3 := 0.9\cdot(q_{core} + q_{floor.3}) = 1.824\times10^4\cdot kN$$

D9.4.2 Check of tension forces in the concrete core

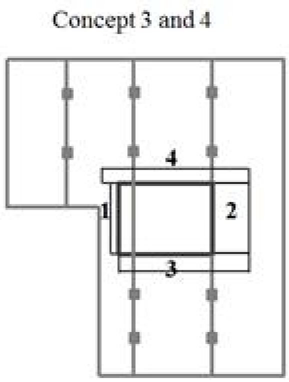
Wind from north

$$\sigma_{3.n} := \frac{-N_3}{A_{core}} + \frac{M_n}{I_n}\cdot(z_n) = 2.49\cdot MPa$$

Wind from east

$$\sigma_{3.e} := \frac{-N_3}{A_{core}} + \frac{M_e}{I_e}\cdot(l_y - z_e) = 0.04\cdot MPa$$

D9.5 Section forces in Concept 4



$A_{floor.4.1} := 9.68m^2$

$A_{floor.4.2} := 37.1m^2$

$A_{floor.4.3} := 33.0m^2$

$A_{floor.4.4} := 7.6m^2$

$A_{floor.tot.4} := A_{floor.4.1} + A_{floor.4.2} + A_{floor.4.3} + A_{floor.4.4} = 87.38\,m^2$

Load from floor $q_{floor.4} := n \cdot \rho_{floor} \cdot g \cdot A_{floor.tot.4} = 1.2 \times 10^3 \cdot kN$

D9.5.1 Load combination for Concept 4

$N_4 := 0.9 \cdot \left(q_{core} + q_{floor.4}\right) = 1.804 \times 10^4 \cdot kN$

D9.5.2 Check of tension forces in the concrete core

Wind from north

$$\sigma_{4.n} := \frac{-N_4}{A_{core}} + \frac{M_n}{I_n} \cdot \left(z_n\right) = 2.505 \cdot MPa$$

Wind from east

$$\sigma_{4.e} := \frac{-N_4}{A_{core}} + \frac{M_e}{I_e} \cdot \left(l_y - z_e\right) = 0.055 \cdot MPa$$