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Parametric Study of a Tall Timber Building in Økern Centre

Master's thesis in Civil and Environmental Engineering
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TITLE:

Parametric Study of a Tall Timber Building in Økern Centre

Parameterstudie av et høyhus ved bruk av tre i Økern sentrum

BY:

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SUMMARY:

In *Økern Centre* there is a need for urban development. This includes an investigation of a tall building placed on *Økerntunnelen*. Wood is believed to be a better option for this purpose due to low self weight. The aim is to have a tall timber structure without exceeding tunnel capacity. The structures studied have a system where diagonals are used for horizontal bracing. This project work is limited to examine ultimate limit state, serviceability limit state, fire capacity and earthquake design of the building.

The first part of the assignment describes applied theory and load definitions. Timber as a construction material and the structural system of *Mjøstårnet* is presented. Loads, dynamics, wind theory, structural fire and seismic procedure follows next. *Dynamo* and *Robot* were used for the modelling and analysis. The structure was modelled as rectangular shape with footprint area on 19,2x32m. Timber decks were made up of GL30c (GLT) beams and Kerto Q (LVL) flanges, with all other elements being glulam bars.

A parametric study was done to map response and accelerations of the timber building. Five models were investigated, with changes made to cross section size, connection stiffness, mass insertion and bracing system. The natural frequency and mass were used to calculate the accelerations. Some combinations of variables were checked for ULS, SLS, fire and seismic design. In the end, a set of final solutions were displayed.

Most challenging design requirement to overcome was the vibration criterion. In acceleration calculations, return period was set to 1-year (c_{prob}=0,73). The most effective way to reduce accelerations was to add stiffness or mass, or both. Neither seismic- nor fire design seemed to be critical. This is because of low seismicity in Norway, and there are used massive cross sections in favour of structural fire.

To avoid exceeding tunnel capacity, the total building weight and base reaction needed to be within the given limit. The maximum base column force was 8768 kN, and the maximum permissible building weight was 7 652 070 kg. Within all design and weight criteria, we were able to design a 14- and 16-storey timber building.

RESPONSIBLE TEACHER: Haris Stamatopoulus

SUPERVISOR: Haris Stamatopoulus

CARRIED OUT AT: Department of Structural Engineering, NTNU

Preface

This master thesis is written as a completion of the 2-year master's program, Civil and Environmental Engineering. The report is fulfilled at the Department of Structural Engineering at the Norwegian University of Science and Technology (NTNU) in the spring semester 2021. Thesis is done in cooperation with *NTNU* and *Trefokus AS* for the project ongoing in *Økern Centre*.

The work consisted of doing numerical analysis and a parametric study of a tall timber truss work building. Through the work we have gained a greater understanding on the challenges and advantageous of using timber as construction material. We learned about the complexities of calculating peak acceleration, and how the results are affected by changes to the building through the parametric analysis. We have also gotten a better understanding on the interaction between *Robot* and *Dynamo*.

A special thanks goes to our supervisor Haris Stamatopoulos for his guidance and all of the helpful discussions throughout the semester. We would also like to thank Aasmund Bunkholt and *Trefokus AS* for providing us with this opportunity. In the end, we would like to thank our fellow students for creating a memorable study experience in Trondheim.

Trondheim, June 2021

Arslan M. Irshad

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Sammendrag

I *Økern Sentrum* er det behov for byplanlegging. Dette inkluderer studie av et høyhus i tre plassert på *Økerntunnelen.* Tre er antatt å være en bedre løsning for dette formålet på grunn av lav egenvekt. Målet for oppgaven er å konstruere et høyhus i tre uten å overskride tunnelkapasiteten. Struktursystemene undersøkt er horisontalt avstivet med diagonaler. Dette prosjektet er begrenset til å undersøke bruddgrensetilstand, bruksgrensetilstand, brannmotstand og jordskjelvsberegninger for bygget.

Den første delen av rapporten beskriver relevant teori og lastdefinisjoner. Tre som et konstruksjonsmateriale og konstruksjonssystemet til *Mjøstårnet* er presentert. Laster, dynamikk, vindteori, brann og seismisk prosedyre følger videre. *Dynamo* og *Robot* ble brukt for modellering og analyser. Modellen ble designet rektangulært med et fotavtrykk på 19,2x32m. Dekkesystemet var komponert av GL30c (limtre) bjelker og Kerto Q (laminert finer) flenser, mens i alle andre elementer ble limtre brukt.

Et parameterstudie ble utført for å kartlegge respons og akselerasjon av tre-bygningen. Fem modeller ble analysert, hvor tverrsnittsstørrelse, stivhet av knutepunkt, innsetting av masse og avstivningssystem ble variert. Egenfrekvens og masse ble brukt til å beregne akselerasjonene. Noen av parametrene ble kombinert og sjekket for bruddgrensetilstand, bruksgrensetilstand, brann og jorskjelv. Til slutt så ble en portefølje av løsninger presentert.

Den mest utfordrende designkrav var å overkomme vibrasjonskriterie. I kalkulasjoner for akselerasjon ble returperioden satt til 1 år (c_{prob}=0,73). Den mest effektive løsningen for reduksjon av akselerasjon var å ha mer stivhet eller masse, eller begge. Verken jorskjelvs- eller brannkrav så ut til å være kritiske. Dette årsakes lite seismisk aktivitet i Norge, og at store tverrsnitsdimensjoner ble brukt til fordel for brannmotstand.

For ikke å overgå tunnelkapasiteten så måtte totalvekten av bygningen samt reaksjonskraft ved fundamentene være innenfor gitte grenser. Maksimal reaksjonskraft ved nederste søyler var 8768 kN, og den totale vekta av bygningen kunne ikke overgå 7 652 070 kg. På bakgrunn av alle design- og vektkriterier, var vi i stand til å muliggjøre 14- og 16-etasjer høyhus ved bruk av tre.

Abstract

In *Økern Centre* there is a need for urban development. This includes an investigation of a tall building placed on *Økerntunnelen*. Wood is believed to be a better option for this purpose due to low self weight. The aim is to have a tall timber structure without exceeding tunnel capacity. The structures studied have a system where diagonals are used for horizontal bracing. This project work is limited to examine ultimate limit state, serviceability limit state, fire capacity and earthquake design of the building.

The first part of the assignment describes applied theory and load definitions. Timber as a construction material and the structural system of *Mjøstårnet* is presented. Loads, dynamics, wind theory, structural fire and seismic procedure follows next. *Dynamo* and *Robot* were used for the modelling and analysis. The structure was modelled as rectangular shape with footprint area on 19,2x32m. Timber decks were made up of GL30c (GLT) beams and Kerto Q (LVL) flanges, with all other elements being glulam bars.

A parametric study was done to map response and accelerations of the timber building. Five models were investigated, with changes made to cross section size, connection stiffness, mass insertion and bracing system. The natural frequency and mass were used to calculate the accelerations. Some combinations of variables were checked for ULS, SLS, fire and seismic design. In the end, a set of final solutions were displayed.

Most challenging design requirement to overcome was the vibration criterion. In acceleration calculations, return period was set to 1-year (c_{prob}=0,73). The most effective way to reduce accelerations was to add stiffness or mass, or both. Neither seismic- nor fire design seemed to be critical. This is because of low seismicity in Norway, and there are used massive cross sections in favour of structural fire.

To avoid exceeding tunnel capacity, the total building weight and base reaction needed to be within the given limit. The maximum base column force was 8768 kN, and the maximum permissible building weight was 7 652 070 kg. Within all design and weight criteria, we were able to design a 14- and 16-storey timber building.

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Chapter 1

Introduction

The thesis will be briefly described in this chapter. This assignment's objectives, boundaries and structure are also addressed.

1.1 Starting Point

This master thesis is a feasibility study done for the external partners. In *Økern Centre*, *Steen & Strøm* and *Storebrand Eiendom* intend to reuse the old centre and create an entirely new district. *Trefokus AS*, in collaboration with NTNU, provided students with the chance to write their master thesis based on this on-going urban development.

One of the issues that must be solved is how to construct high-rise buildings over existing tunnels in the area, as the weight that these tunnels can support is limited. Because of its low self-weight and minimal carbon emissions, timber may be a wise option for these constructions. The goal of this thesis is to convey some principal solutions for tall timber buildings that could be used in *Økern Centre*.

1.2 Problem Description

The thesis will analyse one type of structural system, timber truss work. This structural system is comparable to what *Mjøstårnet* and *Treet* have used. Timber is recognized to be a lighter material than steel and concrete. Due to its low self weight the material is easily put into motion by wind, which makes it more challenging to use in higher structures. The difficulty of satisfying the acceleration criteria of high-rise wooden buildings are well understood.

This study uses a parametric approach to identify how geometrical and other modifications affect the response in both the serviceability- and ultimate limit states. The analysis includes the following:

- Ultimate limit state, ULS
- Serviceability limit state, SLS (acceleration and displacements)
- Seismic evaluation
- Structural fire evaluation

Trefokus AS outlined requirements at the start of the project. *Sweco* provided relevant information concerning design, as well as restrictions on the maximum permitted weight over *Økerntunnelen*, whose are stated in Appendix D [1].

According to *Sweco*, a concrete structure in this place would allow for around eight to ten storeys [1], whereas a wooden building would allow for more than ten storeys due to lighter material.

This project will not include any of the following:

- Detailed solutions
- Acoustic evaluation
- Life cycle cost (LCC) and life cycle analysis (LCA)

1.3 Structure of Thesis

Our thesis will go through design procedure, structural systems and requirements in compliance with Eurocode and *Sweco* requirements. Similar structures have been adopted in the past, and those solutions will be explained. This report will consist of theory, modelling and analysis of our project. The results for static and dynamic finite element analysis will be provided using *Robot Structural Analysis*. *Dynamo Studio* is used to script parametric building geometry. *Microsoft Excel* and *Mathcad Prime* have been used to perform calculations. Following this, there will also be a section of this report dedicated to debate and future work.

Chapter 2

Theoretical Background

The structural system basis theory used in our calculations and thesis will be covered in this chapter.

2.1 Wood as Construction Material

To begin with, wood is neither isotropic nor homogeneous. All properties are determined by the material's orientation. With a good approximation, these directions could be assumed orthotropic. Figure 2.1 depicts three orthogonal directions: longitudinal grain direction (L), perpendicular to grain direction in radial direction (R) and perpendicular to grain direction in tangential direction (T) [2]. Timber, for comparison purposes, does have significantly higher stiffness in the longitudinal direction than in the tangential and radial directions. The wood axes are illustrated in Figure 2.1.

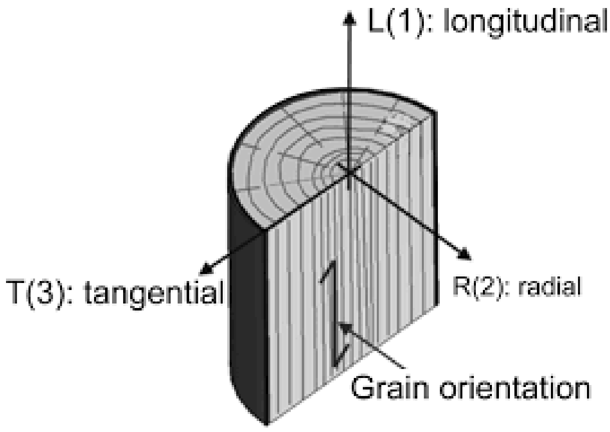


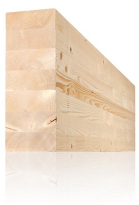
Figure 2.1: Material orientation [32]

As previously stated, timber is a material with high strength and stiffness to weight ratio. The elasticity modulus is low in comparison to steel and concrete, but stiffness is comparable to steel due to its own elasticity-to-mass density ratio. Table 2.1 compares steel, concrete and solid wood [5].

Table 2.1: Comparison of materials

Material	**Elasticity Modulus (E)**	**Mass density (ρ)**	**Ratio (E/ρ)**
Steel (S355)	$2{,}1 \cdot 10^5 N/mm^2$	7800 kg/m^3	27
Concrete (C35/40)	34000 N/mm^2	2300 kg/m^3	15
Timber (C24)	11000 N/mm^2	420 kg/m^3	26

Furthermore, because of the lightness of wood, timber buildings are adventegous for use in urban areas. The use of light-weight timber reduces the amount of foundation needed, which is advantageous for our project. Building higher structures, on the other hand, can lead in acceleration issues that do not meet requirements (Section 3.4.2). The development of engineered wood products takes advantage of these issues in the use of timber in more efficient ways. Today's technology allows us to calculate, use and test wood in entirely new ways. Figure 2.2 portrays the engineered wood products in this thesis, and each is explained.



(a) Glued Laminated Timber, GLT [28]

(b) Laminated Veneer Lumber, LVL [27]

Figure 2.2: Engineered wood products

Glued Laminated Timber, GLT

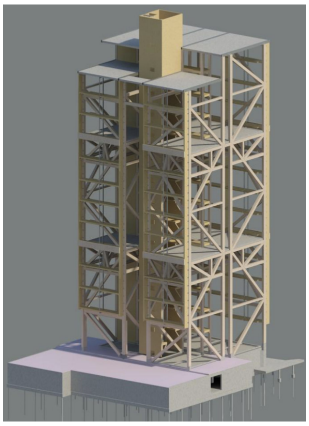
GLT, also known as glulam, is a type of processed wood. The product is constructed by gluing layers of wood into larger cross-sections. Layers can either be directed in-homogeneous or homogeneous, which affects the product's strength. The quality *GL30c* is used in this project. We achieve a much stronger product in structural systems by manufacturing engineered wood products like glulam. The glulam is illustrated in Figure 2.2a.

Laminated Veneer Lumber, LVL

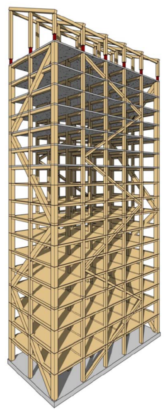
LVL is a type of wood that has been processed. This product is made up of multiple 3 millimeter veneer layers, with all fibers running in the same direction. Laminated veneer lumber is one of the market's strongest engineered wood products. In this assignment, LVL is used for our slab system (Section 4.4.1). The thickness of the product typically ranges from 21 millimeter to 91 millimeter. Slab system in this project employs quality *Kerto Q*. The product is illustrated in Figure 2.2b.

2.2 Structural System

Because of the accelerations on top, wood is typically not used in high-rise buildings. Steel and concrete are considered better alternatives in these structures for dynamic reasons. There seems to be a growing interest in larger and more complex timber structures in recent years. More research and testing has been done recently years due to the wight and sustainability benefits of wood. *Mjøstårnet* and *Treet* are two recent examples, which is shown in Figure 2.3.



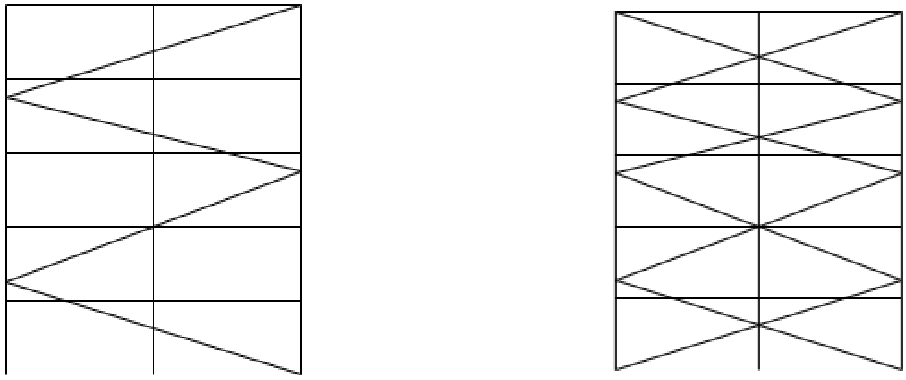
(a) *Treet* in Bergen [36]



(b) *Mjøstårnet* in Brumunddal [35]

Figure 2.3: Structural system of two timber buildings

In this project, two bracing systems are introduced. These two systems are referred to as larger scaled diagonal bracing or larger x-bracing. Both of these bracing systems are depicted in Figure 2.4.



(a) Diagonal braced

(b) Cross- or x-braced

Figure 2.4: Bracing system

Two similar structures will be briefly explained in this part to see how they coped with structural dynamic issues. *Mjøstårnet* and *Treet* are two of the world's tallest timber constructions as of today. For both buildings, the accelerations requrement is met. To withstand horizontal vibrations on the top level, *Mjørstårnet* and *Treet* use diagonal braced system in collaboration with extra concrete mass.

2.2.1 Treet

Treet, is a 14-storey residential building in Bergen. When it was completed in 2015, this construction was the world's tallest timber structure. The total building's height is 51 meters. Diagonals are used to stiffen the entire structure horizontally. Since the timber alone is challenged by motions, "power storeys" were added to the structure. "Power storeys" are floors containing concrete. In addition, CLT shaft were used, but they were not included in structural design [37].

Furthermore, glulam trusswork carries "power storeys". The storeys were installed on the fifth- and tenth floor to add greater mass to the overall construction. These "power storeys" also serve as the foundation for four further storeys above. To provide more weight to the building, a concrete slab was also installed on the roof. More mass equals less acceleration, which satisfies the requirements.

2.2.2 Mjøstårnet

Mjøstårnet, on the other hand, is a higher structure than *Treet*. It is an 18-storey tall timber structure in Brumunddal that was completed in 2019. It is a structural system that is quite similar to *Treet* as mentioned, with huge scaled trusses handling global forces horizontally and vertically to provide necessary rigidity [35]. CLT elements were not used in the design as for *Treet*. The biggest difference between *Mjøstårnet* and *Treet* is that *Mjøstårnet* is 30 meters taller. The highest occupied floor is 68 meters high, with a pergola built above.

Concrete slabs were utilized to add structural weight in a similar way as for *Treet*. Nonetheless, there were no "power storeys" in this structure. These concrete decks were installed on the last seven floors to meet the acceleration criterion. *Mjøstårnet* building consists of apartments, offices and a hotel.

2.3 Dowel-type Connection

Multiple shear connections using slotted-in steel plates and dowel-type fasteners are becoming more common in high-rise timber buildings, and they will be employed in this project to connect diagonals and columns. Large structures, in terms of height or span, necessitate structural elements with large load capacities [20]. A brief description of this sort of connection and the failure modes associated with it will be portrayed.

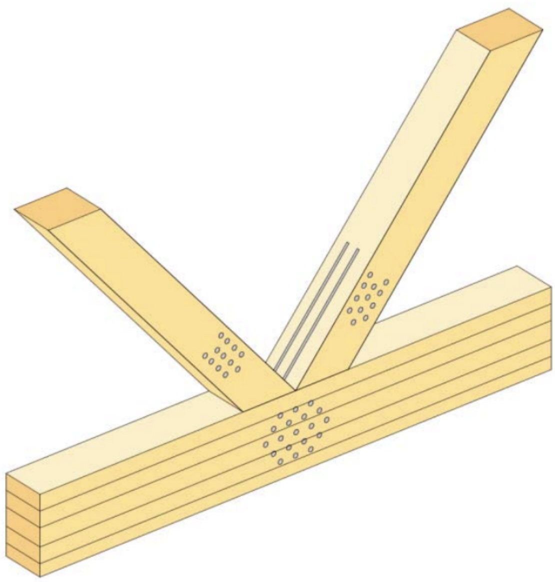


Figure 2.5: Dowel-type connection with multiple slotted-in steel plates

Examples of Usage

There are several contemporary examples of this joint. The connections in *Mjøstårnet* and *Treet* are such examples. Dowel-type connections are appealing because they are simple to construct. Moreover, dowels are fitted into pre-drilled holes not larger than the dowel diameter [18]. This stated connection type allows for reduction in complexity, which is helpful for connecting timber elements in larger structures.

2.3.1 Failure Theory

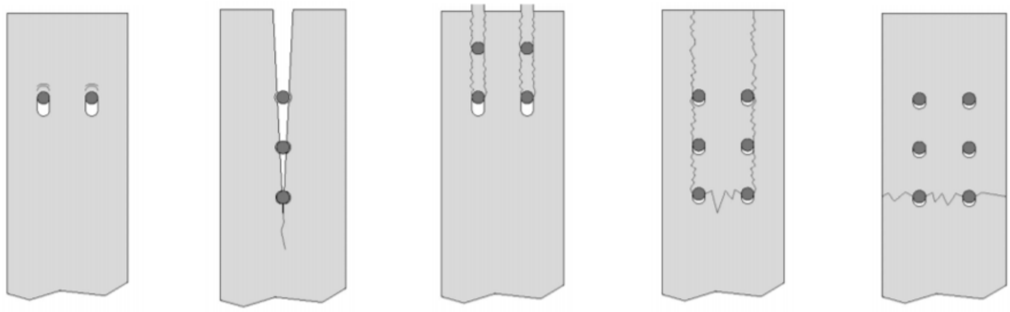
The most common failures for dowel-type connections with several slotted-in-steel plates will be demonstrated and explained. Appendix C2 performs all of the needed checks.

Shear Failure of Dowel-type Fasteners

When dowels are used to make connections, the ductile failure theory is applied. The theory assumes that the fasteners and the wood (or wood-based material) are joined as if they were essentially rigid plastic materials [20]. *Johansen* used this assumption when he created the strength equations for connections made with metal dowel-type connectors in wood. NS-EN1995-1-1, §8.2 [8] includes *Johansen* equations.

The use of multiple slotted-in-steel plates lead to the formation of several shear planes in the connection. The method for calculating this is of the utmost interest. The *European Yield Model* was used to evaluate single and double shear wood connections. According to NS-EN1995-1-1, §8.1.3(1) [8] this method can be used in connections with several shear planes, but no further information is provided. One way to interpret is to calculate the value for a single shear plane and multiply it by number of shear planes. Section 7.1.1 discusses the source of error in taking this approach.

When subjected to lateral loads, a dowel-type fastener connection may fail in a brittle or ductile manner. The design rules in NS-EN1995-1-1 are written in a way that ductile failure is guaranteed rather than brittle failure [18]. Despite this, possible brittle failure modes must be considered (Figure 2.6).



(a) Embedment **(b)** Splitting **(c)** Row shear **(d)** Block shear **(e)** Net tension

Figure 2.6: Brittle failure modes of dowel-type connections

Splitting and Row Shear

Splitting is a crack that shapes along the fasteners as a result of stresses perpendicular to the load [20]. Splitting could arise as a result of tension forces parallel to the grain and tension forces perpendicular to the grain. The tension parallel to grain increases as the spacing between fasteners decreases. Therefore, the minimum spacing specified in Eurocode must be met. By combining *European Yield Model* with an effective number of fasteners per column (n_{ef}), the equation 8.34 in NS-EN1995-1-1 can be used [8].

Block Shear

Block shear failure has been added to Annex A of recent versions of EC5 (NS-EN1995). The block shear (F_B) is defined as the maximum of the capacities of the tensile head plane, the lateral and bottom shear planes [20].

Net Tension

Although net tension is not considered a failure mode of connections, it is established in EC5 (chapter 6.1.2) that the design strength along the grain must be greater than the design tensile strength [20].

Steel Plates

Because there are multiple slotted-in-steel plates, these plates must also be controlled. This was done in accordance with Eurocode 3 (NS-EN1993) [9]. Calculation of it can be found in Appendix C2.

2.3.2 Connection Stiffness

Timber has a low stiffness-to-strength ratio, which results in flexible structural systems. Even when the joint strength criteria are fulfilled in a design, failure due to non-compliance with stiffness criteria is a more likely cause of problems during the design life [18].

Mechanical fasteners are prone to slipping in joints. This is due to tolerance allowances in the connections assembly process, yielding of the fasteners or timber product, or both [18].

Fastener stiffness is defined as ratio of lateral load per shear plane divided by the slip [18]. The slip under any load can be calculated using this relationship. In EC5, this is known as the slip modulus. SLS (K_{ser}) and ULS (K_u) have different stiffness values. The instantaneous slip modulus of the load-displacement curve at approximately 40% of the maximum load that the fastener can withstand [18]. In this project, the slip modulus is calculated in Appendix C2.

2.4 Structural Dynamics

Structural dynamics is concerned with vibrations that occur in systems. Wind and earthquakes, for example, are time-varying loads that cause such vibrations. There are numerous methods for analyzing the dynamic behaviour of structures. This section will go over structural dynamics evaluation and important parameters.

First, we will address the dynamic equilibrium equation. For our thesis, we analyze dynamic response by solving this differential equation and evaluating it using FEM in *Robot Structural Analysis.*

$$\mathbf{m}\ddot{\mathbf{u}} + \mathbf{c}\dot{\mathbf{u}} + \mathbf{k}\mathbf{u} = \mathbf{p} \tag{2.1}$$

where,

$\mathbf{m}$	mass matrix
$\mathbf{c}$	damping matrix
$\mathbf{k}$	stiffness matrix
$\mathbf{p}$	external load
$\mathbf{u}$	velocity vector, dot notation for time derivatives

This dynamic equilibrium on the left is composed of three terms: inertia force($m\ddot{u}$), damping force($c\dot{u}$) and elastic force(ku). Single-degree-of-freedom(SDoF) and multi-degree-of-freedom(MDoF) systems are treated and solved differently. See Figure 2.7 for illustration of the SDoF system. MDoF system is an entry to what has been used in this assignment, namely the modal analysis described in Section 2.4.1.

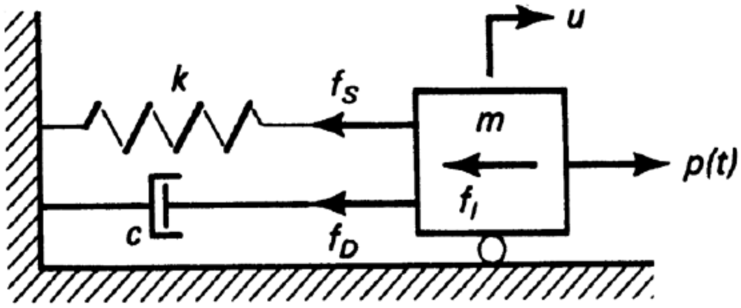


Figure 2.7: System of mass with single translational degree of freedom connected to a spring and a dashpot

2.4.1 Modal Analysis

Most of the structures need more than one single-degree-of-freedom system. Stiffness, mass and damping are assumed to be constant along multi-degree-of-freedom systems. Modal analysis is started out with calculating undamped natural systems of the structure. The outline theory is presented by solving the undamped equation of motion, also known as free vibration analysis [16]. Equation of motion for this kind of system is written

$$\mathbf{m\ddot{u}} + \mathbf{ku} = \mathbf{0} \tag{2.2}$$

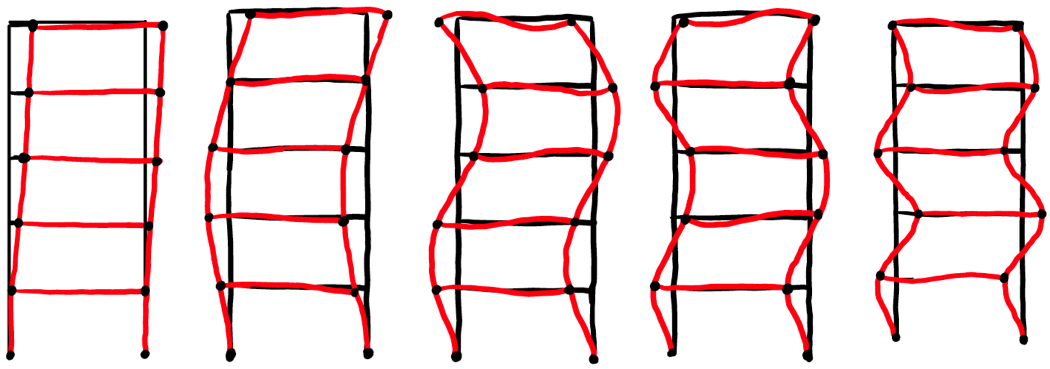
Equation 2.2 will have a solution with all DoFs moving in harmonic motion with natural circular frequency equals to Equation 2.3.

$$\mathbf{u}(t) = \boldsymbol{\phi} sin(\omega_n + \theta) \tag{2.3}$$

where,

t	time variation
$\boldsymbol{\phi}$	mode shape vector of the displacement function
ω_n	circular frequency related to mode shape n
θ	phase angle

It is worth noting that the mode shape vector (ϕ) represents n mode shapes. Each mode shape has a natural frequency associated with it. Figure 2.8 depicts the first five modes of a system.



(a) Mode 1 **(b)** Mode 2 **(c)** Mode 3 **(d)** Mode 4 **(e)** Mode 5

Figure 2.8: First five mode shapes

Furthermore, by inserting solution of the response to mode shape n (Equation 2.3 in Equation 2.2), one can obtain *eigenvalue calculation* problem denoted as,

$$[\mathbf{k} - \omega_n^2 \mathbf{m}] = \mathbf{0} \tag{2.4}$$

By solving the non-trivial solution (trivial solution equals 1) of Equation 2.4, one can solve a set of natural circular frequencies (ω_n) with corresponding mode shape function (ϕ).

Each mode shape function in our thesis is normalized to 1 on the top floor. This is a common technique for evaluating modal analysis. From the first to the n-th mode shape, frequency increases. Seeing as modal analysis decouples the equation of motion, each mode can be solved independently as an SDoF system.

It is not necessary to evaluate all shapes for structural dynamics. Seismic evaluation typically employs a greater number of mode shapes. It is worth noting that *Robot Structural* with FEM-analysis solved mode shapes in all directions (x,y,z). In addition, modal mass is not equivalent to mass, but mass can be obtained from the modal mass. This is also applicable for modal stiffness. Since both (mass and stiffness) matrices are diagonal, they can be decoupled. Decoupled matrices are written

$$\mathbf{M} = \boldsymbol{\phi}^T \mathbf{m} \boldsymbol{\phi} \tag{2.5a}$$

$$\mathbf{K} = \boldsymbol{\phi}^T \mathbf{k} \boldsymbol{\phi} \tag{2.5b}$$

where,

$\mathbf{M}$	modal mass matrix
$\mathbf{K}$	modal stiffness matrix

Moreover, decoupled matrices for damping and external load can be expressed. However, damping matrix is complex and cumbersome to decouple. Orthogonality conditions are needed for decoupling.

2.4.2 Vibrations Parameters

In structural dynamics for tall structures, there are some essential variables to understand. This section will define the terms natural frequency, time period, and damping ratio. These variables are crucial for analyzing dynamic results.

$$\omega_n = \sqrt{\frac{k}{m}} \tag{2.6a}$$

$$f_n = \frac{\omega_n}{2\pi} = \frac{1}{T} \tag{2.6b}$$

where,

ω_n	natural circular frequency [rad/s]
f_n	natural eigenfrequency [Hz]
k	stiffness
m	mass
T	time period [s]

The frequency values indicate the structure's flexibility. Higher structures are more flexible and have a lower frequency due to their slenderness, stiffness and mass. Structures with greater mass have a longer period and a lower frequency, while stiffer structures have a higher frequency. This is important for our thesis, particularly for the accelerations described in Section 3.4.2.1.

Supplementary, dynamic damping affects the motions of the structure. Damping is a complicated phenomenon that is difficult to calculate. One way to determine damping is to measure it using instruments [16]. Logarithmic structural damping is calculated as

$$\delta = \frac{2\pi\xi}{\sqrt{1-\xi^2}} \tag{2.7}$$

where,

δ	logarithmic decrement of strucutrual damping
ξ	structural damping

NS-EN1990-1-4 [7] introduces logarithmic decrement of both aerodynamic and special device damping. In this project, these dampings are not taken into account. We set damping for timber structures as 1,9% as for *Mjøstårnet*.

Chapter 3

Loads

This section will provides an overview and descriptions of the project's main loads and load combinations. All load actions are considered in compliance with NS-EN1991 [4]. In addition, procedures of acceleration, lateral displacements, seismic and structural fire are presented.

3.1 Dead load

The load on the structure varies depending on the materials used in the building. Beams, columns and diagonals are all glulam. Glulam and LVL have been used to build the slab deck. Aside from structural dead weight, no other permanent loads are taken into account (e.g. installations, balconies, walls etc.). However, concrete and green roof are considered as permanent loads in our parametric study (Section 5.2). The dead loads of slab was determined by using Espeland's master thesis [23]. Densities listed are the ones used in this assignment.

Table 3.1: Densities of products used in this thesis

Material	**Density [kg/m^3]**
GLT	390
LVL	480
Deck	1330
Concrete	2500
Wet Green Roof	1900
Dry Green Roof	1600

3.2 Live load

The live load is governed by the building's categorization. Residential, office and commercial use will have different load values. Combination of these categories are to be used for our building shown in parametric study. Live loads are defined by NS-EN1991-1-1. Live loads are defined as (see also Appendix A1).

- Commercial load on 5 kN/m^2
- Office load on 3 kN/m^2
- Residence load on 2 kN/m^2

3.3 Snow load

Only roof top is affected by the snow load. The geographical location, building typology and roof type influence snow load values given in NS-EN1991-1-3 [6]. The procedure in compliance with standard is provided in Appendix A2. However, in Appendix D from *Sweco*, the number is specified as 2.8 kN/m^2, which is used in this project.

3.4 Wind and Acceleration

Wind is omnipresent in our surroundings and a significant factor to consider when designing tall structures. This section will cover the wind and acceleration process and theory. In addition, lateral displacements are presented.

3.4.1 Wind

Firstly, wind is described in terms of modelling. Additionally, important wind theory is explained.

3.4.1.1 Wind Modelling

The goal of wind load modelling, both analytical and physical, is to obtain a comparable static load for the design. This kind of equivalent load assesses the genuine wind loads' variability in time and place, or any dynamic interactions occurring between structure and wind. The wind shifts its velocity as a result of terrain and surroundings. As a consequence, accurately representing of wind on structures is a tough operation. Furthermore, tall building vibrations due to wind must be considered.

The three vibratory components of the wind flow acting on the building are along-wind, crosswind and torsional vibrations [31]. Only along-wind response will be examined in this project.

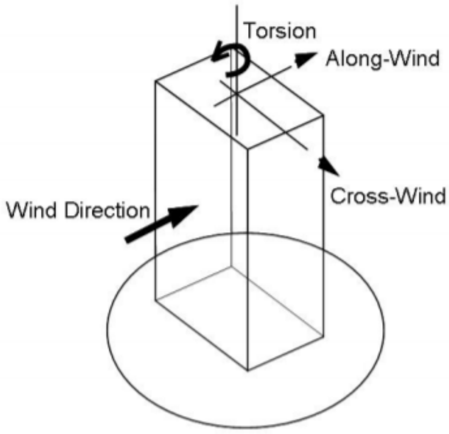


Figure 3.1: Response directions of wind [31]

The along-wind response of a building can be divided into two parts: a mean component and a variable component. The mean component is caused by the average wind speed and can be handled statically. The fluctuating component is wind speed deviations from the mean, known as turbulence (a random process). The shape of the building, surroundings and wind profile influence the random process [31].

The so-called *Gust-factor* methodology is detailed in NS-EN1991-1-4 [7] and employed in this project. It is based on split of wind loading into mean and fluctuating components.

3.4.1.2 Static Wind Load

The equivalent static wind load is calculated in wind standard [7]. The parameters in this method are determined by location and shape of the building. Because the building is presumed to have a dominant external surface, internal wind pressure is ignored. The geometry is expressed as a box structure with reference height z_s (Figure 3.2). NS-EN1991-1-4 is not appropriate for complex geometry.

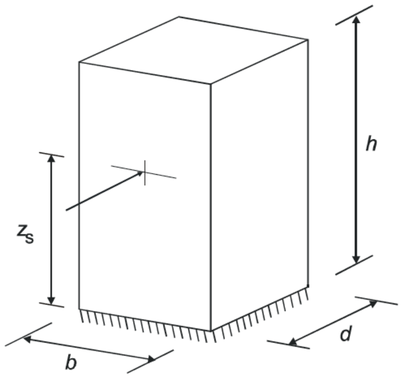


Figure 3.2: Box structure

External wind pressure is taken into account in this project to determine the resulting wind pressure. Only static contribution is considered. Furthermore, the process on acceleration includes dynamic behaviour.

$$F_{w,e} = c_s c_d \cdot \sum w_e \cdot A_{ref} \tag{3.1a}$$

$$w_e = q_p(z_e) \cdot c_{pe} \tag{3.1b}$$

where,

w_e	wind pressure
$q_p(z_e)$	peak velocity pressure at reference height
c_{pe}	external pressure coefficient
$F_{w,e}$	external wind force
$c_s c_d$	structural factor
A_{ref}	reference area of individual surface

The structural factor ($c_s c_d$), which accounts for the fluctuating part of the response, is conservatively set to 1. Wind is modelled and calculated in Appendix A3 in accordance with Equation 3.1b.

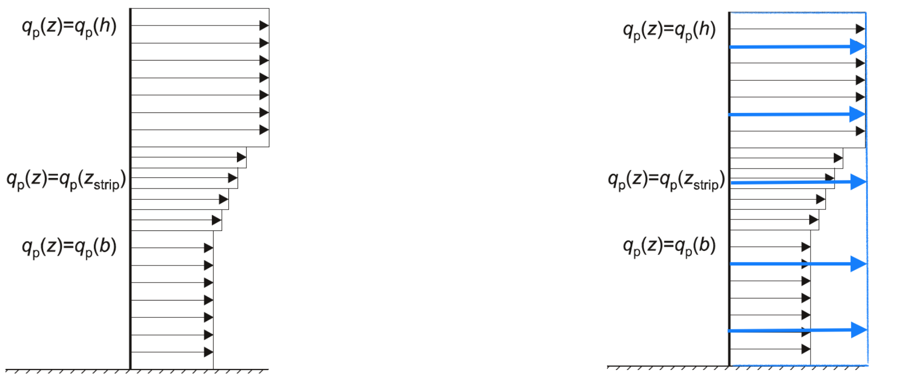
Peak wind velocity is calculated using parameters, such as mean wind velocity (v_m). Our structure is situated on *Økern* with $v_{b,0}$ equalling 22 m/s. Despite wind pressure varies with height, conservatively $q_p(h)$ is uniformly distributed as in Figure 3.3. Further, in accordance with National Annex in wind standard [7], the basic wind velocity is expressed in Equation 3.2.

$$v_b = c_{dir} \cdot c_{season} \cdot c_{alt} \cdot c_{prob} \cdot v_{b,0} \tag{3.2}$$

where,

c_{dir}	direction coefficient
c_{season}	weather season coefficient
c_{alt}	altitude factor
c_{prob}	probability factor dependent on return period of peak wind

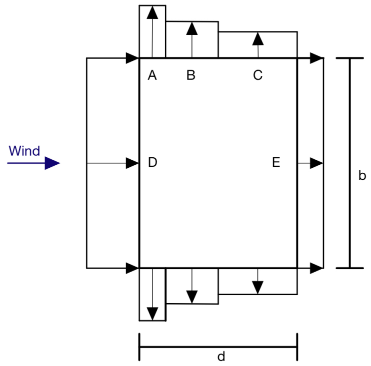
c_{dir}, c_{season}, c_{alt} and c_{prob} are equal to 1 [8]. It is worth noting that c_{prob} is 1 for 50-year return period, while in acceleration calculations is set to 0,73 for 1-year return time (see Section 3.4.2.1). In this project, wind pressure (q_p) is simplified by evenly distribute the maximum magnitude across entire surface (Figure 3.3). Wind- pressure and suction is taken into account for this thesis, but ignored on the roof.



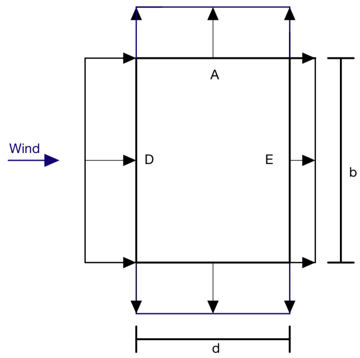
(a) Wind pressure (Eurocode) **(b)** Wind pressure (simplified)

Figure 3.3: Wind pressure force

Each surface is divided into zones A to E. Wind- pressure or suction is determined by a coefficient (c_{pe}). As for peak velocity pressure, maximum pressure magnitude is evenly distributed as in Figure 3.4.



(a) Wind zones (Eurocode)



(b) Wind zones (simplified)

Figure 3.4: Wind pressure force

3.4.2 Acceleration

The intensity of vibrations is determined by accelerations of the upper floor. Wind tunnel testing and Gust factor methodology are two most common ways for estimating accelerations. Wind tunnel testing is more commonly for massive, irregular tall flexible structures, which is not the case here.

3.4.2.1 Calculation Procedure of Acceleration

The acceleration is determined using Annex B in wind standard [7]. Determining accelerations is a complex task due to wind velocity on site. Following formula is used to calculate peak accelerations:

$$a_{peak} = \sigma_{a,i}(z) \cdot k_p \tag{3.3}$$

where,

$\sigma_{a,i}(z)$	standard deviation, where i defines direction x or y
k_p	peak factor, minimum $k_p = 3$

It is worth mentioning that the reference height for accelerations are not as for static wind calculations. For accelerations it is suitable to use z_s (reference height) as in Figure 3.2. Peak acceleration is expressed as:

$$\sigma_{a,i} = \frac{c_f \cdot \rho \cdot b \cdot I_v(z_s) \cdot v_m^2(z_s)}{m_{1,i}} \cdot R \cdot K_i \cdot \Phi_{1,i}(z) \tag{3.4a}$$

$$k_p = \sqrt{2 \cdot ln(\nu \cdot T)} + \frac{0,6}{\sqrt{2 \cdot ln(\nu \cdot T)}} \tag{3.4b}$$

where,

c_f	force coefficient
ρ	air density, equal to 1,25 kg/m^3
b	width of the structure
$I_v(z_S)$	turbulence intensity
$v_m(z_s)$	mean wind velocity calculated with return period of 1 year
R	square root of resonance part of responce
K_i	dimensionless coefficient
$m_{1,i}$	equivalent mass in wind direction, i = x,y
$n_{1,i}$	fundamental frequency along wind direction, i = x,y
ν	up-crossing frequency, $\nu = n_{1,\mathrm{i}}$
T	average time for the mean wind velocity, T = 600 sec

Coefficients mentioned are calculated in NS-EN1991-1-4 [7]. For instance, force factor ($\mathrm{c_f}$) are having major impact on results, and is calculated in wind standard (chapter 7) expressed as in Equation 3.5.

$$c_f = c_{f,0} \cdot \psi_r \cdot \psi_\lambda \tag{3.5}$$

where,

$c_{f,0}$	force factor for rectangle cross section with sharp corners and without free-end flow
ψ_r	reduction factor for square section, $\psi_r = 1$ according to National Annex [7]
ψ_λ	end-effect coefficient for elements with free-end flow, see Appendix

In addition, as stated in Appendix C5.1, variables equivalent mass and non-dimensional coefficient is determined as expressed in Equation 3.6.

$$m_e = \frac{\int_0^l m(s) \cdot \Phi^2(s)\, ds}{\int_0^l \Phi^2(s)\, ds} \tag{3.6a}$$

$$K_i = \frac{\int_0^h v_m^2(z)\Phi_{1,i}(z)\, dz}{v_m^2(z_s) \cdot \int_0^h \Phi_{1,i}^2(z)\, dz} \tag{3.6b}$$

3.4.2.2 Acceleration Requirements

The limitations of acceleration outcomes are not specified in any European codes. In the serviceability limit state, each project can declare its own acceleration criterion. The issue with it is that everyone reacts differently to accelerations. For example, some are more sensitive to vibrations. The acceleration requirement used in this assignment has been established in ISO10137 [12]. The comfort criterion changes with the initial natural frequency as shown in graph (Figure 3.5). Peak acceleration must not exceed the values provided in the assessment curves for wind-induced vibrations.

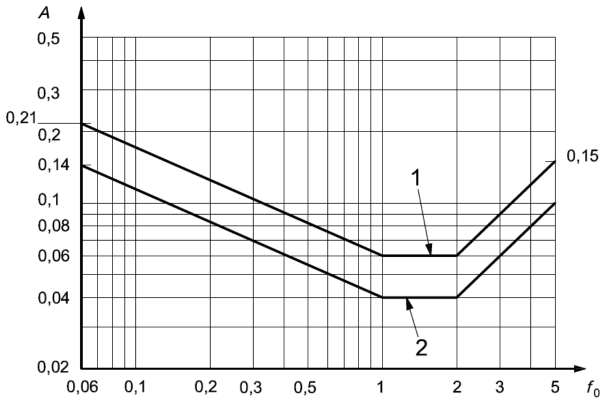


Figure 3.5: Evaluation curves for wind-induced vibrations in buildings [12]

where,

A	peak acceleration [m/s^2]
f_0	first natural frequency [Hz]
1	offices
2	residences

3.4.3 Lateral Displacements

Wind loading generates not only motions, but also lateral displacements of buildings. Displacement criteria in this thesis were created using the timber standard [8].

The maximum criteria for lateral displacement is H/500, where H is entire height of the structure. In addition to lateral displacements, the requirements for interstorey drift is set to h/300, where h is floor height. For displacements and deflections, the characteristic SLS combination is used (see also Section 3.5.2).

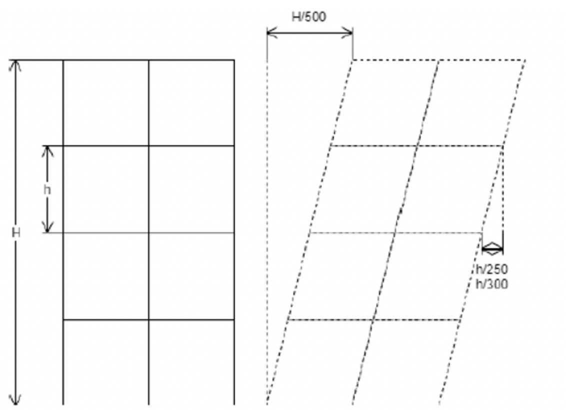


Figure 3.6: Lateral displacements and interstorey drift

The building is displaced horizontally due to lateral actions in Figure 3.6. The figure illustrates lateral displacements and interstorey drift with requirements.

3.5 Load combinations

Since load actions do not have predictable values, a probabilistic approach is required. According to NS-EN1990 [4], load actions must be combined. In this approach, a number of different limit states need to be evaluated. This is done in Appendix A4.

- Ultimate limit state combinations
- Serveciability limit state combinations
- Seismic combinations

Each load combinations must be described related to its limit state. Loads in this assignment to be considered are written.

G	permanent load
Q	live load
S	snow load
W	wind load
E	seismic load

3.5.1 Ultimate Limit State, ULS

To begin, we will go over ULS combinations. These combinations are also employed in the design of structural fire. Each timber member's capacity is determined for the most unfavourable combination. Load combination Equations 3.7 for ULS are given:

$$\sum \gamma_{G,j} \cdot G_{k,j} + \gamma_{Q,1} \cdot \psi_{0,1} \cdot Q_{k,1} + \sum \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \tag{3.7a}$$

$$\sum \gamma_{G,j} \cdot G_{k,j} + \gamma_{Q,1} \cdot Q_{k,1} + \sum \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \tag{3.7b}$$

In this assignment solely Equation 3.7b is considered. For instance, ULS combination 1 is expressed as $1,35 \cdot G + 1,5 \cdot W + 1,05 \cdot S + 1,05 \cdot Q$ in Appendix A4.

3.5.2 Serviceability Limit State, SLS

The design of tall timber structures is intended to be governed by the serviceability limit state. Characteristic SLS load combination are expressed:

$$\sum G_{k,j} + Q_{k,1} + \sum \psi_{0,i} \cdot Q_{k,i} \tag{3.8}$$

Furthermore, e.g. $G + W + 0,7 \cdot S + 0,7 \cdot Q$ is the SLS combination 1 in this thesis. Section 3.4.3 address serviceability limit state design considerations. It is worth mentioning that the maximum horizontal displacement limit is not specified in any standards. That means each project defines their own comfort criteria for displacements. Maximum displacements are already presented in Section 3.4.3.

There are a few key factors to take in mind while doing acceleration calculations. Mass considered in dynamic analysis is given by quasi-permanent serviceability limit state combination. Equation 3.9 (quasi-permananent combination) state that 30% of live loads are added as mass to the global structure. This is favourable for our global structure due to accelerations comfort criterion.

$$\sum G_{k,j} + 0,3 \cdot Q_{k,1} \tag{3.9}$$

3.5.3 Seismic Combinations

When studying seismic response, there are three types of combinations to consider [42].

Firstly, consider *the modal responses combination.*. The periodic modal responses of an earthquake should be coupled with one of the following combination rules, as detailed with formulas in Section 3.6:

- SRSS (Square root of sum of squares):
- CQC (Complete quadratic combination)

Secondly, *the combination of the responses to the seismic spatial components.* These are the directional effects of an earthquake. In this thesis, the vertical component of the seismic effects will be neglected, since $a_{vg} < 0,25g$, see Appendix B2. Each directional response, $\mathrm{E_{Edx}}$ and $\mathrm{E_{Edy}}$ should be calculated using the combination rules introduced above (modal response combination). Once these horizontal components of the seismic action are found they can be combined with the following rules, according to NS-EN1998, §4.3.3.5.1:

- SRSS (see Equation 3.17)
- CQC (see Equation 3.18)
- The 30% rule:

$$E_{Edx} + 0,3 \cdot E_{Edy} \tag{3.10a}$$

$$E_{Edy} + 0,3 \cdot E_{Edx} \tag{3.10b}$$

The 30% rule is recommended as an alternative to the SRSS and CQC methods in seismic standard (§4.3.3.5.1(3)). The 30% criterion was applied in *Robot* as proceed in Appendix B2.

Finally, *the interaction with other loads.* When the seismic design load E_d in each direction is determined, it must be combined with the dead and live loads according to NS-EN1990-1-1 [4]. Because there are no prestress loads in this thesis, this will not be included in the formula:

$$E + G + \sum \psi_{2i} \cdot Q_{ki} \tag{3.11}$$

3.6 Seismic Calculation

This section presents the method used in this thesis to calculate the resulting forces from earthquake. Design done is in compliance with the European earthquake standard [11]. On a global scale, Norway is classified as low to intermediate seismicity area (Figure 3.7). It is still necessary to analyze the seismic loads. This is because it is more critical for larger structures due to geometry, natural circular frequency and damping ratios. In addition, some buildings (hospitals, infrastructure, etc.) are more crucial and essential, thus needed to be satisfied for earthquake.

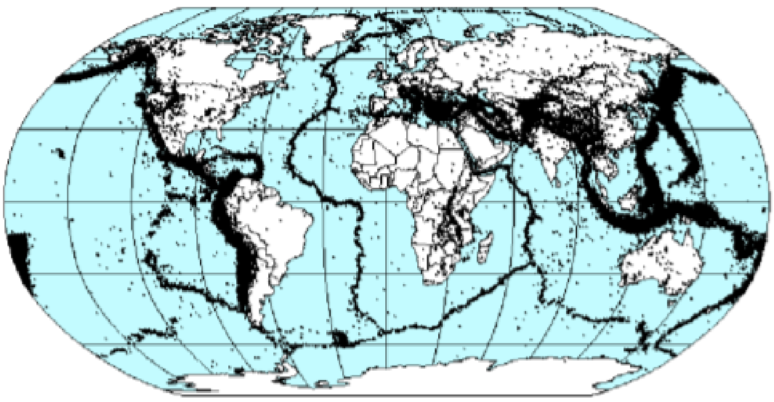


Figure 3.7: Seismological world map showing registred epicenters for earthquakes, with magnitude over 3.5 on the Richter's scale, in the period 1963-1998 [23]

3.6.1 Representation of Earthquake Loading

There are several ways to depict the impact of an earthquake on a structure. The two most common methods are as follows:

- Time history
- Response spectrum

Only the response spectrum method is explained and used in this thesis. This method does not provide a perfect prediction of the peak response, but it does provide an accurate estimate for structural design [19].

The maximum response of a one-degree-of-freedom system to a given earthquake is portrayed by a response spectrum [30]. Linear elastic system's response is determined by its natural circular frequency and damping ratio. Following the response spectrum calculation, all SDoFs can be used for seismic design [19]. In this kind of calculation, no dynamic study is needed as it is done in the spectrum.

Response spectrum is built on the foundation of the dynamic equation of equilibrium. For a linear SDoF system subjected to ground acceleration, it is expressed as follows:

$$\ddot{u}(t) + 2\xi\omega\dot{u}(t) + \omega^2 u(t) = -\ddot{u}_g(t) \tag{3.12}$$

This allows the response spectrum to provide a value for each circular frequency, but it is only valid for one damping condition.

For each dynamic displacement of the degree of freedom (u(t)), there is an equivalent static load that would have resulted in the same displacement. This load can be calculated from the stiffness relation of the system:

$$F(t) = k \cdot u(t) = m\omega^2 u(t) \tag{3.13}$$

When the equivalent static load is known, static analysis of the system can be used to determine any type of response. As a result, the analysis transitions from dynamic to static, which is better in terms of simplicity and efficiency.

The pseudoacceleration of a system ($S_e(t,T,\xi)$) is the acceleration that must be multiplied by the mass of system to achieve the equivalent static load. This means that it is directly related to the load on the system. Equation of pseudoacceleration is denoted as:

$$F(t) = k \cdot u(t) = m\omega^2 u(t) = m \cdot S_e(t,T,\xi) \Longleftrightarrow S_e(t,T,\xi) = \omega^2 u(t) \tag{3.14}$$

As Equation 3.14 implies, $S_e(t,T,\xi)$ is affected by the direction of the ground acceleration as well as system's natural frequency, period, mass and damping. Regardless the response studied, it is the numerically largest value of the response that is of interest. This is the value designed for. The elastic material requirement has been met if system can withstand maximum force (F_{max}) without any plastic deformation. The greatest shear force is expressed as:

$$F_{max} = m \cdot S_e(T, \xi) \Longrightarrow S_e(T, \xi) = S_e(t, T, \xi)_{max} \tag{3.15}$$

A design spectra is created by combining numerous response spectra to account for the irregular nature of earthquakes. This is depicted in a diagram (Figure 3.8).

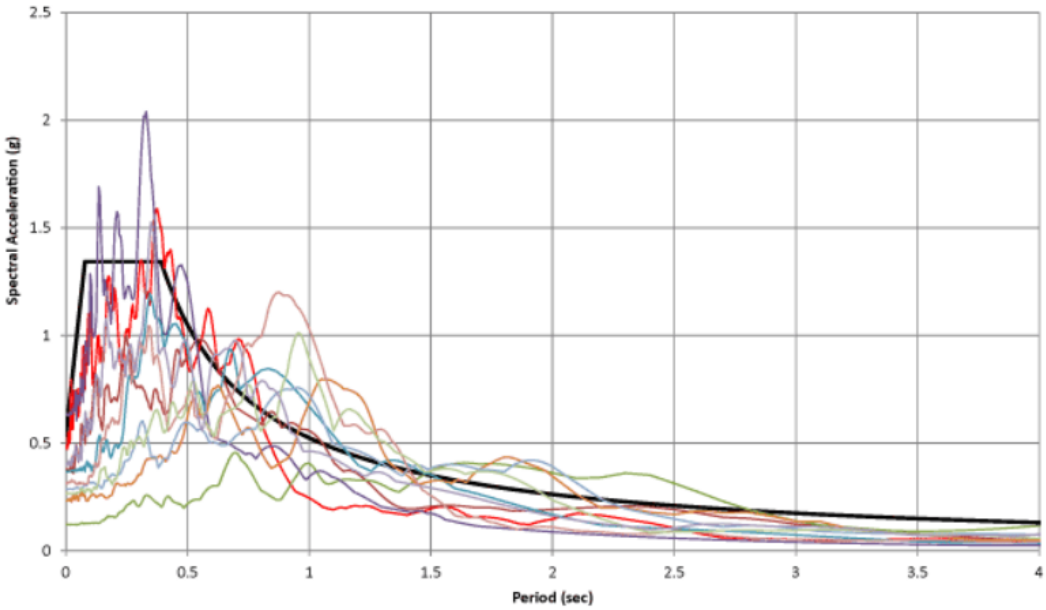


Figure 3.8: Example of design spectra based on the pseudoacceleration of several response spectra [30]

3.6.2 Representation of Earthquake Loading according NS-EN1998-1

According to seismic standard, there are primarily two ways for defining linearly earthquake loading.

- Lateral force method of analysis
- Modal response spectrum analysis

The method used in *Robot* is modal response spectrum. The contribution of each modal form to the total response of the structure is the basis for the modal response spectrum. The period and corresponding mass of the mode determine the modal shape response [19].

Also important is the relationship between the frequency of the mode and the frequency of the seismic waves. This relation influence size of design spectrum value [19]. NS-EN1998-1 denotes the shear force at foundation level that building is subjected to.

$$F_{b,k} = S_d(T_k) \cdot m_k \tag{3.16}$$

where,

$S_d(T_k)$	design spectrum value for modal shape k
m_k	mass of modal shape k

All mode shapes that contribute significantly to the global response must be considered. It is also claimed that if there are enough modal shapes, this requirement can be considered met [11]. It is required that modal shapes to be included to account minimum 90% of overall mass, which has been done in this project.

Loads from each mode shape must be added to get the total seismic force on a structure. Because it is improbable that each model form will experience their maximum load at the same time, simply adding them together is rather conservative. Seismic standard includes the summation methods listed:

- Square root of the sum of the squares (SRSS)
- Complete quadratic combination (CQC)

The simplest method is SRSS, which is often used for manual calculations. This method ignores the intersection of multiple different forms of oscillations. As a result, in order for SRSS, NS-EN1998-1 needs the modal shapes to be independent of one another. This is stated in Equation 3.17.

The other method, CQC, takes into account the correlation between two adjacent modal shapes (i and k). As a result, it gives more accurate findings for mode shapes that are not fully independent from one another. This method is not time expensive in softwares, and therefore used in this thesis. CQC combination is expressed in Equation 3.18.

$$E_E = \sqrt{\sum E_{Ei}^2} \tag{3.17}$$

where,

E_E	is the value of the total seismic loading.
E_E	is the value of the seismic loading due to modal shape k.

$$E_E = \sqrt{\sum_{j}^{n} E_{E.k} \cdot \sum_{i}^{n} E_{E.i} \cdot \rho_{k.i}} \tag{3.18}$$

where,

$\rho_{k.i}$	is the correlation coefficient for modal shape k and i.

3.7 Structural Fire

Performance requirements are computed in accordance with TEK17 and fire standard [29] [10]. Although wood is combustible, improved fire protection for timber has been created through time. When wood is heated up, it loses strength. Comparison with other materials is shown in Figure 3.9.

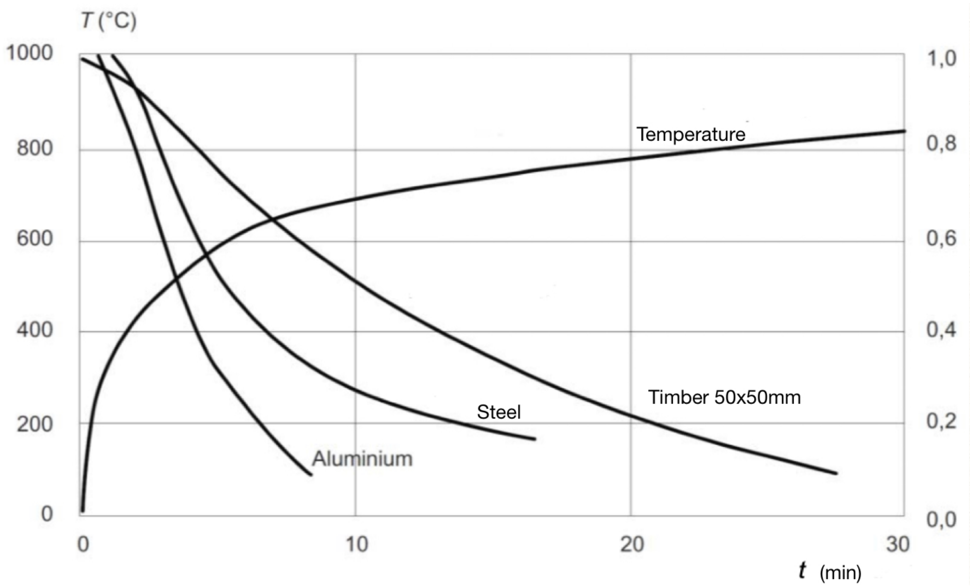


Figure 3.9: Strength properties of materials exposed to fire [2]

In a fire situation, the most crucial factor is to save lives. Structures have an important role in fire safety. For example, fire in high-rise timber structures should be contained in cells to avoid fire- and smoke spread. Additionally, ensuring structure for certain minutes specified is needed in fire exposure. As a conservative simplification, no protective coating (gypsum, fire paint etc.) is contemplated in this structure.

It should be noted that the decks are expected to meet the fire requirements. Fire has varied effects on different timber members in our design. Fire exposure on beams is one sided, exposure on diagonals is three-sided and exposure on columns is all-sided (four-sided). See depicted Figure 3.10 in Section 3.7.2.

3.7.1 Fire Requirements

To classify a building's fire resistance, the load carrying capacity (R), integrity (E), insulation (I), and duration time in minutes are all used. The classes in *Byggteknisk Forskrift* determine the safety criteria for structural fires[29].

- Risk class
- Fire class

The risk classes describe how the building is used, whereas the fire class indicates the severity of the consequences of a structural fire [29]. In our system (building with more than 5 storeys) has a risk class of three to five dependent on office, residential or commercial use. Moreover, fire class is assigned between three and four based on number of storeys and the structure. In this project, we deduce our structural bearing system has fire resistance R90 A2-s1, d0 [29]. Only the primarily load carrying system (beams, columns and diagonals) is checked for fire resistance.

The building structure should be constructed in such a way that fires are less likely to occur. Fire classes three and four must meet certified sufficient load bearing capacity and stability during exposure. In addition, buildings with more than eight storeys, a second staircase is required. Fire safety criteria for staircases, decks, fire cells and doors are not considered.

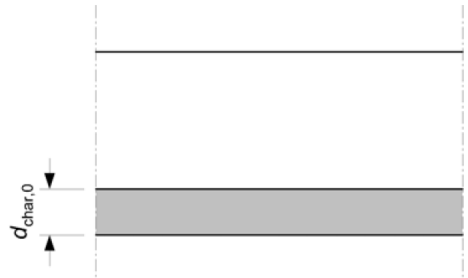
3.7.2 Fire Design

NS-EN1995-1-2 will be used to define fire resistance design approach in timber systems. The methods used is listed:

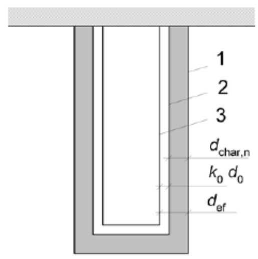
- Reduced cross section method
- Reduced strength method
- Complex calculations based on charring models, temperature profiles and cross section moisture gradients together with strength variation with temperature and moisture.

The structural system's fire design is done using the *reduced cross section* method. Principal load bearing elements would be beams, columns and diagonals. These elements are checked in Appendix C4. Structural system is satisfied when load-bearing is maintained for minimum of 90 minutes (see Section 3.7.1).

Any surface exposed to fire will char, according to fire standard [10]. Non-protective surfaces burn with a constant rate. The char will insulate the wood's inner core, and the charring depth will not bear any load. After specified time of fire exposure, the residual cross section must support at least 60% of load actions. Reduced cross sections with variables is illustrated in Figure 3.10.



(a) Charring on one surface



(b) Charring on three surfaces

Figure 3.10: Reduced cross section [10]

Formulas used in NS-EN1995-1-2 to determine reduced cross section is expressed as:

$$d_{ef} = d_{char,n} + k_0 d_0 \tag{3.19a}$$

$$d_{char,n} = \beta_n \cdot t \tag{3.19b}$$

where,

d_{ef}	effective charring depth
$d_{char,n}$	notional design charring depth
k_0	coefficient
d_0	equal 7mm
β_n	notional design charring rate
t	time for fire exposure

Chapter 4

Modelling

This chapter will provide a review of the modelling in *Dynamo Studio* and *Robot Structural Analysis*, as well as an explanation of the various elements of the structural system - both their construction and how they are modelled. For the sake of certainty, a verification of *Robot* has been done, and it will be presented in Section 4.2. Finally, the five main models studied in the thesis will be introduced, along with their geometrical and structural features.

4.1 Modelling in Software

This section explains how to use the two separate software programs for this project. In our modelling, *Robot Structural Analysis* and *Dynamo Studio* are employed.

4.1.1 Usage of *Dynamo Studio*

Dynamo Studio was utilized for visual programming in order to investigate parametric conceptual design. *Autodesk Revit* includes this application as a built-in app. On the other hand, *Dynamo Sandbox*, an open source and free application, can be used. In this project, *Dynamo* was used to create geometry to connect with *Robot Structural*.

The connection of *Dynamo* and *Robot* has been carried out by a package that has been installed and is known as *Strucutral Analysis with Dynamo*. This add-on package allows *Dynamo* to communicate with *Robot* by converting lines into bars, surfaces into floors, and points into supports or connection stiffness. Load cases may also be added directly into the visual programming [33].

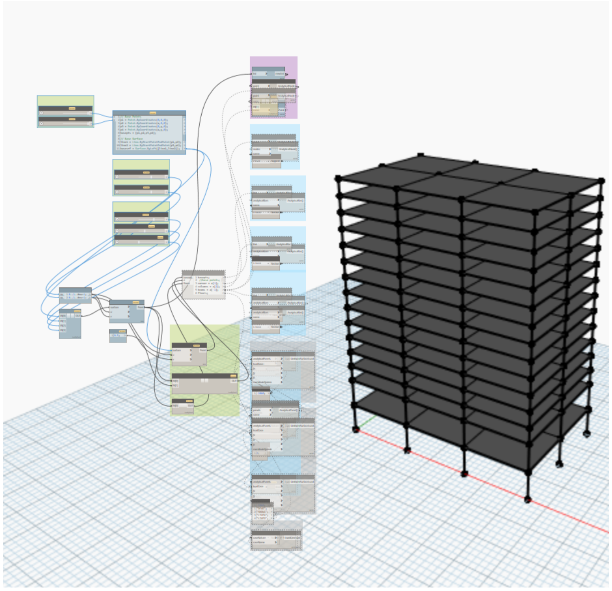


Figure 4.1: Illustration of *Dynamo*

Our research employs *Dynamo* to simplify and improve the efficiency of the analysis in *Robot*. The parametric script allowed us to simply adjust the building shape (footprint, storeys, bays, height and floor height), as shown in Figure 4.1. Not only is building geometry parametric, but so are the following parameters: connection stiffness, support conditions, and load cases. It is worth noting that rigid links in beams and columns have been employed by *Python* scripting inside *Dynamo*. The reasoning behind the rigid links can be found in Section 4.4.2. The *Dynamo* technique can be found in Appendix B1

4.1.2 Modelling in *Robot Strucutural*

This program was used for the majority of our calculations in our thesis. *Robot Structural Analysis* is a structural engineering computational program. *Robot* verifies code conformance and performs static and dynamic analysis using finite element analysis (FEM).

Furthermore, in Section 4.4, all structural elements simulated in *Robot* are discussed. Verification of *Robot* was also performed by a similar structural analysis tool, *Focus Konstruksjon*, as described in the following Section 4.2.

4.2 Verification of *Robot Structures*

Due to the complexity of the modelling process it was decided to do a verification of *Robot*. The authenticity of the results retrieved from Robot was thus ensured. The statics (M/V/N), the displacements (U_z and U_x), and the natural frequencies (first three modes), have all been validated. The following two models (two-dimensional and three-dimensional) were compared:

- Glulam truss model in 2D
- 5-storey glulam model in 3D

Focus Konstruksjon was used to perform the verification. *Focus* is a Microsoft Windows FEM analysis program that employs the finite element approach to solve both complex and basic 2D and 3D issues. It is believed that by employing simple 2D and 3D models in the verification, the quantity of modelling errors (in our modelling in *Robot* and *Focus*) will be minimal, allowing for reliable comparisons.

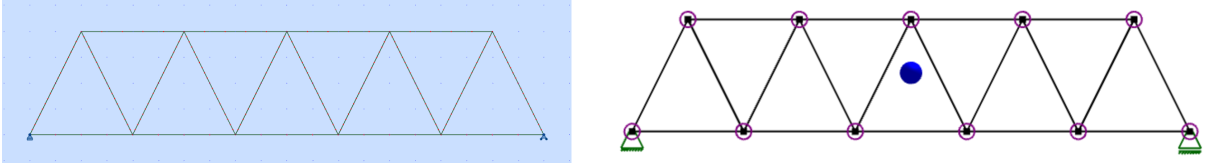
Both models are constructed using FEM computations, with each bar divided into 20 elements. To simplify things, all factors are set to 1. Shear deformation has been accounted for.

4.2.1 2D Model Comparison

The 2D truss-model in *Robot* and *Focus* was modelled according to the following input:

Table 4.1: Input for 2D comparison models

Input	
Total height [m]	2
Total length [m]	10
Line load on top chord [kN/m]	10
Gravity constant [m/s^2]	9,81
Boundary condition	Simply supported
Material	Gl30c
Cross-section dimension for all bars [mm]	115x300



(a) Robot **(b)** Focus

Figure 4.2: 2D comparison models in *Robot* and *Focus*

The results for the 2D model in *Focus* and *Robot* are presented in Table 4.2 and 4.3:

Table 4.2: Results of forces and displacements in 2D model

	Robot Structures	**Focus Konstruksjon**	**Deviation**
$U_{z.max}$	2,44mm	2,50mm	2,46%
$M_{y.max}$	3,52 kNm	3,52 kNm	0,28%
V_{max}	11,65 kN	11,10 kN	4,95%
$N_{max}(Compression)$	56,12 kN	56,10 kN	0,04%
Mass	543,05kg	543,00kg	0,01%

Table 4.3: Frequency of first three modes in 2D Model

	Robot Structures	**Focus Konstruksjon**	**Deviation**
Mode 1	46,36Hz	45,92Hz	0,96%
Mode 2	79,16Hz	78,44Hz	0,92%
Mode 3	119,17Hz	118,83Hz	0,29%

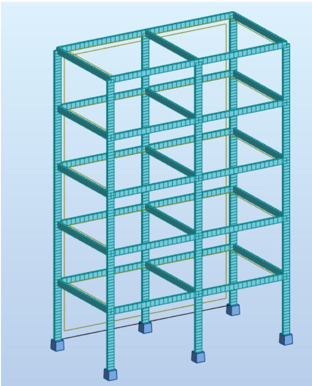
There are some minor differences, as shown by the tables 4.2 and 4.3. These could be the result of varying round-off in the two programs. The changes in outcomes are insignificant, indicating that the simulations are correct.

4.2.2 3D Model Comparison

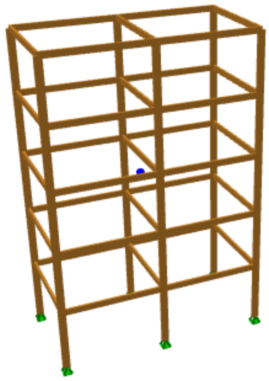
This comparison was carried out to ensure that the simulation of a 3D model produces logical numbers for deformations, reaction forces and modal analysis.

Table 4.4: Input for 3D comparison models

Input	
Total height [m]	15
Plan dimension (bxd) [m]	5x10
Storey height [m]	3
Length of beams between columns [m]	5
Lateral surface load in shortest direction [kN/m^2]	0,5
Vertical surface load on roof [kN/m^2]	1
Gravity constant [m/s^2]	9,81
Boundary condition	Fixed
Column cross-sections [mm]	115x360
Beam cross-sections [mm]	115x300
Material	Gl30c



(a) Robot



(b) Focus

Figure 4.3: 3D comparison models in *Robot* and *Focus*

The results for the 3D Model are presented in Table 4.5 and 4.6:

Table 4.5: Results of forces and displacements in 3D model

	Robot Structures	**Focus Konstruksjon**	**Deviation**
$U_{z.max}$	7,25mm	6,80mm	6,62%
$U_{x.max}$	196,90mm	196,50mm	0,20%
$M_{y.max}$	35,32mm	35,34mm	0,06%
V_{max}	18,84mm	18,92mm	0,42%
$N_{max}(Compression)$	94,50mm	93,86mm	0,68%
Mass	3807,77kg	3808,00kg	0,01%

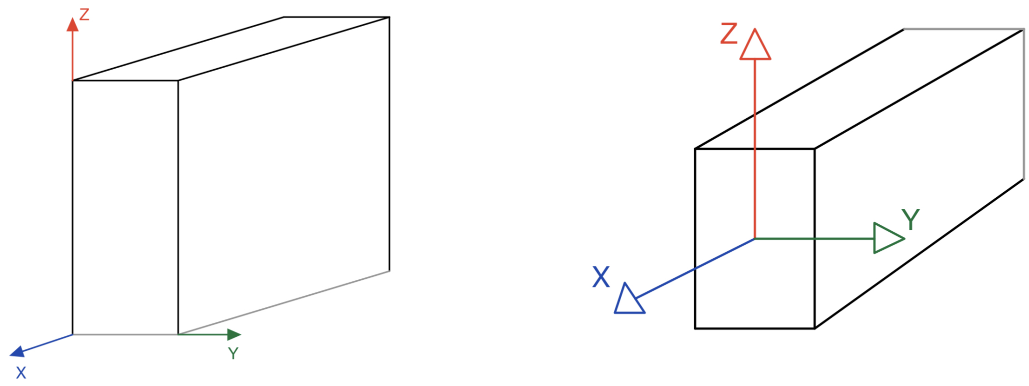
Table 4.6: Frequency of first three modes in 3D Model

	Robot Structures	**Focus Konstruksjon**	**Deviation**
Mode 1	1,72Hz	1,69Hz	1,78%
Mode 2	2,31Hz	2,25Hz	1,32%
Mode 3	3,08Hz	3,05Hz	0,98%

The results suggest that the vertical deformation deviation is a little excessive. This implies that the *Robot* model is less stiff in the z-direction than the *Focus* model. The vertical deformation of beams is not that relevant for the purposes of this thesis, as it is in the frequencies, response forces and horizontal deformation that must be checked for acceleration, fire, earthquake and top displacement criterion. The reaction forces and natural frequencies do not change significantly, whereas the deflection in the x-direction deviates by roughly 0.2%. These deviations are not seen as concerning. As a result, it has been concluded that the simulations in *Robot* are valid, and its use is justified as long as the modelling is done accurately.

4.3 Axis System Defined in Project

In addition, an axis system has been developed for use in this project. The x-direction, for example, determines the longitudinal direction of the construction. Figure 4.4 shows local and global axes, with the local axis system used for structural elements and the global axis system used for the entire building.



(a) Global axis

(b) Local axis

Figure 4.4: Axis system defined

4.4 Structural Elements

The modelling of the structural elements will now be explained.

4.4.1 Slabs

The slabs can be designed in a number of different ways. Timber composite floors were utilized to create greater spans and consequently more flexible use of the structure.

Design

Numerous varieties of timber composite slabs have been produced through the *WoodSol* research program, as well as development from companies such as *Moelven*, *Stora Enso*, *Metsä Wood* and *Lignatur*. It is helpful to have an understanding of what is presently available on the market while looking for an appropriate design for a slab. This summary, as well as numerical simulations of six distinct wooden composite slabs, may be found in Bjørge and Kristoffersen's [26] master thesis. The slabs are modelled according to this system.

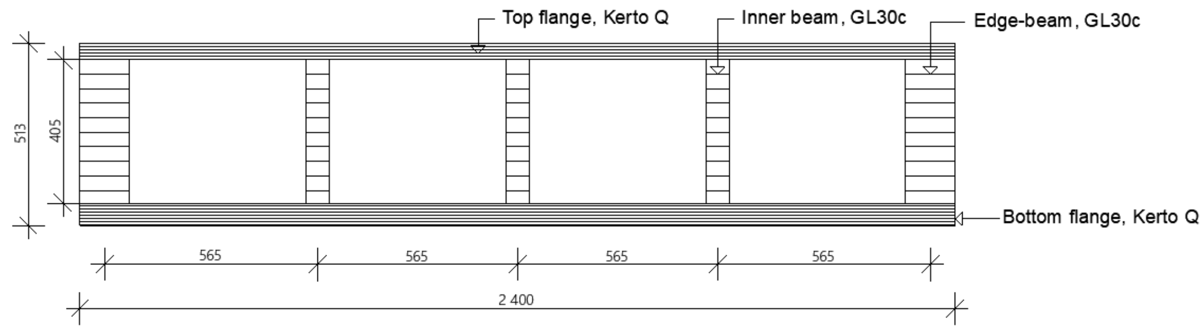


Figure 4.5: Section of the timber composite slab [26]

where,

Top flange:	Kerto Q (LVL), $t = 45mm$
Bottom flange:	Kerto Q (LVL), $t = 63mm$
Inner beams:	GL30c, $66mm \cdot 405mm$
Edge beams:	GL30c, $140mm \cdot 405mm$
Total width of slab-element:	$b = 2400mm$
Center-distance for beams	$C/C = 565mm$

The slabs will be connected to the beams in the manner depicted in Figure 4.6. The top plate is placed on the support (beam or wall) by cantilevering it. This connection is what holds the entire slab together. The connection solution is also used in *Mjøstårnet* [35]. The initial benefit is that no construction height is lost due to the slab. The height of the beams that support the slab must be maintained, and by putting the deck in between, just the plate thickness is lost from the floor height. Another advantage is that it is incredibly simple to put together. Because of this assembly process, the slabs will be represented in *Dynamo* and *Robot* at the same level as the beams, which simplifies the modelling.

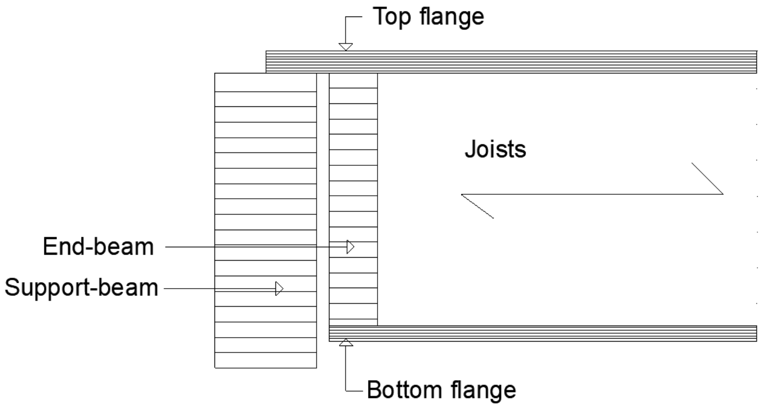


Figure 4.6: Support of slabs

It was important to know exactly how long the spans could be using the slab-system indicated in figure 4.5, while keeping safety and comfort parameters in mind. As a result, a preliminary design was carried out in accordance with timber standard [8]. The results suggest that the critical criterion for the slabs is human-induced vibrations. It should be noted that, in accordance with NS1995-1-1, §7.3, the dead load was used to compute the mass of the deck. A more conservative assumption would have been to employ the quasi-permanent load, which would have resulted in a shorter span length. The calculations can be found in Appendix C1. According to the results, the longest span-length achievable with this slab-system is approximately 8,5 meters.

Modelling in *Robot Structural*

A suitable numerical model of the timber composite slab is a comprehensive process that necessitates a large amount of data in order to be integrated in a full-scale model of the building. Therefore, the slab is simplified by modeling it with shell elements. The slab is modelled as an orthotropic shell element in *Robot Structural*, with the domain meshed into elements using Coons' approach. Using the *Coons' method*, all points formed on the selected contour edge are connected with points formed on the opposite edge of the contour [41]. The type of finite elements used for the shells are the 4-node Quadrilaterals (Q4), which is chosen as default by *Robot.* Because Q4 elements are prone to shear locking, they are not optimal for shells with transverse loading (bending state). However, because the FE mesh in *Robot* has a low aspect ratio (the ratio between the width and length of the elements), it does not lock. It is, however, overly stiff in bending. This is presumed to be acceptable.

Regarding the material properties, they are fictitiously adapted to obtain similar properties to the slab depicted in Figure 4.5. This was accomplished with *Robot*'s "material orthotropy"-option. Table 4.7 displays the stiffness values that were implemented. The stiffness in the vertical direction (E_3) is not included because the material is assumed transversely isotropic, meaning $E_3 = E_2$.

The stiffness of the slab in both the strong and weak directions, as well as the slab's dead weight, are the most essential properties, as shown in Table 4.7. The analytical bending stiffness of the slab in longitudinal direction is calculated using Annex B in NS1995-1-1, which uses the γ-method assuming full interaction between the components [8]. Both the inner- and edge beams are accounted for (see Appendix C1). The bending stiffness in transverse direction has not been calculated, but the assumption $E_2 = 1/4 \cdot E_1$ has been used for the transversal stiffness. The mass of the slab is extracted from the master thesis by Bjørge and Kristoffersen [26], because the exact same slab has been implemented in this thesis.

Table 4.7: Material properties of slab

EI_L	E_1	E_2	m
1,62 $\cdot 10^{14} Nmm^2$	10960 Nmm^2	2740 Nmm^2	200 kg/m^2

To assure that the analytical properties of the slab are representative for the physical properties of the slab, the longitudinal bending stiffness has been compared to the experimental value obtained from the thesis by Bjørge and Kristoffersen:

Table 4.8: Comparison between numerical and experimental value of bending stiffness

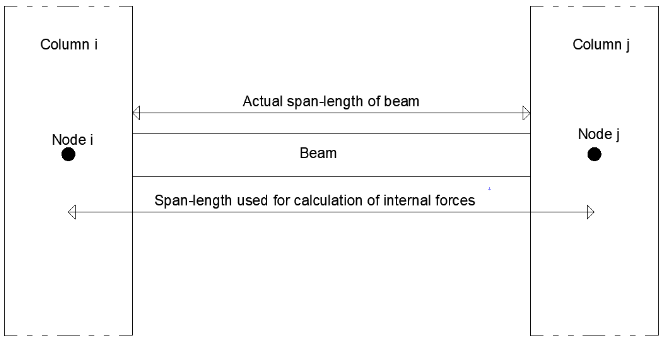
EI_L analytical	EI_L experimental [26]	Deviation
1,62 $\cdot 10^{14} Nmm^2$	1,31 $\cdot 10^{14} Nmm^2$	**23,7 %**

This means that the analytical method overestimates the bending stiffness by 23,7%, if the experimental values are correct.

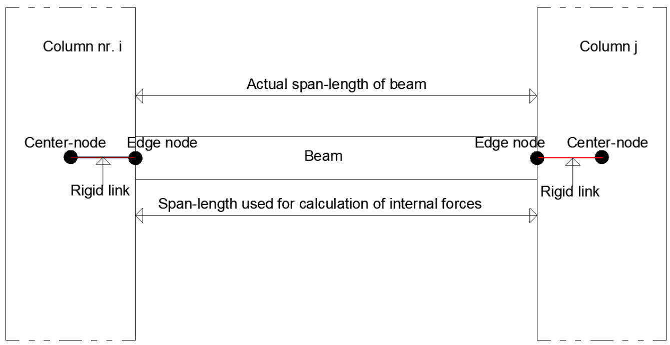
4.4.2 Beams, columns and diagonals

Beams, columns and diagonals are modelled as standard 2-noded beam elements. *Robot* then examines the elements as straight lines between two nodes (i and j). The elements are rectangular shaped and contains the material properties corresponding to *GL30c* in Table 2.1. The columns are modelled as continuous along the height of the buildings, while the diagonals are interrupted by the columns. The beams are interrupted by both the diagonals and the columns.

To achieve more accurate internal forces, an offset from the center to the edge of the columns was required. *Robot* computes internal forces based on center-lines. This means, for example, that the length of a beam between two columns is measured from center to center of the columns. In actuality, the members' lengths are reduced due to the discontinuity at the columns' edges. To address this, nodes were added to the columns' edges where diagonals and beams might link. Then, a rigid link was formed between the columns' outer node and center node, providing rigid compatibility conditions with regard to all displacements in these nodes, as illustrated in Figure 4.7. All nodes linked with a rigid link constitute a group of nodes comparable to a rigid body [38]. In this way, the internal forces are calculated by the actual lengths of the members simultaneously as the forces are transferred by rigid elements to the center of the columns.



(a) Modelling without rigid link



(b) Modelling with rigid link

Figure 4.7: Demonstration on how rigid links makes for a more accurate modelling

4.4.3 Connections

The connections were created in *Robot* using two different release choices. The "Releases"-option releases the nodes, while the "Linear releases"-option releases the slab elements. It is assumed in *Robot* that all bars are connected by fixed connections in their nodes. In the release options, you may specify which degrees of freedom to release as well as values for translational and rotational stiffness.

The diagonals are assumed to have pinned connections with no rotational stiffness. The connections are characterized by the occurrence of a slip between the connected pieces. As a result, the slip modulus has been determined in accordance with Eurocode 5, §7.1. The slip modulus is included because the deformation of the connection affects the overall displacements of the structure. Because the diagonals represent the structure's bracing, this might have a significant impact.

For the beams, a pinned connection has been implemented at both nodes, by releasing the rotational degrees of freedom. The assumption of perfectly pinned connections for the beams is justified by the chosen bracing system where the diagonals provide the lateral stiffness of the building, while the beams have negligible effect on it. The slip in the beam-connections is therefore neglected.

The slabs are modelled as pinned using linear releases at the edges, which is possible for shell elements in *Robot*. The consequence of such modelling of the slabs, is that the results for the finite elements are given for the calculation points. The calculation points are the nodes generated using the so-called DSC algorithm, which is the basis for calculating structures with linear releases in *Robot* [40].

4.4.4 Base Supports

The base supports are defined as pinned. A more accurate numerical model could have been achieved by defining elastic supports. As a simplification and to save some time, the calculation of the elastic stiffness in the foundations were not done, and the pinned assumption were used.

4.5 Models

In this section a brief presentation of the main models will be given. All the models contain some restrictions to make the analyzing process more clear. The restrictions are:

Table 4.9: Constant parameters for main models

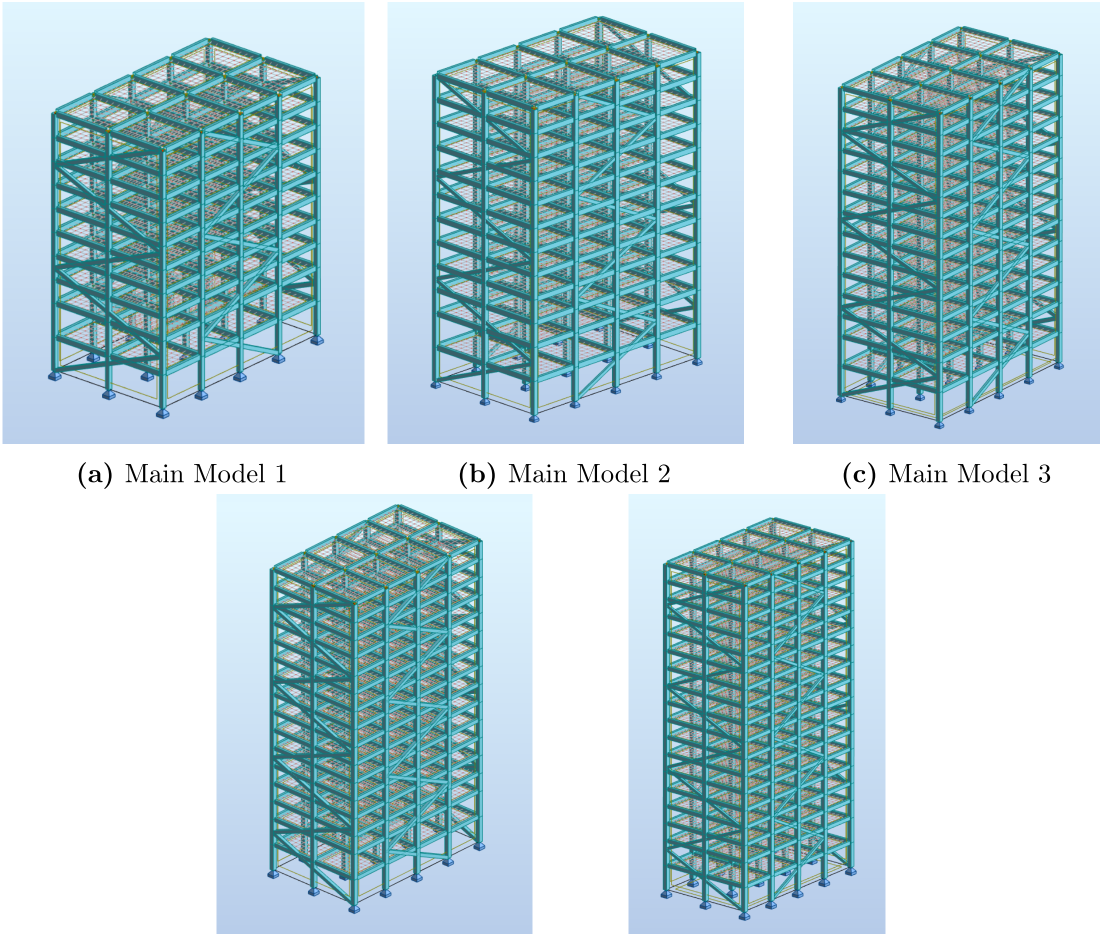
<table>
<tr><td colspan="2">Footprint area</td><td>19,2x32m</td></tr>
<tr><td colspan="2">Base support</td><td>Pinned</td></tr>
<tr><td rowspan="2">Floor height</td><td>Commercial</td><td>5m</td></tr>
<tr><td>Residece/office</td><td>3,5m</td></tr>
<tr><td colspan="2">Slab system</td><td>Composite deck</td></tr>
<tr><td rowspan="2">No. of bays</td><td>x-direction</td><td>4</td></tr>
<tr><td>y-direction</td><td>2</td></tr>
</table>

These restrictions are the same for all of the main models, and will be further explained in Section 5.2. What defines the main models are the number of storeys which in practice means the height of the buildings. By changing the height between the main models, it is possible to analyze how tall the building can become. At the same time, the main models are used as references to learn what affects the building the most with respect to a parametric study, which will be further explained in Section 5.2. They are modelled without shear walls and without a shaft contributing to stability. This is based on the goal of robustness, and to investigate if the requirements can be reached without being dependent on shear walls and a shaft.

The five main models, with their properties and illustrations, will now be presented in Table 4.10 and Figure 4.8.

Table 4.10: Properties for main models

Property		Values
Nr of floors	Main Model 1	10
	Main Model 2	12
	Main Model 3	14
	Main Model 4	16
	Main Model 5	18
Total height	Main Model 1	38m
	Main Model 2	45m
	Main Model 3	52m
	Main Model 4	59m
	Main Model 5	66m
Beam cross-sections	Main Model 1	450mm x 765mm
	Main Model 2	450mm x 765mm
	Main Model 3	450mm x 765mm
	Main Model 4	450mm x 765mm
	Main Model 5	495mm x 765mm
Diagonal cross-sections	Main Model 1	315mm x 495mm
	Main Model 2	360mm x 540mm
	Main Model 3	405mm x 585mm
	Main Model 4	450mm x 540mm
	Main Model 5	495mm x 585mm
Column cross-sections	Main Model 1	630mm x 630mm
	Main Model 2	675mm x 675mm
	Main Model 3	720mm x 720mm
	Main Model 4	810mm x 810mm
	Main Model 5	855mm x 855mm
Slip modulus of diagonal-connections	Main Model 1	512 000 N/mm
	Main Model 2	950 000 N/mm
	Main Model 3	590 000 N/mm
	Main Model 4	1 300 000 N/mm
	Main Model 5	1 600 000 N/mm
Bracing-system	All main models	Diagonal Braced
Number of diagonals (Both directions)	Main Model 1	5
	Main Model 2	6
	Main Model 3	7
	Main Model 4	8
	Main Model 5	8
Category of floors	All main models	1st and 2nd: Commercial Remaining: Residential
Material properties	All main models	Gl30c and slab system



(a) Main Model 1 **(b)** Main Model 2 **(c)** Main Model 3

(d) Main Model 4 **(e)** Main Model 5

Figure 4.8: Design for the main models

As Table 4.10 shows, the main models have equal beams, columns and diagonals cross-sections within each model. This is for simplification and to make the analysing process more stable. The main models have been designed such that all requirements are fulfilled, but they are not optimized designs. Finished solutions will be presented in Section 6.2.

Chapter 5

Analysis

The analysis procedure, parametric study and weight requirements are all covered in this chapter.

5.1 Approach

The main approach in the analysis is based on the following steps:

1. To begin, main models are simplified by keeping the cross-sections of each element constant. All design pertinent to this assignment are conducted on these models, ensuring they are usable designs.

2. Then, on each main model, a parametric analysis is run to see how the different parameters effect the accelerations (see Section 6.1).

3. Combinations of different modifications are made in parametric study. All design checks are performed on the combinations. These combinations can have a number of reasons, but the main goal is to see how different factors combined effect acceleration and remained withing design.

4. Finally, a portfolio of solutions is created based on findings of the parametric analysis. The solutions are more optimized, meaning cross sections of beams, columns and diagonals are not kept constant over entire structure. All design requirements (including the weight requirement (Section 5.4) are examined.

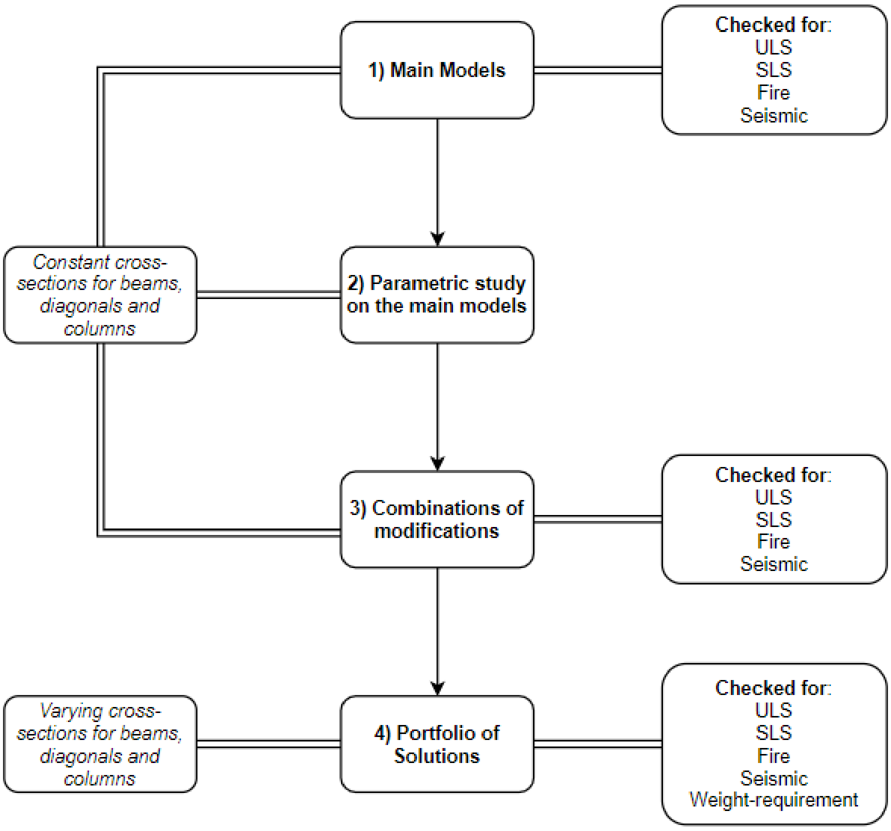


Figure 5.1: Overall analysis process

5.2 Parametric Study

Geometrical parameters were visualised in *Dynamo* with help of programming in *Python*. Parameters are listed:

- Footprint area
- Number of floors
- Height of floors (Residential/office and commercial)
- Number of bays in both directions
- Cross sections
- Stiffness in connections
- Support conditions

Some variables are kept constant in our models as shown in Table 4.9.

In addition to what presented, the first two levels have a constant commercial load. Number of diagonals are also consistent over two bays (in both direction) in each model. The number of floors, dimensions, bracing method, extra mass (concrete or green roof) and connection stiffness were all altered in the parametric study.

It is vital to note that a parametric research is carried out for each of the main models. These main models are already presented in Table 4.10.

5.2.1 Modifications in Parametric Study

Modifications are said to concentrate on acceleration results. For slender tall structures, accelerations are likely to be governing. On the other hand, some combinations are checked for ULS, SLS, seismic and fire design (critical elements). Each model contains a total of 15 parameters, and some of them will be combined. This section describes the modifications.

Cross Sections

We keep the columns (subdivided into corner- and inner columns), beams and diagonals constant throughout the entire structure in the parametric study.

Columns and beams are increased once in modifications, while diagonals are both increased and decreased once.

Stiffness

Appendix C2 has been used to determine the connection stiffness (translational) for each base model. These connection stiffness are only inserted in connections between diagonals and columns. Connection stiffness is increased and lowered once in the study. In addition, cross-braced system is introduced as a parameter to affect the lateral stiffness of the structure (see Figure 2.4b).

Adding Mass in Structure

The insertion of mass into the structure is a parameter that has been investigated. As described in Sections 2.2.2 and 2.2.1, adding concrete is beneficial. In addition, green roof is investigated as of interest. This roof is evaluated in both dry and wet circumstances.

Live Load Category

Floors above commercial (first two levels) are initially in each model set for residential use. In our study, we changed either all floors or some floors to offices.

One can state the advantageous of office use in higher floors due to the less strict acceleration requirement given in Figure 3.5 for offices.

5.3 Portfolio of Solutions

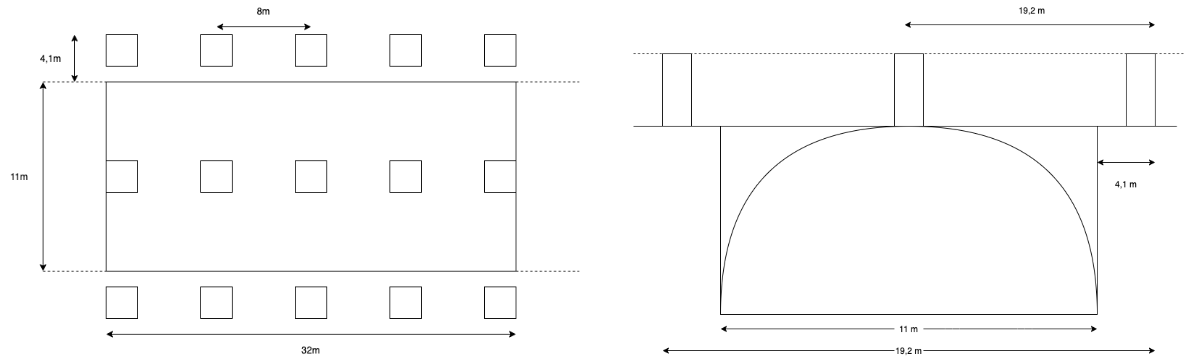
A set of solutions is to be introduced at the end of the parametric study. These solutions are meant to be how one can design a building in *Økern Centre* that meet all requirements. Solutions are designed based on the parametric study of main models. There will be more non-constant parameters over the entire structure listed:

- number and cross section of diagonals in both x- and y-direction
- stiffness of connection in diagonals in longitudinal and transversal direction
- corner and inner columns under and over commercial floors
- beams under floors with added mass, commercial, residence and offices

The portfolio's result will be presented in Section 6.2. All results are reported (properties and design criteria). In addition, the weight requirement is needed to be satisfied for ULS fundamental combinations. The weight requirement is only checked for the solutions in the portfolio.

5.4 Satisfaction of Weight Requirements

Data for tunnel capacity is given in Appendix D. Maximum line load is given as 1370kN/m with 11 meter load width. The total building force and column force is checked for the weight requirement. Only inner mid line columns are standing on the tunnel as Figure 5.2 illustrates.



(a) Plan view over tunnel

(b) Side view of tunnel

Figure 5.2: Columns on top of tunnel

The structure is oriented as portrayed due to maximum protrusion to the side of tunnel is 10 meters. The columns outside the tunnel need to be connected to the foundation of the tunnel on a further study. Maximum force on each column is determined by distributing the maximum load to each column. Total building force is needed to be under total pressure force from tunnel. Note that area outside the tunnel is assumed to have same capacity for simplifications. Calculations of maximum pressure and force is done in Equation 5.1.

$$maximum\,column\,force = \frac{line\,load \cdot width}{number\,of\,columns} \tag{5.1a}$$

$$maximum\,building\,force = \frac{line\,load}{load\,width} \cdot footprint\,area \tag{5.1b}$$

Which results with numbers as,

$$maximum\,column\,force = \frac{1370kN/m \cdot 32m}{5} = 8768kN$$

$$maximum\,building\,force = \frac{1370kN/m}{11m} \cdot (19,2 \cdot 32)m^2 = 76520kN$$

Maximum building force can also be converted to weight on approximately 7 652 070 kg.

Chapter 6

Results

This chapter will go over all of the results from the parametric study as well as the solution portfolio. Some remarks and highlights of results will be done.

6.1 Parametric Study

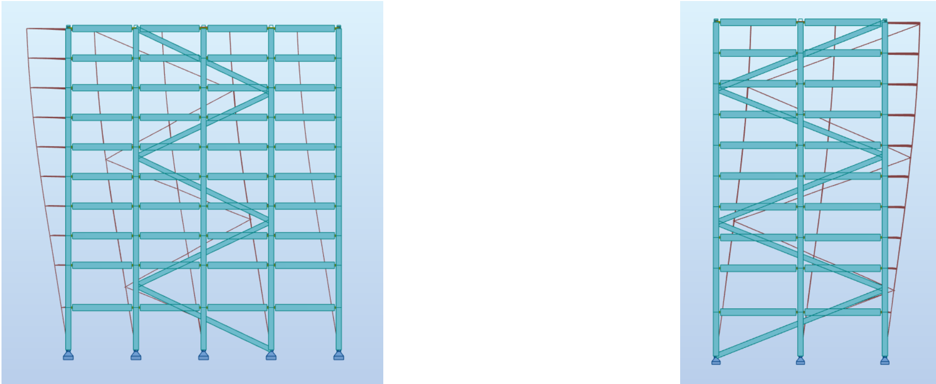
As previously stated in Section 5.2, a parametric investigation was conducted. All parameters are presented in tables together with combinations and design checks. Results of main models are listed in order: accelerations, ULS, structural fire, displacements and seismic design.

6.1.1 Main Model 1

This structure is in total height of 38 meters with 10 floors. The model is already presented in Table 4.10.

Acceleration

Figure 6.1 depicts mode shapes, whereas Table 6.1 express accelerations. Effects of more stiffness or mass are the most prominent.



(a) Mode shape 1 **(b)** Mode shape 2

Figure 6.1: The first two modes of main model 1

Table 6.1: Acceleration, Main Model 1

Modification Parameter	**Mass [kg]**			**Natural Frequency [Hz]**		**Acceleration [m/s^2]**			
	Total	m_{ex}	m_{ey}	$f_{1,x}$	$f_{1,y}$	a_x	$\Delta a_x/a_x$	a_y	$\Delta a_y/a_y$
Main Model 1	1987528	53565	53602	0,70	0,70	0,023	-	0,040	-
1. Corner Columns: 675x855mm	1998212	55843	53903	0,70	0,71	0,022	-0,04	0,039	-0,03
2. Inner Columns: 675x855mm	2016905	54093	54108	0,71	0,70	0,022	-0,04	0,039	-0,03
3. Beams: 495x990mm	2084194	56304	56339	0,69	0,69	0,022	-0,04	0,039	-0,03
4. Diagonals: 450x810mm	2018734	56268	56319	0,81	0,83	0,018	-0,22	0,032	-0,20
5. Diagonals: 360x405mm	1986013	53527	53563	0,61	0,6	0,023	0	0,041	0,03
6. Connection Stiffness: 810000 N/mm	1987528	53539	53584	0,76	0,76	0,021	-0,09	0,036	-0,10
7. Connection Stiffness: 410000 N/mm	1987528	53581	53613	0,67	0,67	0,023	0	0,042	0,05
8. Bracing System: Cross Braced	2010857	54082	54140	0,92	0,94	0,017	-0,26	0,028	-0,30
9. Casted Concrete on Top 2 Floors and Roof	2457413	78287	78112	0,58	0,58	0,020	-0,13	0,035	-0,13
10. Casted Concrete on Top 5 Floors and Roof	2755402	83200	83115	0,68	0,68	0,018	-0,22	0,032	-0,20
11. Casted Concrete on all Floors and Roof	3553812	97934	97963	0,52	0,52	0,017	-0,26	0,032	-0,20
12. Wet Green Roof	2404160	78436	77981	0,59	0,59	0,020	-0,13	0,035	-0,13
13. Dry Green Roof	2338376	74278	73902	0,60	0,60	0,020	-0,13	0,036	-0,10
14. Office Floors	2137891	57851	57907	0,68	0,68	0,021	-0,09	0,038	-0,05
15. Offices on Top 4 Floors	1912347	56361	54624	0,70	0,70	0,022	-0,04	0,039	-0,03
Combinations of Modifications									
Combination 1: 5+7	1986013	53540	53572	0,66	0,66	0,024	0,04	0,043	0,08

Ultimate Limit State

Table 6.2 lists the internal forces under the ULS combinations together with utilities of critical element. Combinations are presented in accelerations results.

Table 6.2: ULS, Main Model 1

Combination	**Critical Element**	**Moment[kNm]**		**Shear Force [kN]**		**Axial Force [kN]**	**Load Combination**	**k$_{mod}$**	**Utility**
		M_y	M_z	V_y	V_z	N			
Main Model 1	Diagonal in y-direction	-	-	-	-	960	ULS Combination 1	1,1	0,83
Combination 1	Diagonal in y-direction	-	-	-	-	955	ULS Combination 1	1,1	0,70

Structural Fire Design

The reduced forces and cross sections are reported in Table 6.3. Diagonals are not satisfied for combination 1.

Table 6.3: Structural Fire, Main Model 1

Combination	Critical Element	Reduced Cross Section [mm]	Reduced Force					Utility
			$M_y[kNm]$	$M_z[kNm]$	$V_y[kN]$	$V_z[kN]$	N [kN]	
Main Model 1	Diagonal in y-direction	359x250	-	-	-	-	575	0,77
Combination 1	Diagonal in y-direction	274x295	-	-	-	-	572	1,00

Displacements

Lateral Displacements

The Table 6.4 shows lateral displacements. Maximum permissible lateral movement (u_{max}) for this model is 76 millimeters.

Table 6.4: Lateral displacements, Main Model 1

Variations	Deformation [mm]			
	u_x	u_x/u_{max}	u_y	u_y/u_{max}
Main Model 1	22,3	0,29	36,2	0,48
Combination 1	25,1	0,33	40,8	0,54

Interstorey Drift

This design comes with two floor heights: 3,5 meters and 5 meters. The allowed drift for these levels is 11,67 millimeters and 16,67 millimeters, respectively. See results in Table 6.5.

Table 6.5: Interstorey drift, Main Model 1

Combination	Displacements [mm]					
	x-direction			*y-direction*		
	Storey number (Height of story)	*Drift*	*Utility*	*Storey*	*Drift*	*Utility*
Main Model 1	2 (5m)	5,8	0,35	3 (3,5m)	8,0	0,68
Combination 1	2 (5m)	6,5	0,39	3 (3,5m)	8,8	0,75

Seismic Design

Only crucial direction is considered in seismic design. Table 6.6 lists mass, natural period, base shear and critical bar utility.

Table 6.6: Seismic, Main Model 1

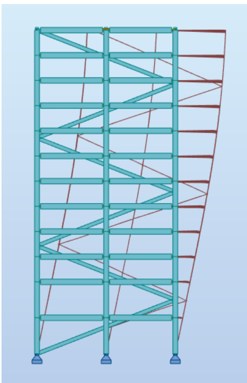
Combination	Modal Mass [kg]	Period [s]	Base Shear [kN]	Critical Bar	Utility
Main Model 1	1987528	1,42	236	Inner Column	0,33
Combination 1	1986013	1,51	222	Inner Column	0,33

6.1.2 Main Model 2

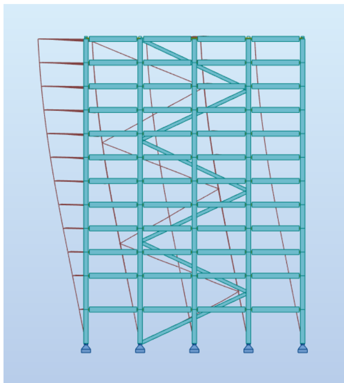
Two more storeys are added to next model. This structure's overall height has been expanded from 38 to 45 meters.

Acceleration

The first two mode shapes are seen in Figure 6.2. Table 6.7 depicts accelerations for main model 2.



(a) Mode shape 1



(b) Mode shape 2

Figure 6.2: The first two modes of main model 2

Table 6.7: Acceleration, Main Model 2

Modification Parameter	**Mass [kg]**			**Natural Frequency [Hz]**		**Acceleration [m/s^2]**			
	Total	m_{ex}	m_{ex}	$f_{1,x}$	$f_{1,y}$	a_x	$\Delta a_x/a_x$	a_y	$\Delta a_y/a_y$
Main Model 2	2402176	54436	54484	0,68	0,68	0,024	0	0,042	0
1. Corner Columns: 675x900mm	2412838	54670	54754	0,68	0,70	0,024	0	0,040	- 0,05
2. Inner Columns: 675x855mm	2425632	54986	55003	0,68	0,70	0,023	-0,04	0,040	- 0,05
3. Beams: 495x990mm	2482731	56347	56395	0,67	0,67	0,023	-0,04	0,041	-0,02
4. Diagonals: 450x855mm	2436280	55131	55191	0,77	0,80	0,021	-0,13	0,035	- 0,17
5. Diagonals: 360x405mm	2393469	54310	54269	0,63	0,63	0,025	0,04	0,046	0,10
6. Connection Stiffness: 1500000 N/mm	2402176	54406	54461	0,72	0,72	0,022	-0,08	0,039	- 0,07
7. Connection Stiffness: 750000 N/mm	2402176	54454	54497	0,66	0,66	0,024	0	0,043	0,02
8. Bracing System: Cross Braced	2437006	55063	55146	0,86	0,89	0,019	-0,21	0,031	- 0,26
9. Casted Concrete on Top 2 Floors and Roof	2872061	76818	76352	0,57	0,58	0,021	-0,13	0,036	- 0,14
10. Casted Concrete on Top 4 Floors and Roof	3185318	87320	86698	0,54	0,54	0,020	-0,17	0,035	- 0,17
11. Casted Concrete on Top 8 Floors and Roof	3811832	97613	97535	0,51	0,51	0,019	-0,21	0,033	- 0,21
12. Wet Green Roof	2809410	76205	75611	0,58	0,58	0,021	-0,13	0,038	- 0,10
13. Dry Green Roof	2756156	73205	72688	0,59	0,60	0,022	-0,08	0,038	- 0,10
14. Office Floors	2590130	58849	58926	0,66	0,66	0,023	-0,04	0,04	- 0,05
15. Offices on Top 4 Floors	2477358	57326	57381	0,66	0,67	0,023	- 0,04	0,04	- 0,05

Combinations of modifications									
Combination 1: 5+7	2393469	54283	54320	0,62	0,61	0,026	0,08	0,047	0,12
Combination 2: 5+13*	2744316	72515	72070	0,55	0,55	0,023	-0,042	0,042	0

*Combination 2 for ULS, lateral displacements, interstorey drift , structural fire and seismic design is given as 5+12.

Ultimate Limit State

Table 6.8 shows results of utilization and internal forces of critical elements.

Table 6.8: ULS, Main Model 2

Combination	**Critical Element**	**Moment[kNm]**		**Shear Force [kN]**		**Axial Force [kN]**	**Load Combination**	**k_{mod}**	**Utility**
		M_y	M_z	V_y	V_z	N			
Main Model 2	Inner Column	-	-	-	-	5685	ULS Combination 5	0,8	0,78
Combination 1	Diagonal in y-direction	-	-	-	-	1125	ULS Combination 1	1,1	0,86
Combination 2	Beam	890	-	-	56	-	ULS Combination 5	0,8	0,97

Structural Fire Design

Table 6.9 displays the results for structural fire design. Diagonal is most utilized with 73%.

Table 6.9: Structural Fire, Main Model 2

Combination	**Critical Element**	**Reduced Cross Section [mm]**	**Reduced Force**					**Utility**
			$M_y[kNm]$	$M_z[kNm]$	$V_y[kN]$	$V_z[kN]$	N *[kN]*	
Main Model 2	Inner Column	535x535	-	-	-	-	3410	0,44
Combination 1	Diagonal in y-direction	274x295	-	-	-	-	675	0,73
Combination 2	Beam	450x700	534	-	-	34	-	0,43

Displacements

Lateral Displacement

Lateral displacements are presented in Table 6.10. The maximum allowable lateral displacement for this model is equal to 90mm (u_{max}).

Table 6.10: Lateral displacements, Main Model 2

Variations	Deformation [mm]			
	u_x	u_x/u_{max}	u_y	u_y/u_{max}
Main Model 2	24,5	0,27	40,4	0,45
Combination 1	29,9	0,33	50,3	0,56
Combination 2	28,2	0,31	47,5	0,53

Interstorey Drift

Maximum allowable interstorey drift is 11,67 millimeters (floors above commercial) and 16,67 millimeters (commercial floors).

Table 6.11: Interstorey drift, Main Model 2

Combination	Displacements [mm]					
	x-direction			*y-direction*		
	Storey number (Height of story)	*Drift*	*Utility*	*Storey*	*Drift*	*Utility*
Main Model 2	3 (3,5m)	4,0	0,34	2 (5m)	8,0	0,48
Combination 1	3 (3,5m)	5,0	0,43	2 (5m)	10,0	0,60
Combination 2	3 (3,5m)	4,7	0,40	2 (5m)	9,5	0,57

Seismic Design

Seismic results are portrayed in Table 6.12. The utility of combination 2 is 68 percent.

Table 6.12: Seismic, Main Model 2

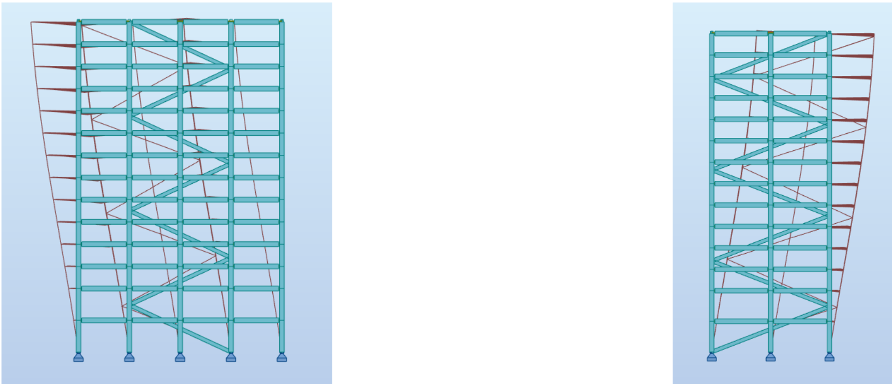
Combination	Modal Mass [kg]	Period [s]	Base Shear [kN]	Critical Bar	Utility
Main Model 2	2402176	1,47	280	Inner Column	0,34
Combination 1	2393469	1,63	250	Inner Column	0,34
Combination 2	2810100	1,86	239	Beam	0,68

6.1.3 Main Model 3

Main model 3 is increased once more with two floors, resulting with total height of 52m and 14 floors.

Acceleration

Table 6.13 present the data of main model 3 acceleration study. The first two modes are demonstrated in Figure 6.3.



(a) Mode shape 1 **(b)** Mode shape 2

Figure 6.3: The first two modes of main model 3

Table 6.13: Acceleration, Main Model 3

Modification Parameter	**Mass [kg]**			**Natural Frequency [Hz]**		**Acceleration [m/s^2]**			
	Total	m_{ex}	m_{ey}	$f_{1,x}$	$f_{1,y}$	a_x	$\Delta a_x/a_x$	a_y	$\Delta a_y/a_y$
Main Model 3	2825986	55241	55295	0,61	0,62	0,027	-	0,046	-
1. Corner Column: 720x900mm	2836499	55454	55529	0,63	0,63	0,026	-0,04	0,045	-0,02
2. Inner Columns: 720x855mm	2847669	55683	55711	0,62	0,62	0,026	-0,04	0,046	0
3. Beams: 495x990mm	296138	57976	58051	0,58	0,60	0,027	0	0,045	-0,02
4. Diagonals: 495x855mm	2864870	55951	55993	0,70	0,69	0,023	-0,15	0,041	-0,11
5. Diagonals: 360x540mm	2817110	55163	55141	0,66	0,59	0,024	-0,11	0,050	0,09
6. Connection Stiffness: 1500000N/mm	2825986	55208	55268	0,65	0,65	0,025	-0,07	0,044	-0,04
7. Connection Stiffness: 850000N/mm	2825986	55256	55306	0,60	0,60	0,027	0	0,047	0,02
8. Bracing System: Cross Braced	2875437	56023	56112	0,75	0,79	0,022	-0,19	0,035	-0,24
9. Casted Concrete on Top 2 Floors and Roof	3295871	75559	75044	0,53	0,53	0,023	-0,15	0,042	-0,09
10. Casted Concrete on Top 4 Floors and Roof	3609128	85610	86106	0,49	0,50	0,022	-0,19	0,039	-0,15
11. Casted Concrete on Top 8 Floors and Roof	4235641	96799	96603	0,46	0,47	0,021	-0,22	0,037	-0,20
12. Wet Green Roof	3233220	74783	74095	0,53	0,54	0,024	-0,11	0,042	-0,09
13. Dry Green Roof	3179966	72100	72509	0,54	0,55	0,024	-0,11	0,042	-0,09
14. Office Floors	3051531	59755	59840	0,59	0,59	0,026	-0,04	0,045	-0,02
15. Offices on Top 5 Floors	2901167	57994	57961	0,60	0,60	0,026	-0,04	0,046	0

Combinations of Modifications									
Combination 1: 5+7	2817110	55058	55151	0,53	0,58	0,032	0,19	0,049	0,07
Combination 2: 5+9	3286995	75535	74701	0,46	0,50	0,028	0,04	0,045	-0,02

Ultimate Limit State

The results of the ULS design for important members are manifested in Table 6.14. Diagonals are on the most 95% utilized.

Table 6.14: ULS, Main Model 3

Combination	Critical Element	Moment[kNm]		Shear Force [kN]		Axial Force [kN]	Load Combination	k_{mod}	Utility
		M_y	M_z	V_y	V_z	N			
Main Model 3	Inner Column	-	-	-	-	6590	ULS Combination 5	0,8	0,79
Combination 1	Diagonal in y-direction	-	-	-	-	1460	ULS Combination 1	1,1	0,94
Combination 2	Diagonal in y-direction	-	-	-	-	1465	ULS Combination 1	1,1	0,95

Structural Fire Design

The fire design results from main model 3 is reported in Table 6.15. Utilities are similar as previous models.

Table 6.15: Structural Fire, Main Model 3

Combination	Critical Element	Reduced Cross Section [mm]	Reduced Force					Utility
			$M_y[kNm]$	$M_z[kNm]$	$V_y[kN]$	$V_z[kN]$	N [kN]	
Main Model 3	Inner Column	580x580	-	-	-	-	3950	0,43
Combination 1	Diagonal in y-direction	295x409	-	-	-	-	876	0,63
Combination 2	Diagonal in y-direction	295x409	-	-	-	-	880	0,64

Displacements

Lateral Displacement

In Table 6.16, lateral displacements are presented. For this model the maximum admissible displacement is equal to 104 millimeters.

Table 6.16: Lateral displacements, Main Model 3

Variations	Deformation [mm]			
	u_x	u_x/u_{max}	u_y	u_y/u_{max}
Main Model 3	29,3	0,27	52,2	0,50
Combination 1	44,4	0,43	60,1	0,58
Combination 2	42,4	0,41	57,5	0,55

Interstorey Drift

This model as previous ones consists of floor heights: 3,5 meters and 5 meters. Max permitted interstorey drift is 11,67 millimeters and 16,67 millimeters. Results are depicted in 6.17.

Table 6.17: Interstorey drift, Main Model 3

Combination	Displacements [mm]					
	x-direction			*y-direction*		
	Storey number (Height of story)	*Drift*	*Utility*	*Storey*	*Drift*	*Utility*
Main Model 3	2(5m)	5,0	0,30	3(3,5m)	7,7	0,66
Combination 1	2(5m)	6,6	0,40	3(3,5m)	9,0	0,77
Combination 2	2(5m)	6,3	0,38	3(3,5m)	8,6	0,74

Seismic Design

Table 6.18 provides seismic design results. All results are close to each other in terms of utilities.

Table 6.18: Seismic, Main Model 3

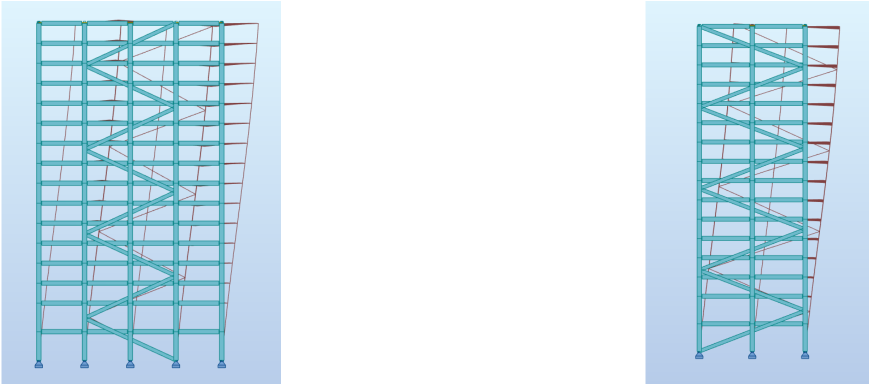
Combination	Modal Mass [kg]	Period [s]	Base Shear [kN]	Critical Bar	Utility
Main Model 3	2825986	1,65	290	Inner Column	0,35
Combination 1	2817110	1,89	240	Diagonal	0,38
Combination 2	3286995	2,15	230	Inner Column	0,41

6.1.4 Main Model 4

We reach a total of 59 meters in height after two more storeys. This is closer to where the structures of wood are limited.

Acceleration

The parameters in Table 6.19 shows extra mass and stiffness reduces motions on top. Acceleration is reduced by 22% by modify to cross braced system (parameter number 8). Mode shapes (x and y) are illustrated in Figure 6.4.



(a) Mode shape 1 **(b)** Mode shape 2

Figure 6.4: The first two modes of main model 4

Table 6.19: Acceleration, Main Model 4

Modification Parameter	Mass [kg]			Natural Frequency [Hz]		Acceleration [m/s^2]			
	Total	m_{ex}	m_{ey}	$f_{1.x}$	$f_{1.y}$	a_x	$\Delta a_x/a_x$	a_y	$\Delta a_y/a_y$
Main Model 4	3274627	56155	56195	0,55	0,56	0,029	0	0,051	0
1. Corner Columns: 900x990mm	3334595	56632	56734	0,55	0,58	0,029	0	0,048	-0,06
2. Inner Columns: 945x945mm	3334595	57334	57339	0,56	0,57	0,028	-0,03	0,049	-0,04
3. Beams: 450x990mm	3429292	59025	59085	0,54	0,55	0,029	0	0,049	-0,04
4. Diagonals: 540x630mm	3452452	59377	594444	0,50	0,51	0,027	-0,07	0,046	-0,10
5. Diagonals: 405x405mm	3255809	55995	56045	0,50	0,51	0,032	0,10	0,056	0,10
6. Connection Stiffness: 2000000N/mm	3274627	56239	56306	0,59	0,59	0,028	-0,03	0,048	-0,06
7. Connection Stiffness: 1000000N/mm	3274627	56288	56345	0,54	0,54	0,030	0,03	0,052	0,02
8. Bracing System: Cross Braced	3332529	57089	57175	0,68	0,71	0,024	-0,17	0,040	-0,22
9. Casted Concrete on Top 2 Floors and Roof	3744512	74631	74125	0,47	0,48	0,026	-0,10	0,046	-0,10
10. Casted Concrete on Top 8 Floors and Roof	4684283	95952	95630	0,41	0,42	0,024	-0,17	0,042	-0,18
11. Casted Concrete on all Floors and Roof	5780682	100956	101001	0,4	0,41	0,023	-0,21	0,040	-0,22
12. Wet Green Roof	3691259	74147	73514	0,47	0,48	0,027	-0,07	0,048	-0,06
13. Dry Green Roof	3625475	71198	70663	0,48	0,49	0,028	-0,03	0,048	-0,06
14. Office Floors	3537763	60895	60982	0,51	0,52	0,029	0	0,051	0
15. Offices on Top 4 Floors	3349809	58781	58789	0,52	0,53	0,030	0,03	0,052	0,02

Combinations of modifications									
Combination 1: 4+6	3297788	56581	56654	0,62	0,64	0,026	-0,10	0,044	-0,14
Combination 2: 5+9	3725694	74069	73576	0,44	0,44	0,029	0	0,052	0,02
Combination 3: 5+12/13	3606657	70629	7+138	0,45	0,45	0,030	0,03	0,053	0,04

Ultimate Limit State

The ULS design result for main model 4 is given in Table 6.20. Beams are utilized 95% in combination 3.

Table 6.20: ULS, Main Model 4

Combination	Critical Element	Moment[kNm]		Shear Force [kN]		Axial Force [kN]	Load Combination	k_{mod}	Utility
		M_y	M_z	V_y	V_z	N			
Main Model 4	Beam	666	-	-	42	-	ULS Combination 5	0,8	0,72
Combination 1	Beam	666	-	-	42	-	ULS Combination 5	0,8	0,72
Combination 2	Diagonal in y-direction	-	-	-	-	1672	ULS Combination 1	1,1	0,92
Combination 3	Beam	892	-	-	92	-	ULS Combination 5	0,8	0,95

Structural Fire Design

The results for the fire design is shown in Table 6.21. Diagonals in combinations 2 and 3 are not satisfied.

Table 6.21: Structural Fire, Main Model 4

Combination	Critical Element	Reduced Cross Section [mm]	Reduced Force					Utility
			$M_y[kNm]$	$M_z[kNm]$	$V_y[kN]$	$V_z[kN]$	N [kN]	
Main Model 4	Inner Column	679x679	-	-	-	-	4484	0,35
Combination 1	Inner Column	679x679	-	-	-	-	4484	0,36
Combination 2	Diagonal in y-direction	274x340	-	-	-	-	1007	1,2
Combination 3	Diagonal in y-direction	274x340	-	-	-	-	1007	1,2

Displacements

Lateral Displacement

Lateral displacements are presented in Table 6.22. The maximum allowable lateral displacement for this model is equal to 118mm.

Table 6.22: Lateral displacements, Main Model 4

Variations	Deformation [mm]			
	u_x	u_x/u_{max}	u_y	u_y/u_{max}
Main Model 4	41,4	0,35	66,1	0,56
Combination 1	33,3	0,28	51,6	0,44
Combination 2	49,7	0,42	81,9	0,69
Combination 3	49,7	0,42	81,8	0,69

Interstorey Drift

Permissible interstorey drift for floors are respectively 11,67mm and 16,67mm. See values listed in Table 6.23.

Table 6.23: Interstorey drift, Main Model 4

Combination	Displacements					
	x-direction			*y-direction*		
	Storey number (Height of story)	*Drift*	*Utility*	*Storey*	*Drift*	*Utility*
Main Model 4	3(3,5m)	4,7	0,40	2(5m)	8,7	0,52
Combination 1	3(3,5m)	3,5	0,30	2(5m)	6,4	0,38
Combination 2	3(3,5m)	5,9	0,51	3(3,5m)	9,7	0,83
Combination 3	3(3,5m)	5,9	0,51	3(3,5m)	9,7	0,83

Seismic Design

Table 6.24 presents the outcomes of the seismic design of model 4. Even if the base shear reduces, the utility of combinations 2 and 3 increases.

Table 6.24: Seismic, Main Model 4

Combination	Modal Mass [kg]	Period [s]	Base Shear [kN]	Critical Bar	Utility
Main Model 4	3274627	1,78	306	Inner Column	0,29
Combination 1	3297788	1,56	367	Inner Column	0,29
Combination 2	3725694	2,26	231	Diagonal	0,42
Combination 3	3672441	2,26	234	Inner Column	0,34

6.1.5 Main Model 5

This model has 18 floors and a total height of 66 meters.

Acceleration

As the natural frequencies decreases in comparison to the previous models, this model is more flexible as seen in Table 6.25. Adding concrete on last nine floors (modification number 10) is reducing accelerations with 20%. Mode shapes are depicted in Figure 6.5.

(a) Mode shape 1 (b) Mode shape 2

Figure 6.5: The first two modes of main model 5

Table 6.25: Acceleration, Main Model 5

Modification parameter	Mass			Natural frequency		Acceleration			
	Total	m_{ex}	m_{ey}	$f_{1.x}$	$f_{1.y}$	a_x	$\Delta a_x/a_x$	a_y	$\Delta a_y/a_y$
Main Model 5	3755328	57529	57608	0,53	0,53	0,031	0	0,054	0
1. Corner Columns: 945x1035mm	3774374	57831	57956	0,52	0,55	0,031	0	0,050	-0,07
2. Inner Columns: 990x990mm	3821478	58576	58598	0,56	0,57	0,029	-0,07	0,051	-0,06
3. Beams: 495x990mm	3881853	59552	59627	0,51	0,53	0,031	0	0,052	-0,04
4. Diagonals: 585x630mm	3774497	57786	57866	0,58	0,58	0,029	-0,07	0,049	-0,09
5. Diagonals: 450x495mm	3739107	57322	57394	0,50	0,51	0,032	0,03	0,056	0,04
6. Connection Stiffness: 2300000 N/mm	3755328	57509	57590	0,54	0,56	0,030	-0,03	0,051	-0,06
7. Connection Stiffness: 1000000 N/mm	3755328	57562	57635	0,50	0,52	0,032	0,03	0,055	0,02
8. Bracing system: Cross Braced	3825616	58438	58523	0,62	0,67	0,026	-0,16	0,042	-0,22
9. Concrete Casting on Top 2 Floors and Roof	4225213	75250	74574	0,46	0,48	0,028	-0,10	0,048	-0,11
10. Concrete Casting on Top 9 Floors and Roof	5321612	97725	97287	0,40	0,42	0,025	-0,19	0,043	-0,20
11. Concrete Casting on all Floors and Roof	6574639	102229	102297	0,39	0,41	0,024	-0,23	0,041	-0,24
12. Wet Green Roof	4171959	74857	73972	0,46	0,48	0,028	-0,10	0,048	-0,11
13. Dry Green Roof	4106175	72008	71287	0,47	0,49	0,028	-0,10	0,049	-0,09
14. Office Floors	4406902	76527	75837	0,46	0,48	0,027	-0,13	0,047	-0,13
15. Offices on Top 4 Floors	3830509	59916	59925	0,52	0,54	0,030	-0,03	0,05	-0,07
Combinations of Modifications									
COMB1: 5+7+15	3588743	60972	60975	0,50	0,51	0,030	-0,03	0,052	-0,04
COMB2: 5+9	4058628	69780	69241	0,45	0,47	0,030	-0,03	0,052	-0,04
COMB3: 6+12/13	3955812	67269	66638	0,50	0,52	0,029	-0,07	0,049	-0,09

Ultimate Limit State

Ultimate limit state design results are expressed in Table 6.26. The most critical combination is utilized with 90 percent.

Table 6.26: ULS, Main Model 5

Combination	Critical Element	Moment[kNm]		Shear Force [kN]		Axial Force [kN]	Load Combination	k_{mod}	Utility
		M_y	M_z	V_y	V_z	N			
Main Model 5	Inner Column	-	-	-	-	8500	ULS Combination 5	0,8	0,69
Combination 1	Diagonal in y-direction	-	-	-	-	1940	ULS Combination 1	1,1	0,90
Combination 2	Diagonal in y-direction	-	-	-	-	1970	ULS Combination 1	1,1	0,90
Combination 3	Inner Column	-	-	-	-	8170	ULS Combination 5	0,8	0,66

Structural Fire Design

Table 6.27 summarizes the results for fire design. Axial forces cause all design utilities in this model.

Table 6.27: Structural Fire, Main Model 5

Combination	Critical Element	Reduced Cross Section [mm]	Reduced Force					Utility
			$M_y[kNm]$	$M_z[kNm]$	$V_y[kN]$	$V_z[kN]$	N [kN]	
Main Model 5	Inner Column	724x724	-	-	-	-	5100	0,35
Combination 1	Diagonal in y-direction	384x364	-	-	-	-	1165	0,51
Combination 2	Diagonal in y-direction	384x364	-	-	-	-	1185	0,52
Combination 3	Inner Column	724x724	-	-	-	-	4845	0,33

Displacements

Lateral Displacement

The maximum allowable lateral displacement for this model is equal to 132mm (u_{max}). Results are portrayed in Table 6.28.

Table 6.28: Lateral displacements, Main Model 5

Variations	Deformation [mm]			
	u_x	u_x/u_{max}	u_y	u_y/u_{max}
Main Model 4	50,0	0,38	76,7	0,58
Combination 1	57,9	0,44	90,9	0,70
Combination 2	53,6	0,41	83,4	0,63
Combination 3	45,8	0,35	69,3	0,53

Interstorey Drift

See interstorey drift results listed in Table 6.29.

Table 6.29: Interstorey drift, Main Model 5

Combination	Displacements					
	x-direction			*y-direction*		
	Storey number (Height of story)	*Drift*	*Utility*	*Storey*	*Drift*	*Utility*
Main Model 5	3(3,5m)	4,7	0,40	3(3,5m)	7,5	0,64
Combination 1	2(5m)	6,6	0,40	2(5m)	10,8	0,65
Combination 2	2(5m)	5,9	0,35	2(5m)	9,5	0,57
Combination 3	3(3,5m)	4,0	0,34	3(3,5m)	6,4	0,55

Seismic Design

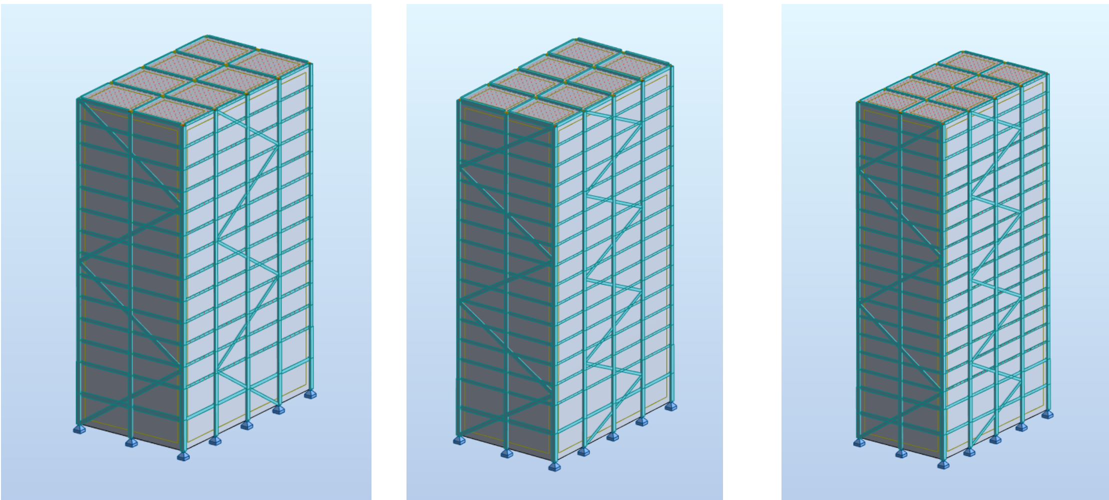
Table 6.30 reports seismic analysis results. All combinations are utilized 30% approximately.

Table 6.30: Seismic, Main Model 5

Combination	Modal Mass [kg]	Period [s]	Base Shear [kN]	Critical Bar	Utility
Main Model 5	3748938	1,87	313	Inner Column	0,29
Combination 1	3588743	2,01	270	Inner Column	0,30
Combination 2	4058628	2,21	270	Inner Column	0,34
Combination 3	3672441	1,95	317	Inner Column	0,34

6.2 Portfolio of Solutions

Three options are offered in this section. These solutions are more complete than the parametric study. Solutions are created based on the previous models. Geometrical properties, cross sections and connection configuration will be presented for each solution. All models are illustrated in Figure 6.6. Procedures and explanations are presented already in Section 5.2.



(a) Solution 1, 14-storey **(b)** Solution 2, 16-storey **(c)** Solution 3, 18-storey

Figure 6.6: Portfolio of solutions

6.2.1 Solution 1, 14-storey

The geometry, cross sections, connection configuration and analysis results for solution 1 of the portfolio will now be displayed.

Properties of Solution

The first option in the portfolio (Table 6.31) is a 14-storey structure with two commercial floors and 12 residential floors. Number of diagonals are also modified in each direction. There is no additional mass in this structure.

Table 6.31: Solution 1, 14-storey

Properties		
Building Geometry	Width x Depth x Height	32x19,2x52m
No. of Floors	Commercial	2
	Residence	12
Height of Floors	Commercial	5m
	Residence	3,5m
No. of Bays	Longitudinal direction	4x8m
	Transversal direction	2x9,6m
No. of Diagonals	Longitudinal direction	5
	Transversal direction	4

When it comes to accelerations, the y-direction is more critical. As a result, cross sections of diagonals are increased in y-direction. See cross sections in Table 6.32.

Table 6.32: Element Dimensions, Solution 1

Timber Element		**Dimension [mm]**
Commercial Beams	x-direction	360x540
	y-direction	450x765
Residential Beams	x-direction	225x405
	y-direction	360x540
Roof Beams	x-direction	270x405
	y-direction	360x540
Commercial Columns	Corner	675x675
	Inner	675x675
Residential Columns	Corner	585x585
	Inner	585x585
Diagonals	x-direction	360x405
	y-direction	405x585

Table 6.33 contains the setup of the diagonal connections. The amount of steel plates was determined by available space in cross sections. Number of dowels and steel plates are more in y-direction.

Table 6.33: Configuration of connections, Solution 1

Properties		**Value**
Thickness of slotted-in steel plate		16mm
Dowel diameter		12mm
No. of steel plates	Diagonals (x)	3
	Diagonals (y)	4
No. of dowels (x)	Column part	15
	Diagonal part	32
No. of dowels (y)	Column part	20
	Diagonal part	45
Stiffness in connection (SLS)	Diagonals (x)	600000 N/mm
	Diagonals (y)	1000000 N/mm
Stiffness in connection (ULS)	Diagonals (x)	420000 N/mm
	Diagonals (y)	740000 N/mm

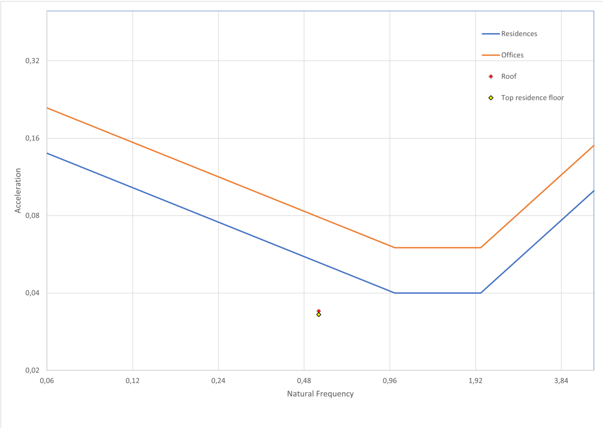
Results from Analysis

Acceleration

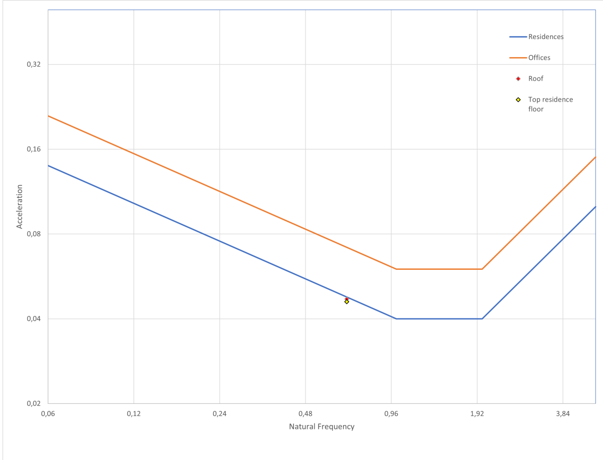
The accelerations on top residential floor and roof are presented in Table 6.34. In Figure 6.7 the accelerations are plotted in the ISO graph (see Figure 3.5). As can be observed in this graph, accelerations are within the limits.

Table 6.34: Acceleration, Solution 1

Natural Frequency	x-direction	0,54 Hz
	y-direction	0,67 Hz
Equivalent mass	x-direction	49978 kg
	y-direction	49956 kg
Roof acceleration	x-direction	0,034m/s^2
	y-direction	0,047 m/s^2
Residence top floor acceleration	x-direction	0,033 m/s^2
	y-direction	0,046 m/s^2



(a) x-direction



(b) y-direction

Figure 6.7: Acceleration satisfaction requirements, Solution 1

ULS

The ULS design result for each critical cross sections are displayed in Table 6.35. Inner residential column is the most critical one to axial force.

Table 6.35: ULS, Solution 1

Timber Element		Combination	k_{mod}	M [kNm]	V [kN]	N [kN]	Utility
Commercial Beams	x-direction	ULS Combination 5	0,8	235	92	-	0,62
	y-direction	ULS Combination 5	0,8	670	253	-	0,73
Residential Beams	x-direction	ULS Combination 5	0,8	76	29	-	0,57
	y-direction	ULS Combination 5	0,8	325	112	-	0,87
Roof Beams	x-direction	ULS Combination 5	0,8	85	29	-	0,52
	y-direction	ULS Combination 5	0,8	325	112	-	0,87
Commercial Columns	Corner	ULS Combination 5	0,8	-	-	3400	0,34
	Inner	ULS Combination 5	0,8	-	-	6360	0,82
Residential Columns	Corner	ULS Combination 5	0,8	-	-	3140	0,43
	Inner	ULS Combination 5	0,8	-	-	5150	0,92
Diagonals	x-direction	ULS Combination 2	1,1	-	-	735	0,49
	y-direction	ULS Combination 1	1,1	-	-	480	0,52

Structural Fire

The fire design results can be seen in Table 6.36. The residential inner column is the most utilized with 69%.

Table 6.36: Structural Fire, Solution 1

Timber Element		Reduced Cross Section	Reduced Force M/V/N [kNm/kN/kN]			Utility
Commercial Beams	x-direction	360x475	141	55	-	0,31
	y-direction	450x610	402	152	-	0,42
Residential Beams	x-direction	225x340	45	17	-	0,31
	y-direction	350x475	195	67	-	0,42
Roof Beams	x-direction	270x340	51	17	-	0,29
	y-direction	350x475	195	67	-	0,42
Commercial Columns	Corner	535x535	-	-	2040	0,26
	Inner	535x535	-	-	3816	0,48
Residential Columns	Corner	445x445	-	-	1884	0,34
	Inner	445x445	-	-	3816	0,69
Diagonals	x-direction	290x265	-	-	441	0,36
	y-direction	335x445	-	-	288	0,10

Displacements

The critical displacement for this model turned out to be the vertical deflection of the roof beams, which gave a utility of 90 percent (see Table 6.37).

Table 6.37: Displacements, Solution 1

		Deformation	Utility
Critical beam	Roof (y-beam)	29	0,90
Lateral displacements	x-direction	47,8	0,46
	y-direction	50,9	0,49
Interstorey drift	x-direction (2nd)	7,2	0,43
	y-direction (3rd)	7,5	0,64

Seismic

The seismic design for this solution was within good range (41 percent). Results are reported in Table 6.38.

Table 6.38: Seismic, Solution 1

Base Shear	x-direction	213kN
	y-direction	285kN
Utility of Critical Element	Inner Column	0,41
Interstorey drift	4th floor (x-direction)	1,97mm
	11th floor (y-direction	3,49mm

6.2.2 Solution 2, 16-storey Building

The parametric study of main model 4 provided this solution. Properties are defined in Table 6.39. The aim is to find a cost-effective solution, which is accomplished by keeping requirements to a minimum.

Properties of Solution

This solution is supplied with 50% green roof, but no concrete floors.

Table 6.39: Solution 2, 16-storey

Properties		
Building Geometry	Width x Depth x Height	32x19,2x59m
No. of Floors	Commercial	2
	Residence	14
Height of Floors	Commercial	5m
	Residence	3,5m
No. of Bays	Longitudinal direction	4x8m
	Transversal direction	2x9,6m
No. of Diagonals	Longitudinal direction	8
	Transversal direction	5

Table 6.40 depicts the cross sections used in this structure. Beams related to green roof is increased for the ultimate limit state purpose.

Table 6.40: Element Dimensions, Solution 2

Timber Element		Dimension [mm]
Commercial Beams	x-direction	360x540
	y-direction	450x765
Residential Beams	x-direction	270x405
	y-direction	360x540
Green Roof Beams	x-direction	360x540
	y-direction	450x810
Commercial Columns	Corner	720x720
	Inner	720x720
Residential Columns	Corner	630x630
	Inner	675x675
Diagonals	x-direction	360x450
	y-direction	450x630

To meet requirements, the connection configuration is done in the same way as the prior solution. Table 6.41 contains information on connections.

Table 6.41: Configuration of connections, Solution 2

Properties		Value
Thickness of slotted-in steel plate		16mm
Dowel diameter		12mm
No. of steel plates	Diagonals (x)	4
	Diagonals (y)	4
No. of dowels (x)	Column part	15
	Diagonal part	35
No. of dowels (y)	Column part	15
	Diagonal part	50
Stiffness in connection (SLS)	Diagonals (x)	824000 N/mm
	Diagonals (y)	917000 N/mm
Stiffness in connection (ULS)	Diagonals (x)	569000 N/mm
	Diagonals (y)	631000 N/mm

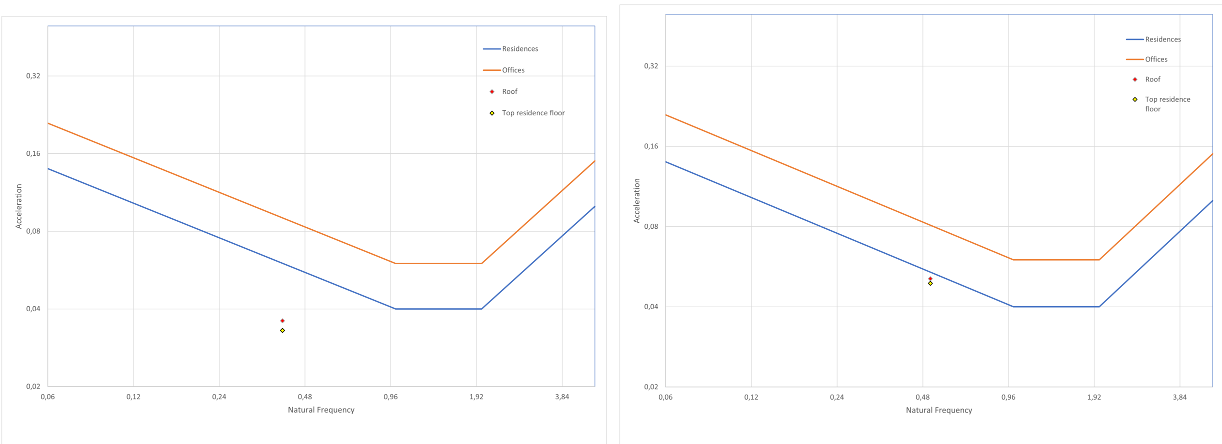
Results from Analysis

Acceleration

Table 6.42 shows the accelerations findings of solution 2. The requirements for this solution are met (Figure 6.8).

Table 6.42: Acceleration, Solution 2

Natural Frequency	x-direction	0,40 Hz
	y-direction	0,51 Hz
Equivalent mass	x-direction	66254 kg
	y-direction	67147 kg
Roof acceleration	x-direction	0,036m/s^2
	y-direction	0,051 m/s^2
Residence top floor acceleration	x-direction	0,033 m/s^2
	y-direction	0,049 m/s^2



(a) x-direction **(b)** y-direction

Figure 6.8: Acceleration satisfaction requirements, Solution 2

ULS

Critical cross section design values for each category are expressed in Table 6.43. The most critical element is commercial column with 90% utilised cross section.

Table 6.43: ULS, Solution 2

Timber Element		Combination	k_{mod}	M [kNm]	V [kN]	N [kN]	Utility
Commercial Beams	x-direction	ULS Combination 5	0,8	235	92	-	0,62
	y-direction	ULS Combination 5	0,8	670	253	-	0,73
Residential Beams	x-direction	ULS Combination 5	0,8	85	40	-	0,51
	y-direction	ULS Combination 5	0,8	325	112	-	0,88
Green Roof Beams	x-direction	ULS Combination 5	0,8	310	110	-	0,84
	y-direction	ULS Combination 5	0,8	900	312	-	0,88
Commercial Columns	Corner	ULS Combination 5	0,8	-	-	4275	0,41
	Inner	ULS Combination 5	0,8	-	-	7870	0,93
Residential Columns	Corner	ULS Combination 5	0,8	-	-	4180	0,25
	Inner	ULS Combination 5	0,8	-	-	6660	0,76
Diagonals	x-direction	ULS Combination 2	1,1	-	-	1080	0,80
	y-direction	ULS Combination 1	1,1	-	-	1320	0,67

Structural Fire

Design results of structural fire can be seen in Table 6.44. By reducing force and cross section due to exposure of fire, the utilisation of each cross section are very well satisfied.

Table 6.44: Structural Fire, Solution 2

Timber Element		Reduced Cross Section	Reduced Force M/V/N [kNm/kN/kN]			Utility
Commercial Beams	x-direction	360x475	141	55	-	0,32
	y-direction	450x700	402	152	-	0,30
Residential Beams	x-direction	270x340	51	24	-	0,29
	y-direction	369x475	195	67	-	0,42
Green Roof Beams	x-direction	360x475	186	66	-	0,40
	y-direction	450x745	540	187	-	0,38
Commercial Columns	Corner	580x580	-	-	2565	0,27
	Inner	580x580	-	-	4722	0,50
Residential Columns	Corner	490x490	-	-	2570	0,37
	Inner	535x535	-	-	3960	0,62
Diagonals	x-direction	290x310	-	-	648	0,19
	y-direction	380x490	-	-	792	0,34

Displacements

The deflections of beam are the same as for solution 1. However, as compared to the preceding solution, lateral displacements have increased. The interstorey drift is reported in Table 6.45.

Table 6.45: Displacements, Solution 2

		Deformation	Utility
Critical beam	Residential beam	29mm	0,90
Lateral displacements	x-direction	66,8	0,57
	y-direction	70,8	0,60
Interstorey drift	x-direction(3rd)	8,2	0,70
	y-direction(3rd)	8,3	0,71

Seismic

As demonstrated in Table 6.46, the seismic analysis results for this solution are within a reasonable range. The green roof beam is utilized 61%.

Table 6.46: Seismic, Solution 2

Base Shear	x-direction	170kN
	y-direction	260kN
Utility of Critical Element	Green Roof Beam	0,61
Interstorey drift	2nd floor (x-direction)	2,3mm
	7th floor (y-direction	7,5mm

6.2.3 Solution 3, 18-storey Building

Because of weight restrictions, the third approach is becoming increasingly difficult to implement. This solution may not meet weight criteria, however it is implemented for external usage in order to do more research. This solution is the one used as example for calculations in Appendix C.

Properties of Solution

The third solution in portfolio is displayed in Table 6.47. It is an 18-storey structure with two commercial floors, 14 residential floors and 2 office floors. There is no additional mass in this solution due to weight satisfaction.

Table 6.47: Solution 3, 18-storey

Properties		
Building geometry	Width x Depth x Height	32x19,2x66m
No. of floors	Commercial	2
	Residence	14
	Offices	2
Height of floors	Commercial	5m
	Residence/Office	3,5m
No. of bays	Longitudinal direction	4x8m
	Transversal direction	2x9,6m
No. of diagonals	Longitudinal direction	8
	Transversal direction	5

All cross sections are listed in Table 6.48. To increase the stiffness of this construction, large diagonals are required.

Table 6.48: Element Dimensions, Solution 3

Timber Element		**Dimension [mm]**
Commercial Beams	x-direction	360x540
	y-direction	450x675
Residential Beams	x-direction	225x405
	y-direction	360x540
Office Beams	x-direction	270x495
	y-direction	360x630
Roof Beams	x-direction	270x495
	y-direction	360x540
Commercial Columns	Corner	675x675
	Inner	765x765
Above Commercial Columns	Corner	675x675
	Inner	720x720
Diagonals	x-direction	450x495
	y-direction	540x585

Connection configuration is presented in Table 6.49.

Table 6.49: Configuration of connections, Solution 3

Properties		**Value**
Thickness of slotted-in steel plate		16mm
Dowel diameter		12mm
No. of steel plates	Diagonals (x)	4
	Diagonals (y)	4
No. of dowels (x)	Column part	20
	Diagonal part	40
No. of dowels (y)	Column part	35
	Diagonal part	66
Stiffness in connection (SLS)	Diagonals (x)	1400000 N/mm
	Diagonals (y)	1600000 N/mm
Stiffness in connection (ULS)	Diagonals (x)	722000 N/mm
	Diagonals (y)	1150000 N/mm

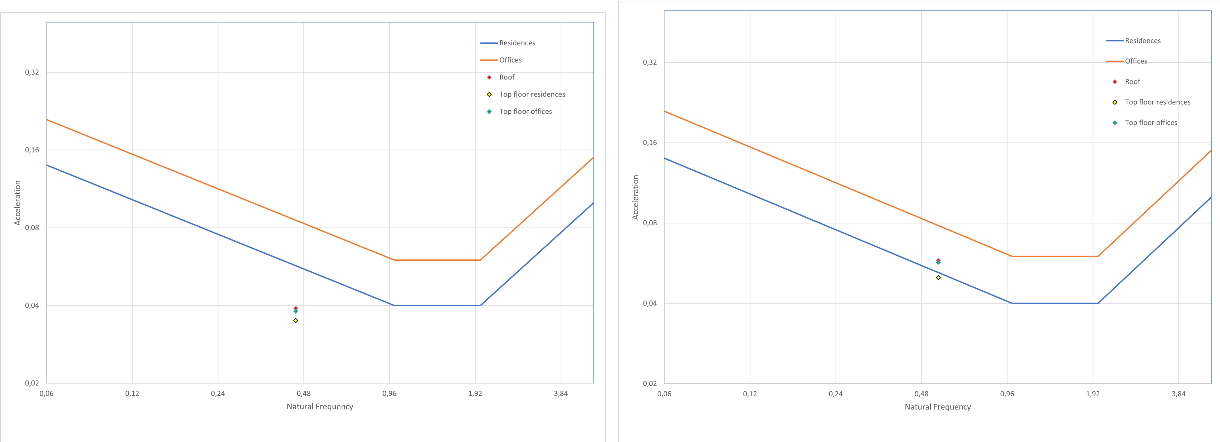
Results from Analysis

Acceleration

There are more conditions to meet in this solution. Both the top floor of an office and the top floor of a residence must be considered. Figure 6.9 depicts this, and values are given in Table 6.50.

Table 6.50: Acceleration, Solution 3

Natural Frequency	x-direction	0,45Hz
	y-direction	0,55Hz
Equivalent mass	x-direction	53209 kg
	y-direction	53219 kg
Roof acceleration	x-direction	0,039 m/s^2
	y-direction	0,058 m/s^2
Residence top floor acceleration	x-direction	0,035 m/s^2
	y-direction	0,050 m/s^2
Office top floor acceleration	x-direction	0,038 m/s^2
	y-direction	0,055 m/s^2



(a) x-direction **(b)** y-direction

Figure 6.9: Acceleration satisfaction requirements, Solution 3

ULS

Diagonals are less utilised in comparison with solution 2, but they are increased due to accelerations requirements. Columns are critical as for both of the previous solutions due to axial compression force. See all design results in Table 6.51.

Table 6.51: ULS, Solution 3

Timber Element		Combination	Kmod	M[kNm]	V[kN]	N[kN]	Utility
Commercial Beams	x-direction	ULS Combination 5	0,8	232	91	-	0,63
	y-direction	ULS Combination 5	0,8	670	251	-	0,73
Residential Beams	x-direction	ULS Combination 5	0,8	130	51	-	0,80
	y-direction	ULS Combination 5	0,8	325	114	-	0,88
Office Beams	x-direction	ULS Combination 5	0,8	140	56	-	0,71
	y-direction	ULS Combination 5	0,8	412	150	-	0,82
Roof Beams	x-direction	ULS Combination 5	0,8	110	45	-	0,62
	y-direction	ULS Combination 5	0,8	321	116	-	0,87
Commercial Columns	Corner	ULS Combination 5	0,8	-	-	4100	0,55
	Inner	ULS Combination 5	0,8	-	-	8400	0,85
Above Commercial Columns	Corner	ULS Combination 5	0,8	-	-	4050	0,52
	Inner	ULS Combination 5	0,8	-	-	7600	0,87
Diagonals	x-direction	ULS Combination 2	1,1	-	-	1260	0,31
	y-direction	ULS Combination 1	1,1	-	-	2400	0,41

Structural Fire

Results of structural fire is portrayed in Table 6.52. All elements in each category are in good range of utility. Most critical one is a office beam on 52%.

Table 6.52: Structural Fire, Solution 3

Timber Element		Reduced Cross Section	Reduced Force M/V/N [kNm/kN/kN]			Utility
Commercial Beams	x-direction	360x475	140	55	-	0,30
	y-direction	450x700	402	150	-	0,32
Residential Beams	x-direction	225385	78	30	-	0,41
	y-direction	360x475	195	68	-	0,42
Office Beams	x-direction	270x430	84	34	-	0,30
	y-direction	360x545	307	90	-	0,47
Roof Beams	x-direction	270x430	66	27	-	0,23
	y-direction	360x475	195	72	-	0,42
Commercial Columns	Corner	535x535	-	-	2500	0,30
	Inner	625x625	-	-	5040	0,50
Above Commercial Columns	Corner	535x535	-	-	2430	0,30
	Inner	580x580	-	-	4560	0,52
Diagonals	x-direction	335x310	-	-	756	0,36
	y-direction	380x445	-	-	1440	0,42

Displacements

See Table 6.53 for values, all utilities are given for strictest requirements for displacements and deflection.

Table 6.53: Displacements, Solution 3

		Deformation	Utility
Critical beam	Office Beam	29mm	0,90
Lateral displacements	x-direction	70,0	0,53
	y-direction	77,4	0,59
Interstorey drift	x-direction(3rd)	6,7	0,57
	y-direction(3rd)	6,5	0,56

Seismic

Seismic results are given in Table 6.54 with 37 percent utilized.

Table 6.54: Seismic, Solution 3

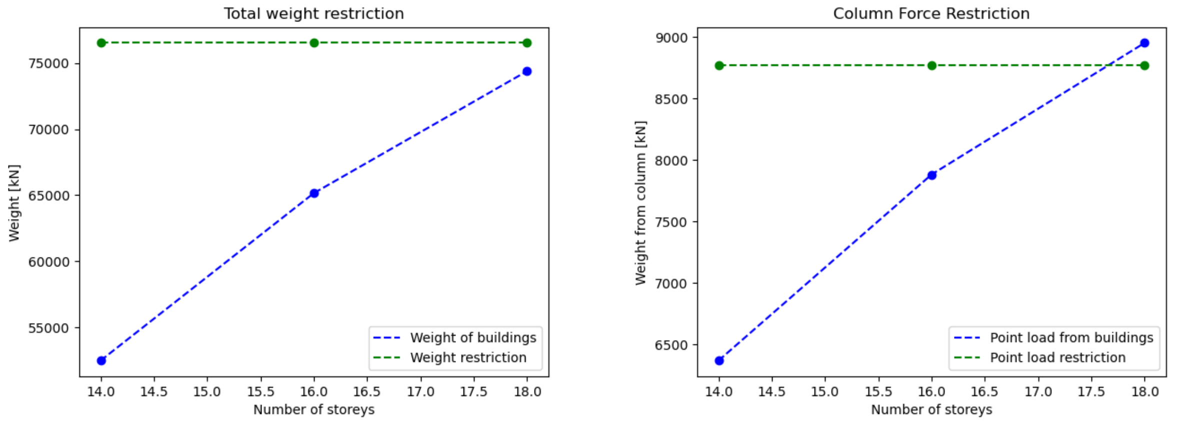
Base Shear	x-direction	220kN
	y-direction	312kN
Utility of Critical Element	Inner Column Commercial	0,37
Interstorey drift	3rd floor (x-direction)	1,6mm
	3rd floor (y-direction)	2,0mm

6.2.4 Results of Weight Satisfaction

The weight requirements are separated into two parts, as detailed in Section 5.4. Weight limits are displayed in graph with results (Figure 6.10). Building and column forces are depicted in Table 6.55. 18-storey (solution 3) is not satisfying the column force in Figure 6.10b.

Table 6.55: Force and weight of solutions

	Total Mass [kg] (ULS Combinations)	**Total Force [kN]**	**Critical Column Force [kN]**
Solution 1, 14-storey	5248100	52481	6370
Solution 2, 16-storey	6515300	65153	7883
Solution 3, 18-storey	7438400	74384	8953



(a) Weight restriction: Total Weight **(b)** Weight restriction: Point Load on Columns

Figure 6.10: Weight restriction for solutions

Chapter 7

Discussion and Conclusion

This chapter includes a discussion of the results and the sources of error. Following the discussion, a short conclusion and ideas for future work is presented.

7.1 Discussion

The discussion will address each topic and exchange of views on all the subjects represented. Firstly we start out with the possibilities of errors in this project.

7.1.1 Sources of Error

All models computed in this thesis might have sources of error. The errors can be in the calculations, simplifications or modelling.

Modelling

Generally, the model might contain some errors. The timber members to be discussed for uncertain errors are slab, rigid link and diagonals.

Slab

In reality, the slabs can differ from how it is modelled in this thesis. For instance, the connected part to beams is assumed pinned. The slabs are screwed and connected to beams, which can give some rotational stiffness in addition. It is believed to be negligible though.

The slab is simplified in *Robot*, and modelled as shell element with a thickness. See Section 4.4.1 for procedure and explanation of how the slab is modelled. Further, the slab in *Robot* is a single panel. The actual deck defined is a composite with more elements composed together.

Following by this method, simplifications can lead to errors in model. Frequencies and deflections might be altered due to slab treated as one single part instead of composite of more elements. In our thesis it is believed that our way is a good approximation of how the slab is modelled.

Diagonals

Diagonals are introduced with translational connection stiffness in each end. This stiffness can vary with reality, in both favourable or unfavourable values. This is uncertain value that can affect the total structural stiffness.

In our model, diagonals are intersecting with beams over the height. Those intersections are made as fully stiff connected, and this is giving wrong values on some points to the internal forces. However, for our thesis these values are ignored completely.

Connection Calculation

For the calculation of transverse failure of the multiple-shear plane dowel-connection, it was assumed that the statement of Eurocode 5 §8.1.3(1) was acceptable (see Section 2.3.1 and Appendix C2). However, the calculated value of a multiple shear connection estimated by this method may not always agree with the actual shear strength of the connection [21]. The reason for this is that the shear strength of dowel connections as obtained by the yield theory corresponds to the yield mode, and multiple shear connections have characteristic yield modes [21]. This is the main source of error for this way of calculating the shear strength of a multiple shear connection, but since the Eurocode 5 states it this way, it is assumed to be permitted, if compatible failure modes are combined. Timber standard requires that failure modes (e), (f) and (j/l) from the table must not be combined with the other failure modes.

Calculation of Wind Load

The wind has been modelled as a uniformly distributed wind field on each side of the building, with no variation over the height or width. This is done as a conservative simplification to NS-EN 1991-1-4. The wind pressure on the roof has been neglected in the simulations because they were assumed to have little effect on the results due to small values (see Appendix A3). The details of the distribution can be found in Figure 3.3.

Additionally, it is important to note that the method in NS-EN 1991-1-4 is a simplification, both with respect to simplification of the geometry for the building and the calculation of wind forces.

In the NS-EN1991-1-4, the wind is assumed to be a static load, only blowing orthogonal on one side of the building. In reality, wind is a dynamic action, and the structural factor $c_s c_d$ is conservatively set equal to 1 in this thesis, this is used to simplify the wind to be a static force.

Calculation of Acceleration

The calculation of acceleration is done according to the simplified method presented in Annex B in NS-EN 1991-1-4, which is a simplification and contains a lot of room for interpretations. This process is lengthy and complex with many uncertain parameters and variables: such as the force factor (c_f), the non-dimensional frequnecy (f_L(z,n)), the equivalent mass (m_e), and the dimensionless coefficient (K).

Weight Calculation

The calculation of the weight requirement contains some assumptions based on information given by external part (*Sweco*). The requirement was given as a line load of 1370 kN/m with a load-width of 11m. More information was not given, and the calculation is therefore based on assumptions, as explained in Section 5.4.

7.1.2 Parametric Study

Section 5.2 describes the variables in the parametric study. Each of the parameters will now be explored in detail, with an emphasis on their influence on acceleration.

Cross Sections of Columns and Beams

The starting point of chosen cross sections were made out of using similar structure as *Mjøstårnet* (Section 2.2.2) [35]. When the structure approached the same height, the results were also expected to be close to *Mjøstårnet*.

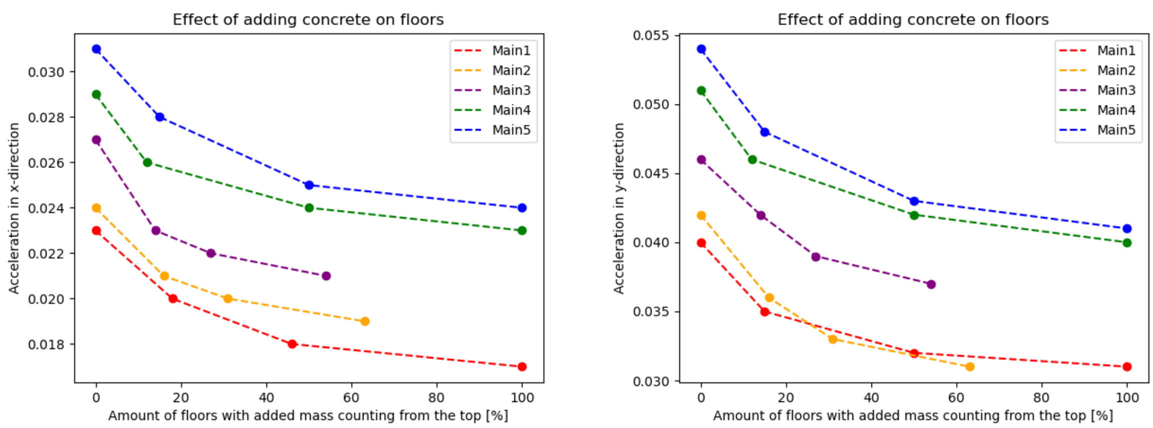
The stiffness of the global structure should theoretically be increased by increasing the cross section area. The second moment of inertia for a rectangular cross section, $I = \frac{w \cdot h^3}{12}$, demonstrates this, where h represents the cross section's height and w represents its width. Since stiffness is expressed by the relationship between EI (bending stiffness) and L (length), choosing a larger cross section size clearly increases the total stiffness (k).

In this study, modifying the cross sections of columns and beams had minimal effect on accelerations compared to other factors. Table 6.25 shows the best effect of modifying the cross section of columns, with a 7 percent drop in accelerations. Increasing the size of the columns rises the stiffness and mass of the building. The natural frequency increases when the proportion increase in stiffness as a result of increased cross section size exceeds the percentage rise in mass (Equation 2.6b). However, because the stiffness of the beams and columns have little effect on the lateral stiffness of the building (owing to the bracing technique used), this increase in stiffness had little influence on the acceleration.

Mass

The mass is an important variable in structural vibrations, as explained in Section 2.4. The natural eigenfrequency decreases as structures gain mass without changing stiffness. It should be noted that lower frequency is not advantageous for acceleration. On the other hand, mass reduces accelerations, as seen in the tables in Section 6.1. Newton's second law is one way to explain this: $F = ma$, where F denotes force, m denotes mass, and a denotes acceleration. When the equation is expressed in terms of acceleration instead of mass, $a = F/m$, it is clear that increasing the mass causes a decrease in acceleration.

The approach of adding mass is used to reduce accelerations in both *Mjøstårnet* and *Treet* (Sections 2.2.2 and 2.2.1). Figure 7.1 depicts the graphical representation of adding concrete to each of the main models.



(a) x-direction **(b)** y-direction

Figure 7.1: Effect of adding concrete

Figure 7.1 and results in Tables 6.1, 6.7, 6.13, 6.19 and 6.25 indicate that the effect flattens as the number of floors with added concrete grows (starting from the top floor). In other words, the right number of levels from top to bottom must be added with concrete to provide the best impact of minimizing motions on top.

The equivalent mass formula can explain why the effect is dying out. Adding concrete or other masses to the top floors is more advantageous since it is combined with a greater value of the mode shape vector (ϕ), which is normalized to 1 at the roof. For further information on calculating m_e, see Equation 3.6a. The equivalent mass calculations are performed in Appendix C5.1.

Stiffness

In dynamics, the only variables used to calculate frequency are stiffness and mass. Following mass, stiffening the structure is beneficial for roof vibrations. Diagonal cross sections, connection stiffness and bracing system are the parameters to be discussed.

Cross Sections of Diagonals

Figure 7.2 depicts acceleration as the diagonal cross section area, and hence the axial stiffness of them, changes. The results from the parametric study show that changing the cross-section of diagonals have a major impact on roof accelerations of the building. Modifying the cross-section area means the axial stiffness changes. This is demonstrated by the stiffness formula EA/L, where A is the cross-sectional area, E is the elasticity modulus and L is the length of the member. The effect is present in all the main models, as can be observed in Section 6.1. For instance, Table 6.7 shows a decrease of the acceleration in y-direction by an amount of 17% by increasing the cross-sections of diagonals from 360x540mm to 450x855mm.

This is to be expected given that the diagonals are primarily responsible for the structure's lateral stiffness as a result of the chosen bracing system. When the lateral stiffness of a structure grows, so does the natural frequency, provided that the mass of the building does not rise in the same proportion, as shown by Equation 2.6b.

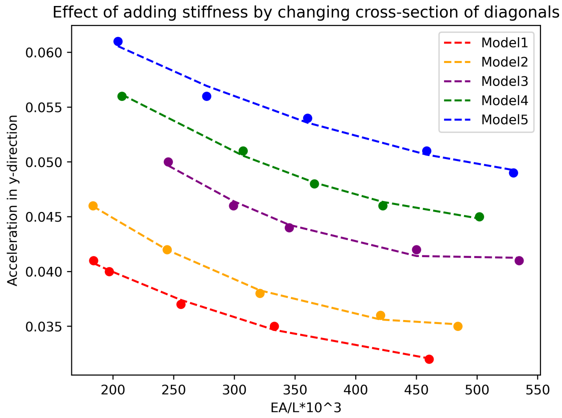


Figure 7.2: Effect of modifying connection stiffness (y-direction)

Increasing the stiffness and keeping the mass approximately the same results in higher frequency and thus lower accelerations, which is illustrated in the figure above.

Bracing System

Increasing stiffness by changing from a diagonal braced to a cross-braced bracing system made a big difference for the accelerations. In proportion to mass, stiffness increases significantly. This is due to the additional stiffness provided by such a system to the building. Table 6.1 indicates a 30% drop in y-direction acceleration and a 26% drop in x-direction acceleration for main model 1. Similar results may be shown in Section 6.1 for the remaining main models.

Connection Stiffness

The slip modulus of the diagonal-column connection has been demonstrated to have an influence on acceleration. As the stiffness of the connection increases, the frequency rises, resulting in lesser acceleration (see also Section 6.1). For example, doubling the slip modulus of the connections in main model 2 resulted in an 8% drop in the x-direction and a 7% drop in the y-direction. The connection, however, cannot be stiffer than a pinned connection, and any additional increase in translational stiffness is considered to be flattened when approaching perfectly pinned behaviour.

7.1.3 Damping Coefficient

The damping coefficient in timber constructions is typically assumed to be between 1,5 and 2 percent for each project. The acceleration varies noticeably as the damping ratio is changed, with an increase in damping resulting in lower accelerations. This has not been tested in this thesis, but it can be observed in Equation 3.4a as the damping is accounted for in the resonance component of the response (R) in the standard deviation. When estimating the frequency of each mode, modal analysis does not take damping into account.

In this project, the damping ratio (ξ) is 1,9 percent, which is also used in *Mjøstårnet*. Aerodynamic damping and damping devices are ignored, as stated in Section 3.4.2. Aerodynamic damping is specified in NS-EN-1994-1-1 [7] and would add damping to the overall structure. This is a gain for this thesis because it can further lower accelerations, but it is conservatively ignored here. Damping devices were not taken into account in this assignment.

Following the aerodynamic damping, the damping coefficient can be included in modal analysis to achieve a lower value of natural eigenfrequency. This is particularly advantageous for the acceleration needs, as seen in Figure 3.5.

Damping is an uncertain variable. Since the value used in this thesis is most likely conservative, better results could be obtained by raising the damping coefficient, either by adding aerodynamic damping, damping devices or by performing modal analysis with damping coefficient, or combined.

7.1.4 Discussion of Limit State Results

This section will go through how the parameters affected the structure in terms of ULS, SLS, structural fire and seismic design.

ULS

Ultimate limit state design for this project is not expected to be critical, but there are some timber elements on the limit of utilization. These high utilization presented in Sections 6.1 and 6.2 are mainly due to gravitational loads. However, diagonals are critical due to wind load.

The inner columns at the lower levels are critical due to high axial stresses. The total axial compressive force of solution 3 is about 9000 kN, as anticipated by the findings in *Mjøstårnet* [35] construction. The axial force is important in the majority of the findings, but beams become crucial when extra loads are placed on the flooring. Appendix C3 has calculation examples.

There is always the possibility of reducing utility by increasing cross section size, as seen in Table 6.8 on the roof beam owing to the addition of a green roof on top. In accordance with NS-EN-1995-1-1 [8], square cross sections of columns are done with regard to compression and less slenderness, which is good for the buckling criteria.

SLS

Vertical deflections are not considered before the portfolio of solutions. Usually beams are the most critical one in such a structure due to gravity load (deflections). All solutions in the portfolio is satisfied. The effect of increasing cross sections are beneficial for deflections, but one can also adjust the deflection requirement, as explained in Section 3.4.3.

Lateral displacements and interstorey drift results are affected by horizontal stiffness and the flexibility of the structure. As the number of floors increases, the structure becomes more flexible, resulting in greater lateral displacements. The graphic depiction is presented in Figure 7.3.

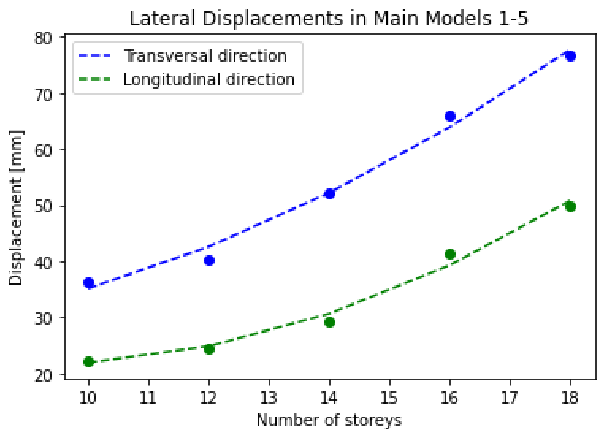


Figure 7.3: Effect of adding stories for lateral displacements

As shown in the illustration, increasing storeys increases the lateral displacement. The most critical utility for the models in the parametric study is 70%, while for the solutions in the portfolio, it is 60% (Chapter 6).

The interstorey drift is becoming increasingly critical as floors are added to the building. The initial floors are usually the most crucial for any model, however this is also reliant on adding mass on top and floor height in the structure. The most critical interstorey drift utilization for the portfolio of solutions is 70% (Table 6.45).

Structural Fire

Fire design in this thesis is done with a conservatively assumption. The structural fire calculation in Appendix C4 do not take any extra measures to protect the cross sections. This is known to be conservatively as gypsum, fire paint, and other materials may be used for protection of each cross section. It should be noted that structural fire does not apply to the slab system employed in this thesis.

Diagonals exposed to fire do reduce the effective depth twice in height and once in width resulting in even more slender cross section, since they have considerable lengths. Slender cross sections are unfavourable for buckling. An example is shown in Table 6.21.

Aside from that, fire is a not a critical requirement for the considered tall building with such big cross sections in use. The most critical for structural fire design in the portfolio of solutions is solution 1 with 69 percent utility in the inner column (Table 6.3).

Seismic

The parametric analysis illustrates the influence of parameters on seismic design through parametric combinations.

Mainly, the results present that adding of mass on the top floors results in higher seismic utilities, see Section 6.1. On the other hand, it may be noted that adding of mass provides the structure with higher natural periods, which can be explained from the formula: $T = 2\pi/\sqrt{k/m}$. This is favourable with respect to seismic design.

The distribution of the added mass in plan and elevation of the building is an important factor. Placing the mass uniformly in the plan and elevation renders earthquake-induced inertia forces to be uniformly distributed throughout the building, instead of being localized at a few parts of the building [43]. Thus, since the added masses in the parametric study is localized in the top of the building, this could affect the seismic design. The results do not show clearly that this is the case.

For main model 2, 3 and 4, the results show that less base shear is created when reduction of diagonal stiffness is combined with adding of mass at top floors. It can be noted that when the natural period increases the base shear decreases, see Figure 3.8. As a result, it is difficult to distinguish the effects of combinations including a decrease in diagonal stiffness and an increase in top mass. However, the higher utilities may be due to the fact that overall loading increases as a result of the extra weights, rather than the effects on the seismic design. For instance, 6.7 shows that the utility of the critical beam becomes 0,62 for combination 2. In this combination the weight from green roof is added, and thus it is only natural that the utility increases.

In combination 3 of main model 5, see Table 6.25, where increased connection stiffness and adding of weight on roof were combined, the natural period and the base shear increased. This is interesting, because it shows that this combination is unfavourable for seismic design. Therefore, adding of mass at top floors could be interpreted as not beneficial for seismic design because of the uneven distribution of mass. On the contrary, the results of this thesis do not necessarily substantiate this statement since the mass-parameter has been combined with different stiffness-parameters in all cases.

Regardless, the seismic design has shown to be not critical for this project. The increase of seismic utilities seem to be more related to adding of weight rather than favourable or unfavourable effects on the seismic design, as discussed.

7.1.5 Weight Satisfaction

As previously stated, the weight requirement is set by the external part. Both weight constraints are met for the 14- and 16-storey designs in the portfolio. When looking at the overall weight requirement, the 18-storey structure is met, but when looking at the critical point load, it becomes critical. The causes of mistake for the interpretation of the weight requirement are explained in Section 7.1.1 and the requirements are portrayed in Section 5.4.

7.2 Conclusion

The acceleration requirements of this structure in height were first difficult to overcome, as projected. When floors were added to each main model design, the vibrations increased. For all base models, the greatest acceleration was 0,054 m/s^2. Along with the main models, the portfolio of solutions had the highest acceleration result of 0,058 m/s^2. All of the options met the acceleration criterion depicted in Figure 3.5.

Moreover, the parametric research is carried out to demonstrate how accelerations for tall timber constructions might be influenced. Modifying stiffness or adding stiffness (or both) has the greatest impact on acceleration (see Section 6.1). Increasing rigidity with diagonals or inserting concrete in the upper floors are most significantly decreasing vibrations on upper floors. In addition, damping coefficient is said to have a significant impact.

For tall timber structures, lateral displacements might be challenging to satisfy. The interstorey drift follows in relation with lateral movements. Our findings reveal that if acceleration issues are addressed, horizontal displacements are satisfied. Both the lateral displacements and interstorey drift increase when the floors are added to the structure.

It is worth noting ultimate limit state, structural fire, deflections and seismic design were not found to be critical in our project. Even though high utilities (particularly ULS and deflection) are observable in Chapter 6, they can easily be altered by adjusting cross sections or reconsidering deflection criteria.

This thesis began with weight requirements from outside partners (Appendix D). In our results, solution 1 or 2 (Figure 6.10) satisfies this criteria. In other words, 18-storey building cannot be satisfied without any additional research.

Finally, based on the research findings, a tall timber building can be built over tunnel in *Økern Centre, Area 2* [1]. Timber is proven to be a better option than other materials as concrete or steel structures for this purpose.

7.3 Future Work

This project is limited to a single system, with several parameters being investigated to see how the structure might be improved or cost-effective. More research on the aforementioned subjects should be done in the future. These topics are listed:

- Geometrical changes
  - Footprint area
  - Number of bays
  - Height of floors
- Support conditions
- Detailed solution
  - Connections
  - Foundations
  - Optimizing elements (beams, columns and diagonals)
  - Acoustic
- Study of wight satisfaction in detail
- Bracing systems (e.g. shear walls, cores, etc.)
- Use of CLT-elements

Because our thesis is a parametric study, the items listed are those that can be explored in depth. In addition, our thesis contains simplifications for both calculation and modelling. More experiments and comprehensive calculations should be performed in the future.

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List of Appendices

Appendix A

Loads

A1. Live Load

Table shows how the buildings vary from different live load cases in accordance with NS-EN1995-1-1, Table 6.2.

Category	**Load**
A – Residence Area	2,0 kN/m^2
B – Office Area	3,0 kN/m^2
D – Commercial Area	5,0 kN/m^2

In addition, live load values are given by *Sweco* (Appendix D). This load is in compliance with Eurocode values. First two floors are assumed for commercial use, remaining floors are for office or residential use.

A2. Snow Load

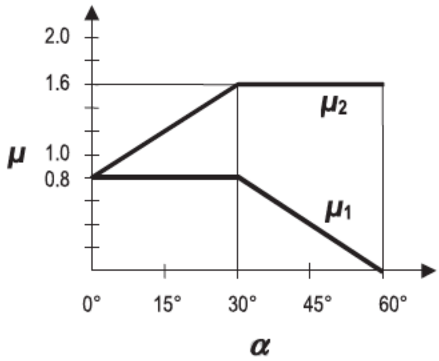
Snow load is calculated in NS-EN1991-1-3. Location, height and shape of roof decides the characteristic value of snow load. Calculation of snow load on roof is given in Equation (EC1-1-3, 5.1). It is worth to mention that s has been used as s_k in this assignment.

$$s = \mu_i C_e C_t s_k$$

where,

μ_i	is shape factor, equal 0,8 for flat roof (Table 5.2)
C_e	is the exposure factor, equal 1
C_t	is the thermal coefficient, equal 1
s_k	is characteristic snow load

In figure below, one can see how shape factor vary with roof angle α. Figure is given in (EC1-1-1-3, Figure 5.1).



Characteristic snow load in National Annex (NA.4.1, Table NA.4.1(901)) located in Oslo is varying in different heights above sea level.

Height (meters above sea level, m.s.l)	$s_{k,0}$
0 – 150 m.s.l	$3{,}5\ kN/m^2$
151 – 250 m.s.l	$4{,}5\ kN/m^2$
251 – 350 m.s.l	$5{,}5\ kN/m^2$
> 350 m.s.l	$6{,}5\ kN/m^2$

Sweco has given value for snow load as characteristic value on **2,8 kN/m²** in Appendix D. This value is the one used for this project.

A3. Calculation of static wind load

General remarks:

Units used in script:

- Length/height: $[m]$
- Force: $[N]$
- Velocity: $\left[\frac{m}{s}\right]$
- Density: $\left[\frac{kg}{m^3}\right]$

All equations- and chapter-references are from the EC1-1-4.

Geometry of the building:

Height: $h := 66$

Width: $b := 32$

Depth: $d := 19.2$

Basic values:

The fundamental value of basic wind velocity: $v_{b.0} := 22$ (Table NA.4)

Directional factor: $c_{dir} := 1$ (Chapter 4.2(2), NOTE2)

Season factor: $c_{season} := 1$ (Chapter 4.2(2), NOTE3)

Probability factor:
For characteristic wind combination (EN1990, eq.(6.14), SLS) we set return period, T=50, which gives cprob=1. (this is an irreversible load combination happening rarely, every 50 years, therefor damage should be limited when it happens).

$c_{prob} := 1$

Mean wind:

Height above sea level at construction site:

$$H := 0$$

Height above sea level where level correction begins:
(When c_alt = 1)

$H_0 := 900$ (Table NA.4 (901.2))

The height above sea level where max. level correction is reached
(When c_alt is at its max.):

$H_{topp} := 1500$ (Table NA.4 (901.2))

Threshold value for wind velocity:

$v_0 := 30$ (NA.4.2(2)P (901.1))

Factor for the wind increasing with the height over the sea:

$c_{alt} := 1$ (Table NA.4(901.3))

Basic wind velocity: $v_b := c_{dir} \cdot c_{season} \cdot c_{alt} \cdot c_{prob} \cdot v_{b.0}$ (eq. NA.4.1)

$$v_b = 22$$

Referance height:
As a conservative assumption we make the windload uniformly distributed over the height of the building with the peak value. This means the only referance height needed is the total height of the building, both for internal and external pressure.

$z_e := h$ (Figure 7.2)

$z_i := h$ (Figure 7.2)

Terrain category: $TK := 4$ (Table 4.1)

Orography factor: $c_0 := 1$ (Chapter 4.3.1, NOTE 1)

Roughness length:

$$z_0 := \begin{array}{l} \text{if } TK = 0 \\ \quad \| \; 0.003 \\ \text{else if } TK = 1 \\ \quad \| \; 0.01 \\ \text{else if } TK = 2 \\ \quad \| \; 0.05 \\ \text{else if } TK = 3 \\ \quad \| \; 0.3 \\ \text{else if } TK = 4 \\ \quad \| \; 1 \end{array} = 1$$

(Chapter 4.3.2)

Minimum height:

$$z_{min} := \begin{array}{l} \text{if } TK = 0 \\ \quad \| \; 2 \\ \text{else if } TK = 1 \\ \quad \| \; 2 \\ \text{else if } TK = 2 \\ \quad \| \; 4 \\ \text{else if } TK = 3 \\ \quad \| \; 8 \\ \text{else if } TK = 4 \\ \quad \| \; 16 \end{array} = 16$$

(Table 4.1)

Max. height: $z_{max} := 200$ (Chapter 4.3.2)

Terrain factor: $k_r := 0.24$ (Table NA.4.1)

Roughness factor:
$$c_r(z) := \text{if } (z \ge z_{min}) \wedge (z \le z_{max}) \; \left\| k_r \cdot \ln\left(\frac{z}{z_0}\right) \right. \;\; \text{else if } (z \le z_{min}) \; \left\| c_r(z_{min}) \right.$$
(Eq. 4.4)

Mean wind velocity:
$$v_m(z) := c_0 \cdot c_r(z) \cdot v_b$$
(Eq. 4.3)

$$v_m(z_e) = 22.121$$

Wind turbulence:

Turbulensfaktor: $k_I := 1$ (Eq. 4.7)

Standard deviation: $\sigma_v := k_r \cdot v_b \cdot k_I$ (Eq. 4.6)

Turbulence intensity:
$$I_v(z) := \text{if } (z \ge z_{min}) \wedge (z \le z_{max}) \; \left\| \frac{\sigma_v}{v_m(z)} \right. \;\; \text{else if } (z \le z_{min}) \; \left\| I_v(z_{min}) \right.$$
(Eq. 4.7)

Peak velocity pressure:

Air density: $\rho := 1.25$ (Chapter 4.5)

Peak factor: $k_p := 3.5$ (Chapter NA.4.4)

Mean velocity pressure: $q_m(z) := 0.5 \cdot \rho \cdot v_m(z)^2$ (Chapter NA. 4.5)

Peak velocity pressure:
(4.5, eq. 4.8)
$$q_p(z) := (1 + 2 \cdot k_p \cdot I_v(z)) \cdot q_m(z)$$
(Eq. NA. 4.8)

$$q_p(z_e) = 816.851$$

Wind pressure on surfaces:

External pressure coefficients for buildings:

External wall surfaces: (Table 7.1)

$$c_{pe.10.A} := -1.2$$

$$c_{pe.10.B} := -0.8$$

$$c_{pe.10.C} := -0.5$$

$$c_{pe.10.D} := 0.8$$

$$c_{pe.10.E} := -0.7$$ (varies)

External flat roof: (Table 7.2)

$$c_{pe.10.F} := -1.8$$

$$c_{pe.10.G} := -1.2$$

$$c_{pe.10.H} := -0.7$$

$$c_{pe.10.I} := 0.2$$

Internal pressure coefficients for buildings is ignored

Wind pressure when the pressure is at the longest surface:

Longest surfaces:

Sone D: $q_{p.D} := q_p\left(z_e\right) \cdot c_{pe.10.D} = 653.481$ (pressure)

Sone E: $q_{p.E} := q_p\left(z_e\right) \cdot c_{pe.10.E} = -571.796$ (suction)

Short surfaces:

Sone A and B:

$q_{p.A} := q_p\left(z_e\right) \cdot c_{pe.10.A} = -980.221$ (suction)

Wind pressure when the pressure is at the shortest side:

Longest sides:

Sone A and B:

$q_{p.A} := q_p\left(z_e\right) \cdot c_{pe.10.A} = -980.221$ (suction)

Short sides:

Sone D: $q_{p.D} := q_p\left(z_e\right) \cdot c_{pe.10.D} = 653.481$ (pressure)

Sone E: $q_{p.E} := q_p\left(z_e\right) \cdot c_{pe.10.E} = -571.796$ (suction)

Wind pressure on roof (neglected in this project):

Sone F/G/H/I - Pressure on sone I (Suction on rest)
$q_{p.I} := q_p\left(z_e\right) \cdot c_{pe.10.I} = 163.37$

A4. Load Combinations

All load combinations are determined in accordance with NS-EN-1990. Only a set of most decisive load combinations are chosen out.

Combination equation as in Eurocode (6.10a and 6.10b)

	Permanent	Dominant	Non-dominant
6.10a	$\gamma_{Gj,sup} \cdot g_{kj,sup}$	$\gamma_{Q,1} \cdot \psi_{0,1} \cdot q_{k,1}$	$\gamma_{Q,i} \cdot \psi_{0,i} \cdot q_{k,i}$
6.10b	$\xi \cdot \gamma_{Gj,sup} \cdot g_{kj,sup}$	$\gamma_{Q,1} \cdot q_{k,1}$	$\gamma_{Q,i} \cdot \psi_{0,i} \cdot q_{k,i}$

Firstly, we define our load- cases and factors in project.

Load Cases	Symbol
Dead Load	g
Snow Load	s
Wind Load	w
Live Load	q

Load Factor	Value
$\boldsymbol{\xi}$	*0,89*
$\boldsymbol{\gamma_{Gj,sup}}$	*1,35*
$\boldsymbol{\gamma_{Q,1}}$ *(unfavourable)*	*1,5*
$\boldsymbol{\gamma_{Q,i}}$ *(unfavourable)*	*1,5*
$\boldsymbol{\psi_{0,1}}$ *(snow/live)*	*0,7*
$\boldsymbol{\psi_{0,i}}$ *(snow/live)*	*0,7*
$\boldsymbol{\psi_{0,1}}$ *(wind)*	*0,6*
$\boldsymbol{\psi_{0,i}}$ *(wind)*	*0,6*

We present now the combinations used for this project.

	Permanent	Dominant	Non-dominant
Combination 1 and 2	*g*	*w*	*q+s*
Combination 3 and 4	*g*	*q*	*w+s*
Combination 5	*g*	*q*	*s*

Wind load is divided into w1 and w2 where

w1	is pressure on longitudinal surface
w2	is pressure on transversal surface

Load combinations will now be portrayed for both, Ultimate Limit State and Serviceability Limit State with all factors involved (defined in tables above). Combinations are expressed in tables, where alle are in accordance with Eurocode.

ULS Combinations

	Permanent	Dominant	Non-dominant
Combination 1a	$1{,}35g$	$(1{,}5 \cdot 0{,}6)w1$	$(1{,}5 \cdot 0{,}7)(q + s)$
Combination 1b	$(0{,}89 \cdot 1{,}35)g$	$1{,}5w1$	$(1{,}5 \cdot 0{,}7)(q + s)$
Combination 2a	$1{,}35g$	$1{,}5 \cdot 0{,}6)w2$	$(1{,}5 \cdot 0{,}7)(q + s)$
Combination 2b	$(0{,}89 \cdot 1{,}35)g$	$1{,}5w2$	$(1{,}5 \cdot 0{,}7)(q + s)$
Combination 3a	$1{,}35g$	$(1{,}5 \cdot 0{,}7)q$	$(1{,}5 \cdot 0{,}6)w1 + (1{,}5 \cdot 0{,}7)s$
Combination 3b	$(0{,}89 \cdot 1{,}35)g$	$1{,}5q$	$(1{,}5 \cdot 0{,}6)w1 + (1{,}5 \cdot 0{,}7)s$
Combination 4a	$1{,}35g$	$(1{,}5 \cdot 0{,}6)q$	$(1{,}5 \cdot 0{,}6)w2 + (1{,}5 \cdot 0{,}7)s$
Combination 4b	$(0{,}89 \cdot 1{,}35)g$	$1{,}5q$	$(1{,}5 \cdot 0{,}6)w2 + (1{,}5 \cdot 0{,}7)s$
Combination 5a	$1{,}35g$	$(1{,}5 \cdot 0{,}7)q$	$(1{,}5 \cdot 0{,}7)s$
Combination 5b	$(0{,}89 \cdot 1{,}35)g$	$1{,}5q$	$(1{,}5 \cdot 0{,}7)s$

$*0{,}89 \cdot 1{,}35 = 1{,}20$
$*1{,}5 \cdot 0{,}7 = 1{,}05$
$*1{,}5 \cdot 0{,}6 = 0{,}90$

SLS Combinations

	Permanent	Dominant	Non-dominant
Combination 1	g	$w1$	$0{,}7(q + s)$
Combination 2	g	$w2$	$0{,}7(q + s)$
Combination 3	g	q	$0{,}6w1 + 0{,}7s$
Combination 4	g	q	$0{,}6w1 + 0{,}7s$
Combination 5	g	q	$0{,}7s$

Appendix B

Procedures in Software

B1. Procedure of *Dynamo Sandbox* for Parametric Study

In this appendix, the *Dynamo* script is followed step by step. First out is the illustration of how the footprint base is defined.

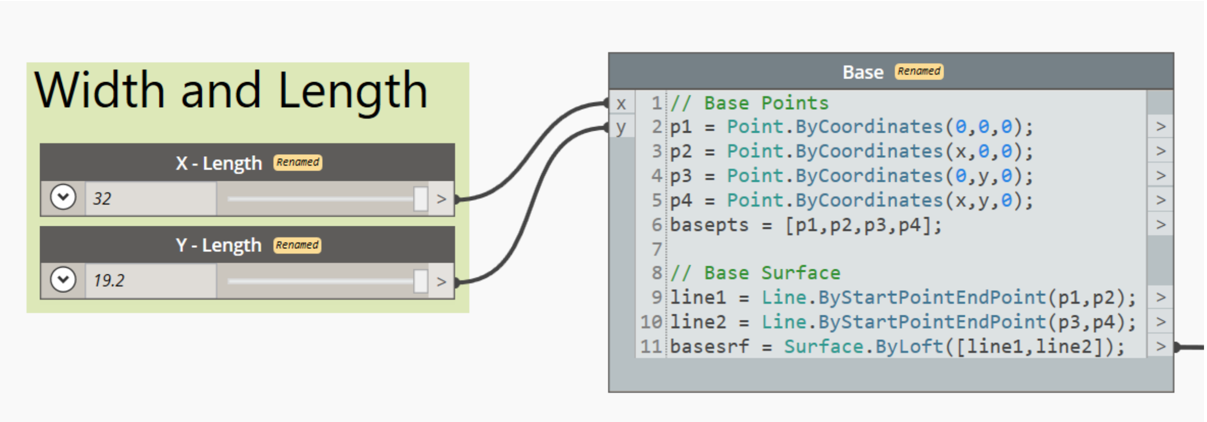


Figure 1: Base surface

The base surface is defining the footprint area. Width and length can be decided parametrically as shown. This code boxes are defining and used for our geometrical structure. Further on, the structure is made with lines and panels shown below.

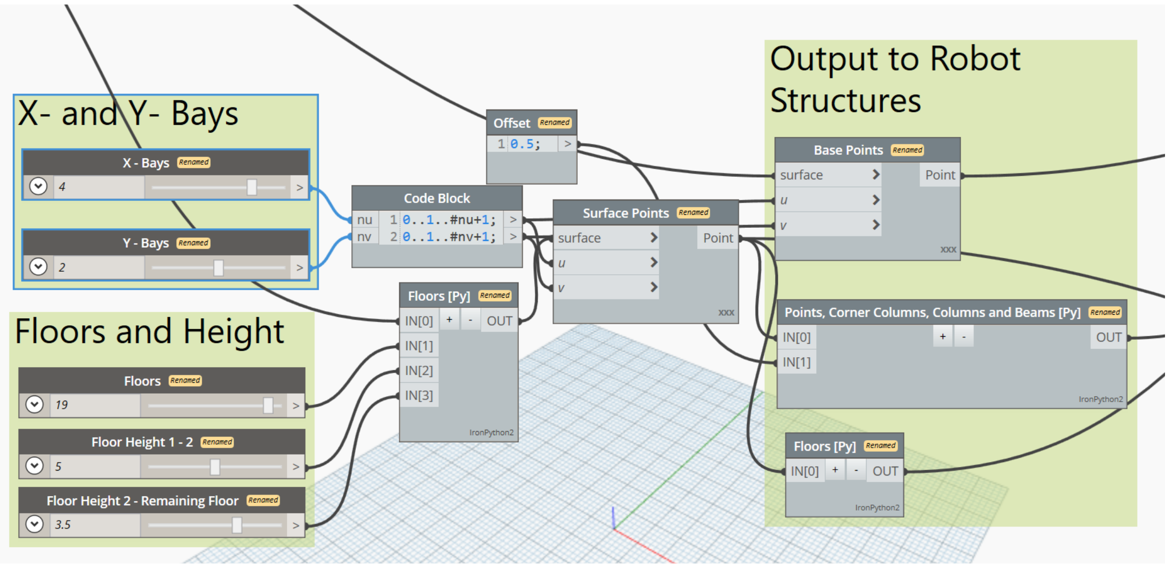


Figure 2: Geometry of structure

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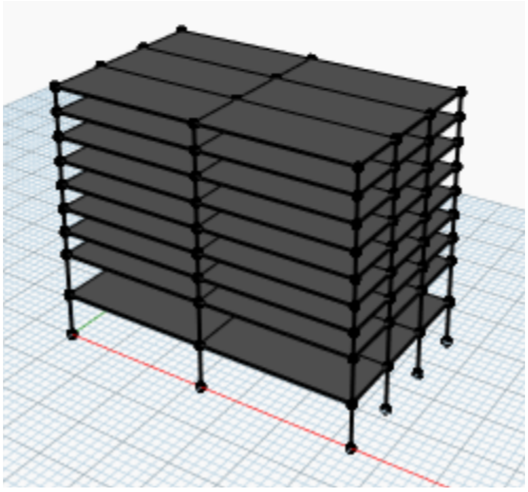
The geometry of structure is decided by defining number of floors, number of x- and y-bays and the height of commercial floors and residential. This is resulting in separate floors between bays, separate beams between columns and continuously columns. Lines (beams and columns) and surfaces (floors) are scripted in Python as shown in Figure above. Example of an output would look like.



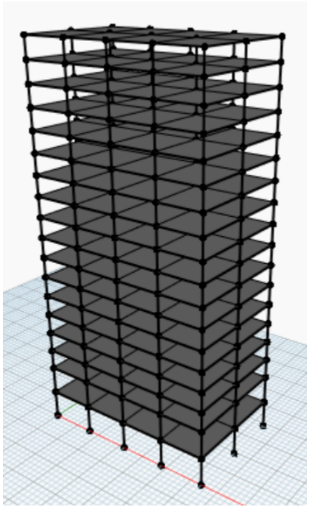


Figure 3: Geometrical output

This figure shows how one can change bays, floors and floor height easily in structure. By defining the geometry of the structure as shown, the next procedure is to connect and transfer to *Robot Structures* by using the package *Structural Analysis for Dynamo*. Diagonals are manually modelled in *Robot Structures*. The following next steps are converting the geometry over to *Robot*.

In *Dynamo* with help of the package, we create analytical nodes, bars and panels. Surface can be done as shown in figure below. It is also possible to assign thickness to the surface by defining one in *Robot*, and it will automatically be shown up in *Dynamo*.

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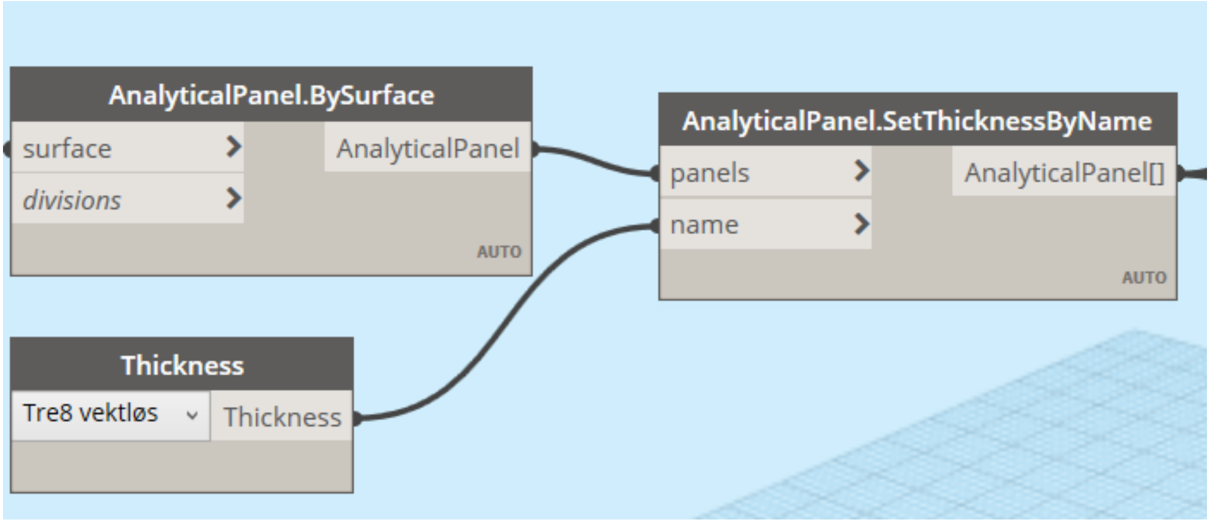

Figure 4: Floors from Dynamo to Robot

Further releases on the surfaces are also manually done in *Robot Structures*. There is no attribute in the structural package to linear release slabs, which is needed for floors.

Following the surfaces, beams and columns are done similarly. However, it is possible to give releases on lines, and this is also done in *Dynamo*. Base supports are also defined in *Dynamo*, as it is possible by help of the package to convert nodes into one of boundary conditions. This is shown in a figure.

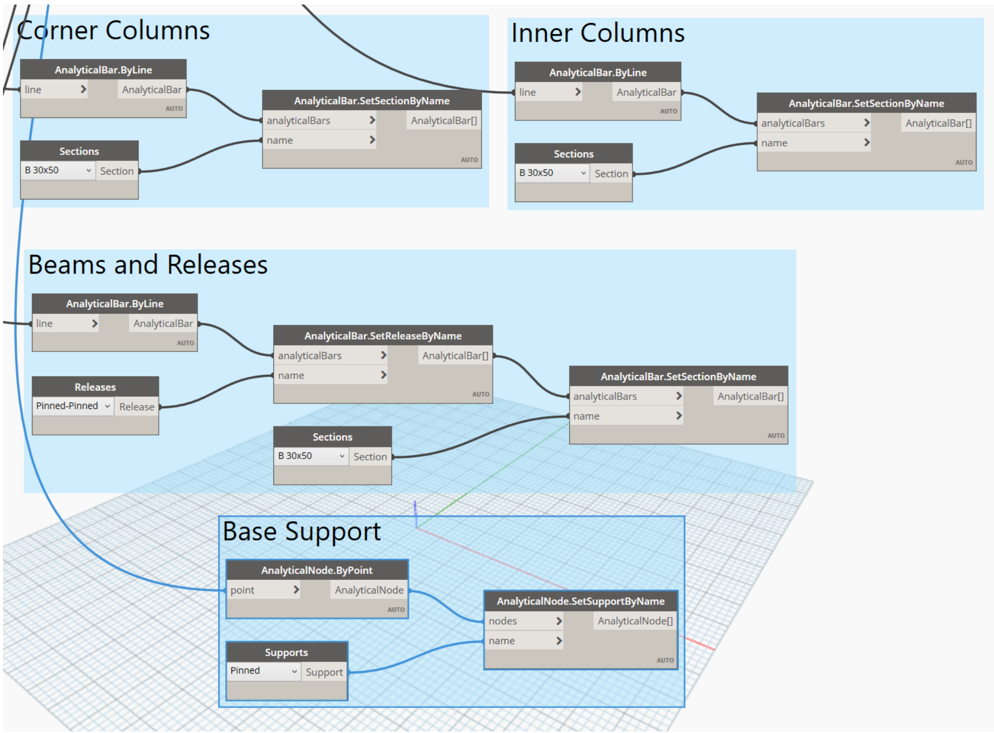

Figure 5: Code for transfer beams, columns and nodes from Dynamo to Robot

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As for deciding thickness in Robot for floors, by adding cross section for beams and columns it will automatically be listed in Dynamo. The same procedure goes for the boundary conditions. Another important parameter is the rigid link, which is done with help of Python script for separating master and slave nodes. This is shown here.

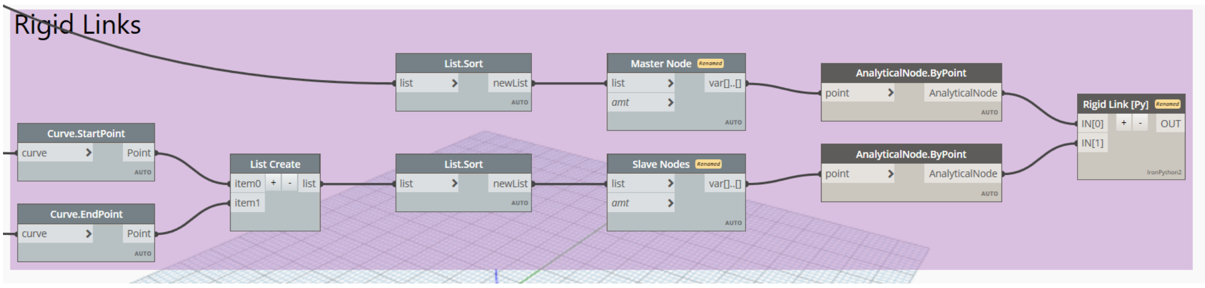


Figure 6 Rigid links in Dynamo

By doing all this, the structure is quite good modelled and transferred to *Robot*. Further, the diagonals are manually modelled in *Robot* as mentioned. However, it is important to know that the load cases for dead, snow and live is also done in *Dynamo* with help of the package, and the visual programming would look like.

After the structural model is introduced to *Robot*, the load analysis is given by adding it to *Dynamo*. Notation is that wind load and added mass as concrete, slab weight and green roof is done in *Robot* manually. The combinations of the loads are also not done in *Dynamo*. Live loads, dead loads and snow load is given in *Dynamo*.

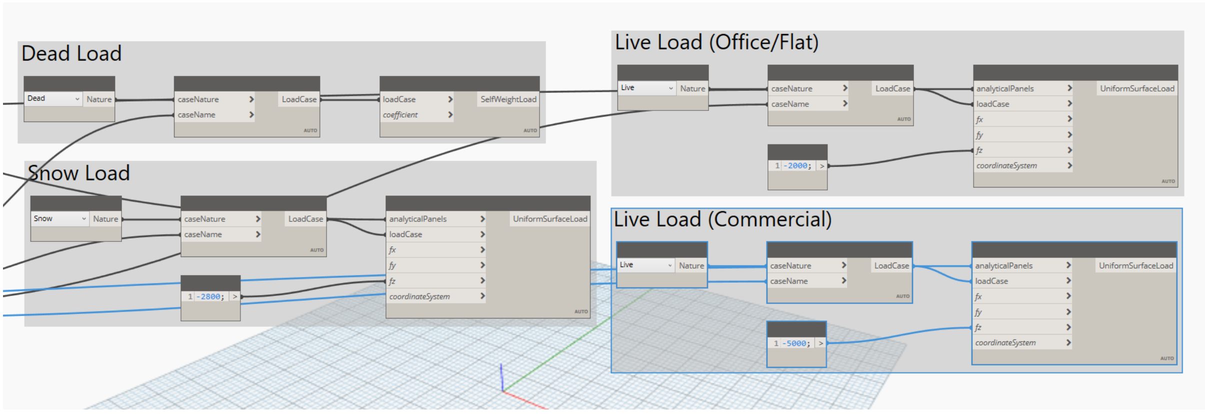


Figure 7: Load cases in Dynamo

Ending this Appendix, we clearly see that this is quite helpful for the efficiency for a parametric study. One can easily do modifications to obtain best optimized structure.

B2. Seismic design in *Robot Structural*

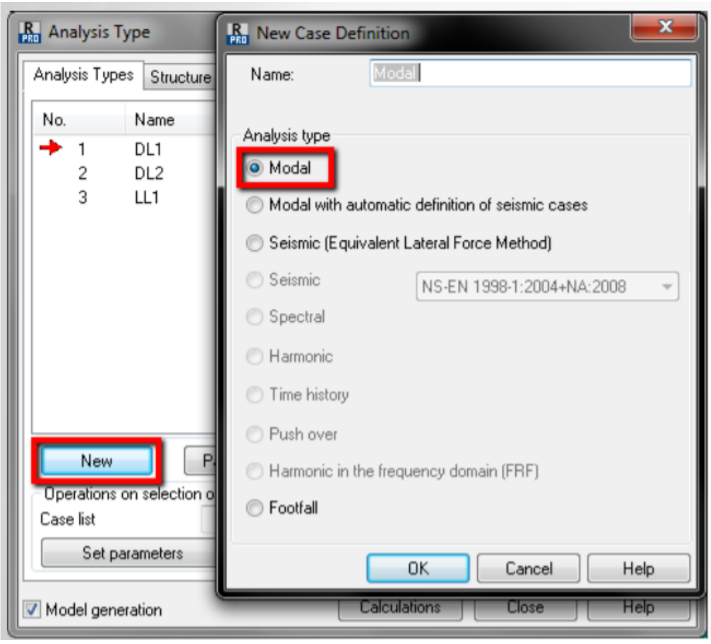
The seismic design will be performed in *Robot Structural*. In this software there are two options as to how the seismic calculation should be performed:

- Lateral force method of analysis
- Response spectrum method

The response spectrum method is based upon the modal analysis and is more accurate than the Lateral force method of analysis and is thus chosen for this thesis. In this Appendix the method for how to perform this type of analysis in *Robot Structural* will be presented.

For seismic analysis based on the response spectrum method, all data is defined the same way as in modal analysis. Additionally, parameters required by a specific national code to establish the response spectrum shape must be specified. Calculations and results are the same as those for spectral analysis.

Step 1: Defining modal analysis.



The modal analysis is defined as a load case under "Analysis Type". For the seismic design it is convenient to define the no. of modes based on the demand that over 90% of mass participation is accounted for.

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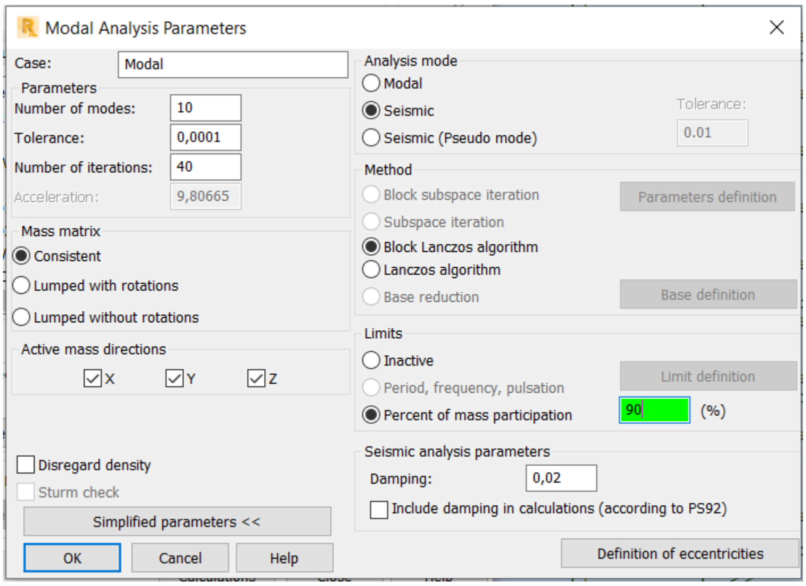
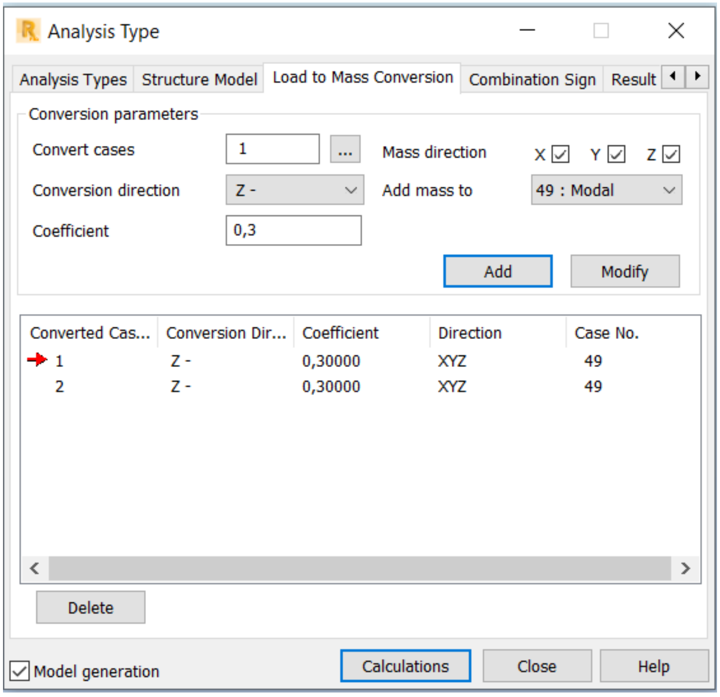
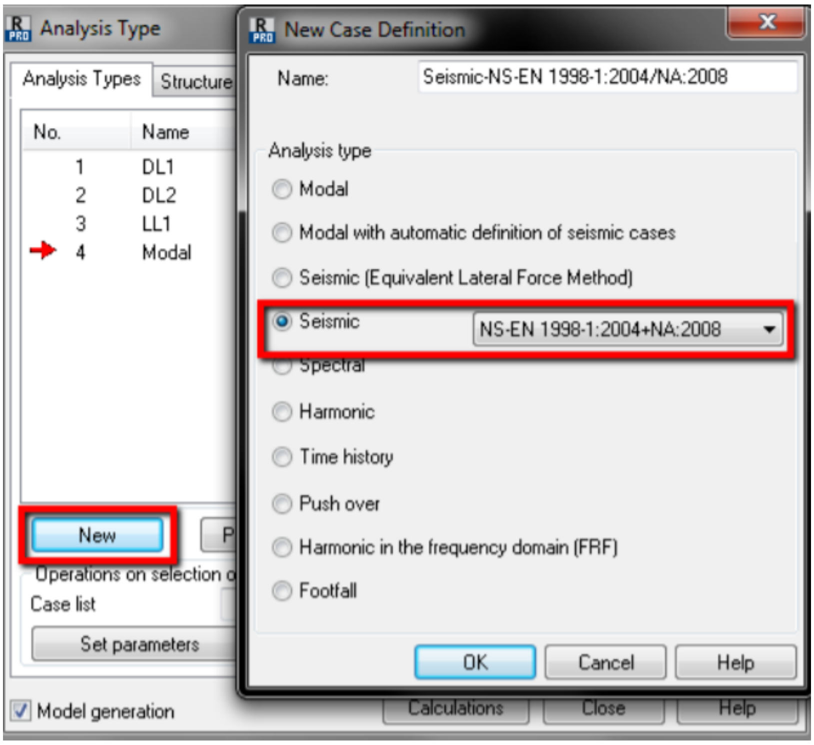


Step 2: Load to mass conversion.

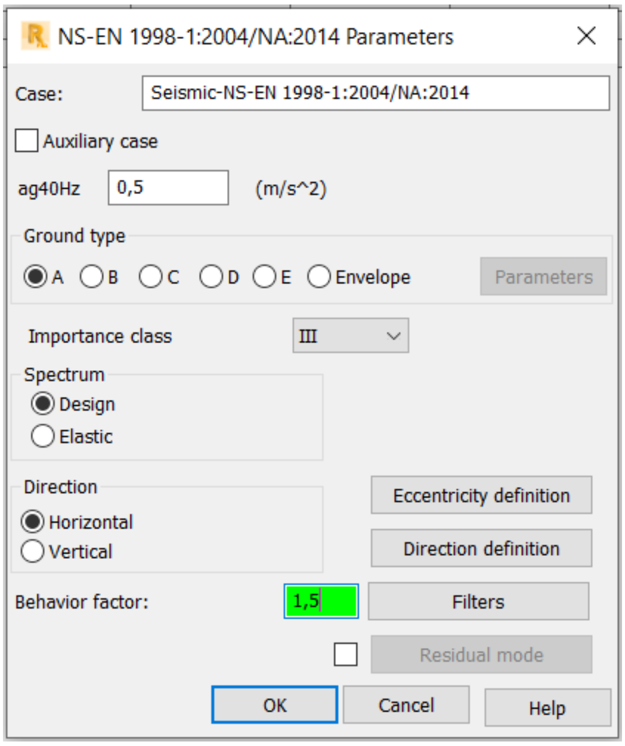


Defining added masses manually using “Load Definition”, or by converting existing load cases to masses.

Step 3: Defining seismic analysis



The seismic analysis is defined as a new load case.



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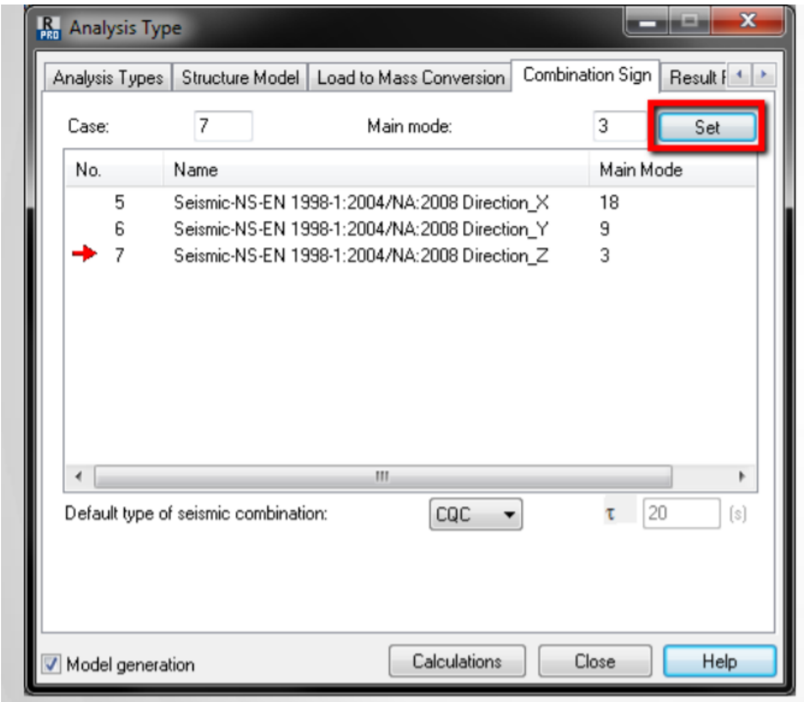
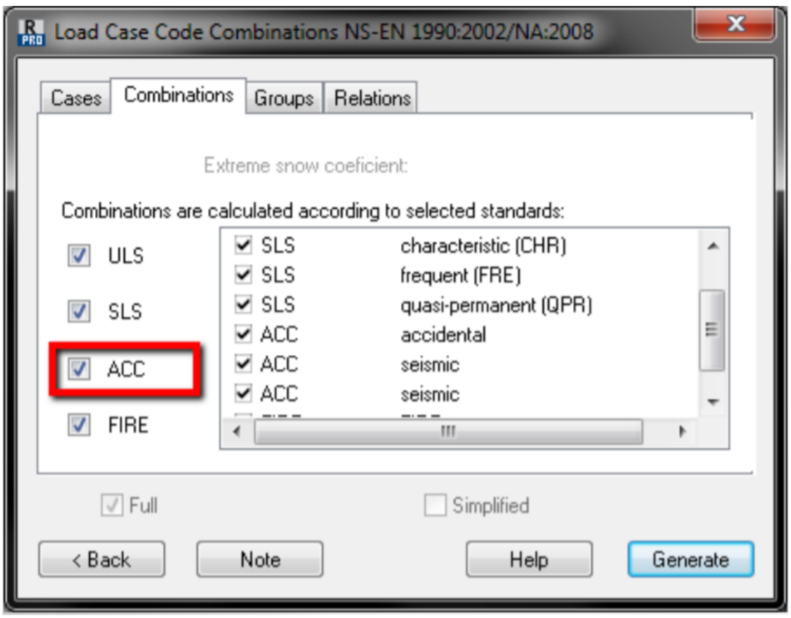
Step 4: Set combination signs.

In case of using signed quadratic combinations, it will be necessary to set main modes for each of the directions. Usually, the main criterion to select such modes is their contribution to participation mass for given direction. This contribution can be checked in Dynamic Analysis Results with appropriate columns added. In this thesis we have only looked at the seismic effect in x and y direction and neglected the z direction.



Step 5: Making seismic code combinations.

Defining design combinations (manual or automatic ones) considering static load cases and dynamic combinations.



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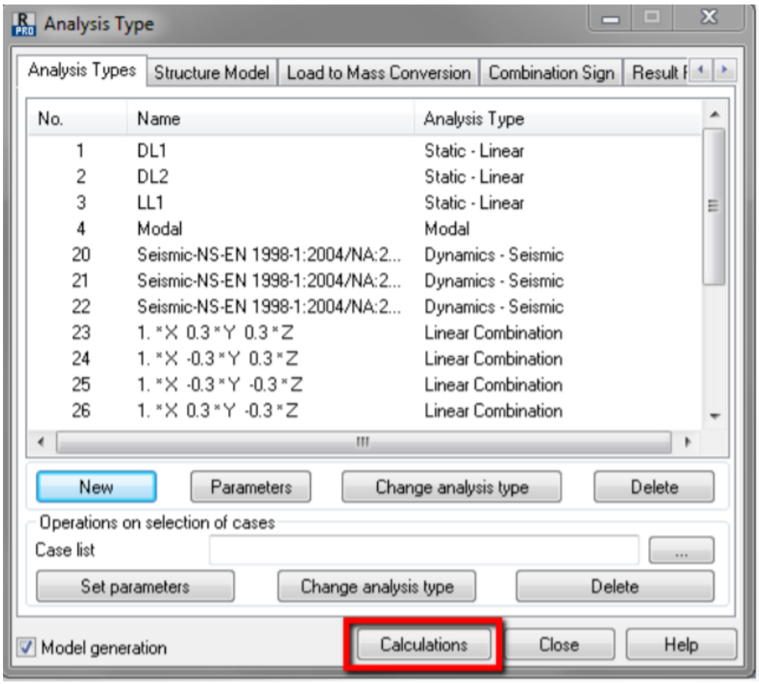
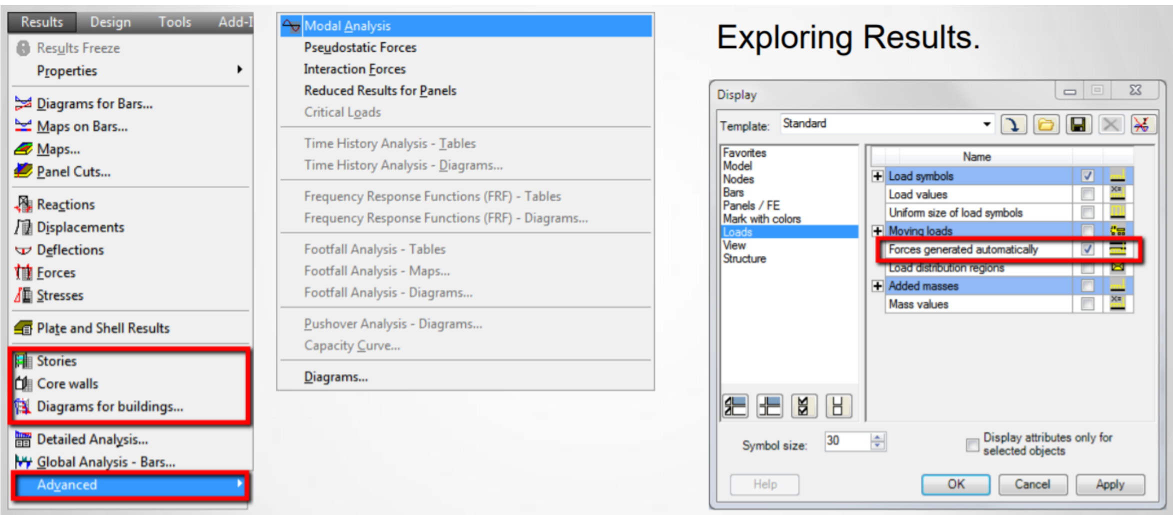
Step 6: Calculations

Run calculations.



Step 7: Analyse the results



Appendix C

Manual Calculations

C1. Pre-liminary design of slabs

Formulas for this calculation has been extrated from NS-EN 1995-1-1 (2004).
Material properties for Glulam elements have been extracted from EN 14080 (2013).
Material properties for LVL Kerto Q elements have been extracted from Metsa Wood's catalogue.

Units used in script:

- Dimensions/lengths:	$[mm]$
- Forces:	$[N]$
- Moments:	$[Nmm]$
- Stresses/strengths:	$\left[\frac{N}{mm^2}\right]$
- Areas:	$\left[mm^2\right]$
- 2nd moment of inertia:	$\left[mm^4\right]$
- Densities:	$\left[\frac{kg}{m^3}\right]$

General data:

Span-length: $l := 8500$

Width of 1 slab element: $b := 2400$

Distance between Glulam members: $CC := 565$

Service class: $Sc := 2$

Load-duration: $Ld := \text{"Medium"}$

Loads:

ULS: $q_{Ed.ULS} := 9.9 \cdot 10^{-3} \cdot CC = 5.594$

SLS-characteristic: $q_{Ed.SLS1} := 7 \cdot 10^{-3} \cdot CC = 3.955$

The combinations that have given these design loads are the ones that can be found in Appendix A4.
ULS COMB5 has been used for the ULS load
SLS COMB5 has been used for the SLS characteristic load

Internal forces:

Formulas corresponding to simply supported floors.

Bending moments $M_{Ed.ULS} := \frac{1}{8} \cdot q_{Ed.ULS} \cdot l^2 = 5.052 \cdot 10^7$

$M_{Ed.SLS1} := \frac{1}{8} \cdot q_{Ed.SLS1} \cdot l^2 = 3.572 \cdot 10^7$

Shear forces $V_{Ed.ULS} := \frac{1}{2} \cdot q_{Ed.ULS} \cdot l = 2.377 \cdot 10^4$

$V_{Ed.SLS1} := \frac{1}{2} \cdot q_{Ed.SLS1} \cdot l = 1.681 \cdot 10^4$

Material properties:

Safety-factor for Glulam and LVL: $\gamma_M := 1.15$
NS-EN 1995-1-1, table NA.2.3

Webs: GL30c

Dimensions:

Middle beams: $h_w := 405$ $b_w := 66$

Edge beams: $h_{w.edge} := 405$ $b_{w.edge} := 140$

Factors:

Modification factor: $k_{mod.web} := 0.8$
NS-EN 1995-1-1, table 3.1

Time-property factor: $k_{def.web} := 0.8$
NS-EN 1995-1-1, table 3.2

Cracking factor: $k_{cr.w} := 0.8$

Bending strength: $f_{mk.w} := 30$

$$f_{md.w} := \frac{f_{mk.w}}{\gamma_M} \cdot k_{mod.web} = 20.87$$

Shear strength: $f_{vk.w} := 3.5$

$$f_{vd.w} := \frac{f_{vk.w}}{\gamma_M} \cdot k_{mod.web} = 2.435$$

Axial compr. strength: $f_{c.0.k.w} := 24.5$

$$f_{c.0.d.w} := \frac{f_{c.0.k.w}}{\gamma_M} \cdot k_{mod.web} = 17.043$$

Axial tension strength:

$f_{t.0.k.w} := 19.5$

$f_{t.0.d.w} := \frac{f_{t.0.k.w}}{\gamma_M} \cdot k_{mod.web} = 13.565$

Mean young`s modulus: $E_{0.mean.w} := 13000$

Mean shear modulus: $G_{web.mean} := 650$

Characteristic density: $\rho_{k.w} := 390$

Mean density: $\rho_{mean.w} := 430$

Top flange: LVL Kerto Q

Height: $h_{t.f} := 43$

The width of the flange will be an effective width that is chosen according to NS-EN 1995-1-1, table 9.1, where shear lag and plate-buckling will be accounted for. This will be done later in the calculation-process.
Properties for LVL found in EN 13986 (2004).

Factors:

Modification factor:
NS-EN 1995-1-1, table 3.1

$k_{mod.t.f} := 0.8$

Time-property factor:
NS-EN 1995-1-1, table 3.2

$k_{def.t.f} := 1$

Bending strength:

$f_{mk.t.f} := 36$

$f_{md.t.f} := \frac{f_{mk.t.f}}{\gamma_M} \cdot k_{mod.t.f} = 25.043$

Shear strengths:

$$f_{v.0.edge.k.tf} := 4.5$$

$$f_{vd.f.tf} := \frac{f_{v.0.edge.k.tf}}{\gamma_M} \cdot k_{mod.t.f} = 3.13$$

$$f_{v.0.flat.k.tf} := 1.3$$

$$f_{v.0.flat.d.tf} := \frac{f_{v.0.flat.k.tf}}{\gamma_M} \cdot k_{mod.t.f} = 0.904$$

Axial compr. strength:

$$f_{c.0.k.tf} := 26$$

$$f_{c.0.d.tf} := \frac{f_{c.0.k.tf}}{\gamma_M} \cdot k_{mod.t.f} = 18.087$$

Axial tension strength:

$$f_{t.0.k} := 26$$

$$f_{t.0.d.f} := \frac{f_{t.0.k}}{\gamma_M} \cdot k_{mod.t.f} = 18.087$$

Young`s modulus:

$$E_{0.mean.t.f} := 10500$$

Shear modulus:

$$G_{t.f.mean} := 600$$

Mean density:

$$\rho_{mean.t.f} := 510$$

Bottom flange: LVL, Kerto Q

Height: $h_{b.f} := 61$

Factors:

Modification factor: $k_{mod.b.f} := 0.8$
NS-EN 1995-1-1, table 3.1

Time-property factor: $k_{def.b.f} := 1$
NS-EN 1995-1-1, table 3.2

Bending strength: $f_{mk.b.f} := 36$

$$f_{md.b.f} := \frac{f_{mk.b.f}}{\gamma_M} \cdot k_{mod.b.f} = 25.043$$

Shear strengths: $f_{v.0.edge.k.bf} := 4.5$

$$f_{v0.edge.d.bf} := \frac{f_{v.0.edge.k.bf}}{\gamma_M} \cdot k_{mod.b.f} = 3.13$$

$f_{v.0.flat.k.bf} := 1.3$

$$f_{v.0.flat.d.bf} := \frac{f_{v.0.flat.k.bf}}{\gamma_M} \cdot k_{mod.b.f} = 0.904$$

Axial compr. strength: $f_{c.0.k.bf} := 26$

$$f_{c.0.d.bf} := \frac{f_{c.0.k.bf}}{\gamma_M} \cdot k_{mod.b.f} = 18.087$$

Axial tension strength: $f_{t.0.k.bf} := 19.5$

$$f_{t.0.d.bf} := \frac{f_{t.0.k.bf}}{\gamma_M} \cdot k_{mod.b.f} = 13.565$$

Young`s modulus: $E_{0.mean.b.f} := 10500$

Shear modulus: $G_{b.f.mean} := 600$

Densities: $\rho_{k.b.f} := 510$ $\rho_{mean.b.f} := 510$

Effective width of top and bottom flange:

Acc. to NS-EN 1995-1-1, table 9.1

The effective flange widths found in table 9.1 is the maximum allowable flange widths that we can have so that we avoid shear lag and plate buckling.
Another thing we should avoid is the overlapping of flange widths. If the effective width used is larger than the center distance of webs, then they overlap, which is not allowed. Therefor, we choose flange widths below the demand in table 9.1, and also below the centre distance.
EC5-1-1, §9.1.2.(5): Unrestrained flange width is smaller than twice the plate buckling value in Table 9.1 -> no detailed buckling investigation required.

Tensile flange:
Must account for shear lag

$$b_{ef.tensile} := min\left(0.1 \cdot l , 20 \cdot h_{b.f}\right) = 850$$

Compression flange:
both shear lag and plate buckling must be accounted for

$$b_{ef.compr} := min\left(0.1 \cdot l , 20 \cdot h_{t.f}\right) = 850$$

- Max. effective width top flange:

Middle beams: $b_{ef.t} := b_{ef.compr} + b_w = 916$

Edge beams: $b_{ef.t.edge} := 0.5 \cdot b_{ef.compr} + b_{w.edge} = 565$

- Max. effective width bottom flange:

Middle beams: $b_{ef.b} := b_{ef.tensile} + b_w = 916$

Edge beams: $b_{ef.b.edge} := 0.5 \cdot b_{ef.tensile} + b_{w.edge} = 565$

- Chosen width of bottom flange:

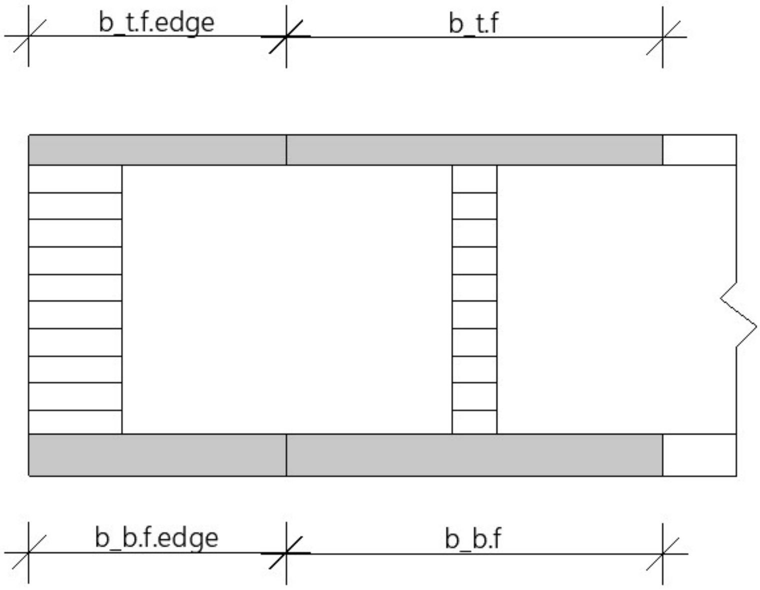
Middle beams: $b_{b.f} := 565$

Edge beams: $b_{b.f.edge} := 282$

- Chosen width of top flange:

Middle beams: $b_{t.f} := 565$

Edge beams: $b_{t.f.edge} := 282$



- Control of shear lag and plate buckling:

Middle beams:

$$\begin{array}{l} \text{if } \left(b_{b.f} \le b_{ef.tensile}\right) \wedge \left(b_{t.f} \le b_{ef.compr}\right) \\ \quad \| \text{“OK”} \\ \text{else} \\ \quad \| \text{“Not OK”} \end{array} = \text{“OK”}$$

Edge beams:

$$\begin{array}{l} \text{if } \left(b_{b.f.edge} \le b_{ef.tensile}\right) \wedge \left(b_{t.f.edge} \le b_{ef.compr}\right) \\ \quad \| \text{“OK”} \\ \text{else} \\ \quad \| \text{“Not OK”} \end{array} = \text{“OK”}$$

The chosen flange-widths will not encounter any shear lag nor plate buckling.

- Control of overlapping flanges:

Top flange:

Middle beams:

$$\begin{array}{l} \text{if } b_{t.f} > CC \\ \quad \| \text{“Overlap”} \\ \text{else} \\ \quad \| \text{“No overlap”} \end{array} \quad = \text{“No overlap”}$$

Edge beams:

$$\begin{array}{l} \text{if } b_{t.f.edge} > \dfrac{CC}{2} \\ \quad \| \text{“Overlap”} \\ \text{else} \\ \quad \| \text{“No overlap”} \end{array} \quad = \text{“No overlap”}$$

Bottom flange:

Middle beams:

$$\begin{array}{l} \text{if } b_{b.f} > CC \\ \quad \| \text{“Overlap”} \\ \text{else} \\ \quad \| \text{“No overlap”} \end{array} \quad = \text{“No overlap”}$$

Edge beams:

$$\begin{array}{l} \text{if } b_{b.f.edge} > \dfrac{CC}{2} \\ \quad \| \text{“Overlap”} \\ \text{else} \\ \quad \| \text{“No overlap”} \end{array} \quad = \text{“No overlap”}$$

After finding effective flange width and controlled it for overlapping, the sections may be interpreted as a thin-flanged beam, and the controls needed can be done acc. to NS-EN 1995-1-1.

Only the thin-flanged beams for the middle webs will be controlled, assuming that the edge beams will be ok since they have bigger cross-sections and less moment acting on them. But, the stiffness of the edge beams will be extracted for the purpose of finding a more accurate stiffness of the slab-element.

ULS: Instantaneous

After finding effective flange width and controlled it for overlapping, we may interpret the section as a thin-flanged beam.

Design checks for flanges according to EC5, 9.1.2(7)
Design checks for web according to EC5, 9.1.2(9)

Calculation of stresses according to Annex B in EC5-1-1

- Cross sectional parameters:

2nd moment of area:

Middle beams:

$$I_1 := \frac{1}{12} \cdot b_{t.f} \cdot h_{t.f}^{\ 3} = 3.743 \cdot 10^6$$

$$I_2 := \frac{1}{12} \cdot b_w \cdot h_w^{\ 3} = 3.654 \cdot 10^8$$

$$I_3 := \frac{1}{12} \cdot b_{b.f} \cdot h_{b.f}^{\ 3} = 1.069 \cdot 10^7$$

Edge beams:

$$I_{1.edge} := \frac{1}{12} \cdot b_{t.f.edge} \cdot h_{t.f}^{\ 3}$$

$$I_{2.edge} := \frac{1}{12} \cdot b_{w.edge} \cdot h_w^{\ 3}$$

$$I_{3.edge} := \frac{1}{12} \cdot b_{b.f.edge} \cdot h_{b.f}^{\ 3}$$

Areas:

Middle beams:

$$A_1 := b_{t.f} \cdot h_{t.f} = 2.43 \cdot 10^4$$

$$A_2 := b_w \cdot h_w = 2.673 \cdot 10^4$$

$$A_3 := b_{b.f} \cdot h_{b.f} = 3.447 \cdot 10^4$$

Edge beams:

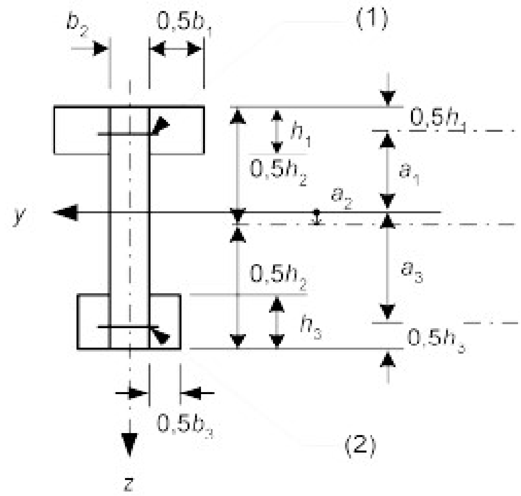
$$A_{1.edge} := b_{t.f.edge} \cdot h_{t.f} = 1.213 \cdot 10^4$$

$$A_{2.edge} := b_{w.edge} \cdot h_w = 5.67 \cdot 10^4$$

$$A_{3.edge} := b_{b.f.edge} \cdot h_{b.f} = 1.72 \cdot 10^4$$

Gamma-values: $\gamma_1 := 1$ $\gamma_2 := 1$ $\gamma_3 := 1$
(Glued interfaces)

Steiner-distances:
Calculated acc. to Annex B in EC5.



The figure shows the Steiner-distances as illustrated in EC5-1-1, Annex B.

Middle beams:

$$a_2 := \frac{\gamma_1 \cdot E_{0.mean.t.f} \cdot A_1 \cdot \left(h_{t.f} + h_w\right) - \gamma_3 \cdot E_{0.mean.b.f} \cdot A_3 \cdot \left(h_{b.f} + h_w\right)}{2 \cdot \left(\left(\gamma_1 \cdot E_{0.mean.t.f} \cdot A_1\right) + \left(\gamma_2 \cdot E_{0.mean.w} \cdot A_2\right) + \left(\gamma_3 \cdot E_{0.mean.b.f} \cdot A_3\right)\right)}$$

$$a_2 = -28.178$$

$$a_1 := \frac{h_w + h_{t.f}}{2} - a_2 = 252.178$$

$$a_3 := \frac{h_w + h_{b.f}}{2} + a_2 = 204.822$$

Edge beams:

$$a_{2.edge} := \frac{\gamma_1 \cdot E_{0.mean.t.f} \cdot A_{1.edge} \cdot \left(h_{t.f} + h_w\right) - \gamma_3 \cdot E_{0.mean.b.f} \cdot A_{3.edge} \cdot \left(h_{b.f} + h_w\right)}{2 \cdot \left(\left(\gamma_1 \cdot E_{0.mean.t.f} \cdot A_{1.edge}\right) + \left(\gamma_2 \cdot E_{0.mean.w} \cdot A_{2.edge}\right) + \left(\gamma_3 \cdot E_{0.mean.b.f} \cdot A_{3.edge}\right)\right)}$$

$$a_{2.edge} = -12.98$$

$$a_{1.edge} := \frac{h_w + h_{t.f}}{2} - a_{2.edge} = 236.98$$

$$a_{3.edge} := \frac{h_w + h_{b.f}}{2} + a_{2.edge} = 220.02$$

Effective bending stiffness:
Equation B.1 in EC5-1-1

Middle beams:

$$EI_{ef.inst1} := E_{0.mean.t.f} \cdot I_1 + \gamma_1 \cdot E_{0.mean.t.f} \cdot A_1 \cdot {a_1}^2 = 1.626 \cdot 10^{13}$$

$$EI_{ef.inst2} := E_{0.mean.w} \cdot I_2 + \gamma_2 \cdot E_{0.mean.w} \cdot A_2 \cdot {a_2}^2 = 5.026 \cdot 10^{12}$$

$$EI_{ef.inst3} := E_{0.mean.b.f} \cdot I_3 + \gamma_3 \cdot E_{0.mean.b.f} \cdot A_3 \cdot {a_3}^2 = 1.529 \cdot 10^{13}$$

$$EI_{ef.inst} := EI_{ef.inst1} + EI_{ef.inst2} + EI_{ef.inst3} = 3.658 \cdot 10^{13}$$

Edge beams:

$$EI_{ef.inst1.edge} := E_{0.mean.t.f} \cdot I_{1.edge} + \gamma_1 \cdot E_{0.mean.t.f} \cdot A_{1.edge} \cdot {a_{1.edge}}^2 = 7.17 \cdot 10^{12}$$

$$EI_{ef.inst2.edge} := E_{0.mean.w} \cdot I_{2.edge} + \gamma_2 \cdot E_{0.mean.w} \cdot A_{2.edge} \cdot {a_{2.edge}}^2 = 1.02 \cdot 10^{13}$$

$$EI_{ef.inst3.edge} := E_{0.mean.b.f} \cdot I_{3.edge} + \gamma_3 \cdot E_{0.mean.b.f} \cdot A_{3.edge} \cdot {a_{3.edge}}^2 = 8.8 \cdot 10^{12}$$

$$EI_{ef.inst.edge} := EI_{ef.inst1.edge} + EI_{ef.inst2.edge} + EI_{ef.inst3.edge} = 2.617 \cdot 10^{13}$$

Effective 2nd moment of inertia:

Middle beams:

$$I_{ef.inst} := \left(I_1 + \left(A_1 \cdot {a_1}^2\right)\right) + \left(I_2 + \left(A_2 \cdot {a_2}^2\right)\right) + \left(I_3 + \left(A_3 \cdot {a_3}^2\right)\right) = 3.392 \cdot 10^9$$

Edge beams:

$$I_{1.ef.edge} := I_{1.edge} + \left(A_{1.edge} \cdot {a_{1.edge}}^2\right)$$

$$I_{2.ef.edge} := I_{2.edge} + \left(A_{2.edge} \cdot {a_{2.edge}}^2\right)$$

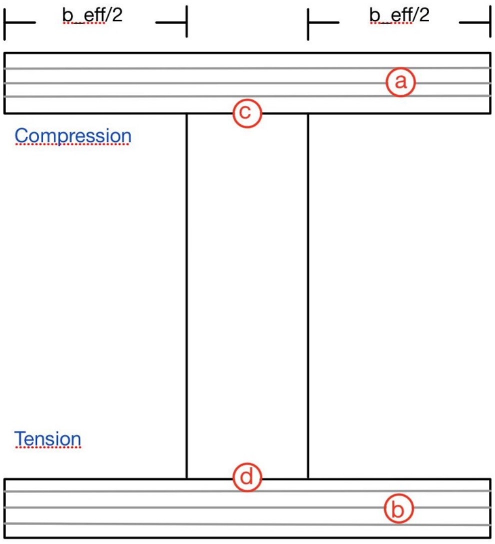
$$I_{3.ef.edge} := I_{3.edge} + \left(A_{3.edge} \cdot {a_{3.edge}}^2\right)$$

$$I_{ef.inst.edge} := I_{1.ef.edge} + I_{2.ef.edge} + I_{3.ef.edge}$$

Now we have the stiffness of the middle beams and edge beams, and from here there will only be done checks for the middle beams.

Calculation of stresses:

Axial stresses:
Eq. B.7 in EC5-1-1



The figure shows the position of the points in which the stresses are found.

Point a) Design compression stress

$$\sigma_1 := \frac{\left(\gamma_1 \cdot E_{0.mean.t.f} \cdot a_1 \cdot M_{Ed.ULS}\right)}{EI_{ef.inst}} + \frac{\left(0.5 \cdot E_{0.mean.t.f} \cdot h_{t.f} \cdot M_{Ed.ULS}\right)}{EI_{ef.inst}} = 3.968$$

Point b) Design tension stress

$$\sigma_3 := \frac{\left(\gamma_1 \cdot E_{0.mean.b.f} \cdot a_3 \cdot M_{Ed.ULS}\right)}{EI_{ef.inst}} + \frac{\left(0.5 \cdot E_{0.mean.b.f} \cdot h_{b.f} \cdot M_{Ed.ULS}\right)}{EI_{ef.inst}} = 3.412$$

Point c) Design bending + axial stress in the web (same as in point d)

$$\sigma_2 := \frac{\left(\gamma_1 \cdot E_{0.mean.w} \cdot a_2 \cdot M_{Ed.ULS}\right)}{EI_{ef.inst}} + \frac{\left(0.5 \cdot E_{0.mean.w} \cdot h_w \cdot M_{Ed.ULS}\right)}{EI_{ef.inst}} = 3.129$$

Shear stress:
(Eq. B.9 in EC5-1-1)

Distance from center of web to the place of zero normal stress:

$$h := \frac{h_w}{2} + a_2$$

$$\tau_{2.max} := \frac{\left(\gamma_3 \cdot E_{0.mean.b.f} \cdot A_3 \cdot a_3 + 0.5 \cdot E_{0.mean.w} \cdot b_w \cdot h^2\right)}{k_{cr.w} \cdot b_w \cdot EI_{ef.inst}} \cdot V_{Ed.ULS} = 1.073$$

<u>Design checks:</u>

Utilizations:

Normal stresses:

Point a)

$$\text{if } \sigma_1 < f_{c.0.d.tf} \;\Vert\; \text{“OK”} \;\text{else}\; \Vert\; \text{“Not OK”} = \text{“OK”}$$

$$\frac{\sigma_1}{f_{c.0.d.tf}} = 0.219$$

Point b)

$$\text{if } \sigma_3 < f_{t.0.d.bf} \;\Vert\; \text{“OK”} \;\text{else}\; \Vert\; \text{“Not OK”} = \text{“OK”}$$

$$\frac{\sigma_3}{f_{t.0.d.bf}} = 0.252$$

Point c)

$$\text{if } \sigma_2 < f_{c.0.d.w} \;\Vert\; \text{“OK”} \;\text{else}\; \Vert\; \text{“Not OK”} = \text{“OK”}$$

$$\frac{\sigma_2}{f_{c.0.d.w}} = 0.184$$

Point d)

$$\text{if } \sigma_2 < f_{t.0.d.w} \;\Vert\; \text{“OK”} \;\text{else}\; \Vert\; \text{“Not OK”} = \text{“OK”}$$

$$\frac{\sigma_2}{f_{t.0.d.w}} = 0.231$$

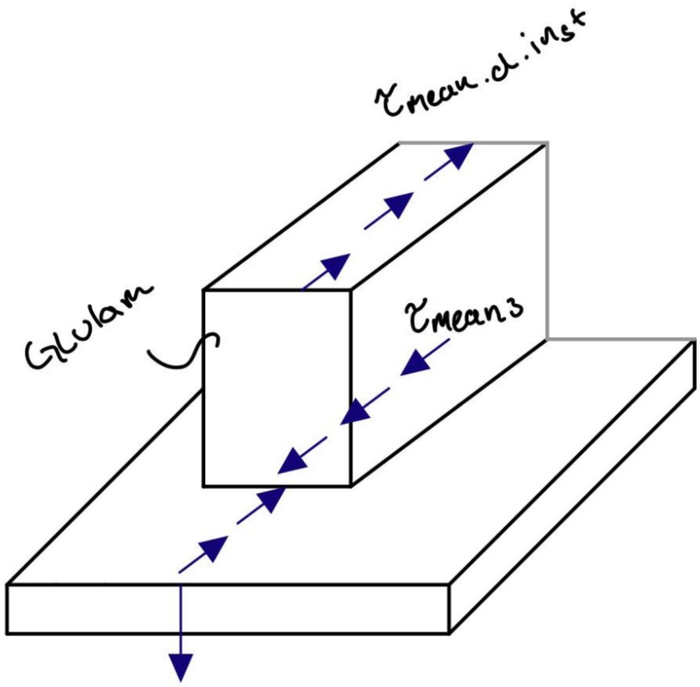
Shear stress:

$$\text{if } \tau_{2.max} < f_{vd.w} \;\Vert\; \text{“OK”} \;\text{else}\; \Vert\; \text{“Not OK”} = \text{“OK”}$$

$$\frac{\tau_{2.max}}{f_{vd.w}} = 0.441$$

Glue-line check:

We assume that the glue itself will be ok, but a check of the shear that arises in the LVL flanges along the grain in the interface will have to be done. The corresponding shear along the grain in the Glulam bottom flange can also be assumed to be ok because it will not be as critical as the shear in the Glulam web, and the strength will be the same for the two cases, fvk. We will only check the top flange because this is thinner than the bottom flange and therefor more critical.



Shear that arises in the flange over the width of the web. This can be found using equation B.5 in EC5, which is equivalent as load on a fastener.

- Shear stress (assumed uniform) in the flange-area over the width of the web:
Acc. to eq. B.5 in EC5

$$\tau_{mean.d.inst} := \frac{E_{0.mean.t.f} \cdot A_1 \cdot a_1}{EI_{ef.inst} \cdot b_w} \cdot V_{Ed.ULS} = 0.633$$

- Flatwise shear strength of LVL:

$$f_{v.0.flat.d.tf} = 0.904$$

- Check acc. to NS-EN 1995-1-1, §9.1.2(6):

Capacity:

$$f_{v.0d.check} := \begin{cases} f_{v.0.flat.d.tf} & \text{if } b_w \le 8 \cdot h_{t.f} \\ f_{v.0.flat.d.tf} \cdot \left(\dfrac{8 \cdot h_{t.f}}{b_w}\right)^{0.8} & \text{else} \end{cases} = 0.904$$

Check:

$$\begin{cases} \text{“OK”} & \text{if } f_{v.0d.check} \ge \tau_{mean.d.inst} \\ \text{“Not OK”} & \text{else} \end{cases} = \text{“OK”}$$

Utilization:

$$\frac{\tau_{mean.d.inst}}{f_{v.0d.check}} = 0.7$$

ULS: Final

$$\begin{array}{l} \text{if } k_{def.t.f} = k_{def.web} \\ \quad \| \text{"Final cond. not needed"} \\ \text{else} \\ \quad \| \text{"Final cond. needed"} \end{array} = \text{"Final cond. needed"}$$

Since the time-dependent factors kdef is not the same between two parts of the composite, we must check the final condition. This means that we must consider the time-dependent effects (such as creep) on the different parts.

Stiffness in final condition:

NS-EN 1990-1-1,
Table A1.1: $\psi_2 := 0.3$

Flanges:

$$E_{mean.fin.1} := \frac{E_{0.mean.t.f}}{\left(1 + \psi_2 \cdot k_{def.t.f}\right)} = 8.077 \cdot 10^3$$

$$E_{mean.fin.3} := \frac{E_{0.mean.b.f}}{\left(1 + \psi_2 \cdot k_{def.b.f}\right)} = 8.077 \cdot 10^3$$

$$G_{mean.fin.1} := \frac{G_{t.f.mean}}{\left(1 + \psi_2 \cdot k_{def.t.f}\right)} = 461.538$$

$$G_{mean.fin.3} := \frac{G_{b.f.mean}}{\left(1 + \psi_2 \cdot k_{def.t.f}\right)} = 461.538$$

Web:

$$E_{mean.fin.2} := \frac{E_{0.mean.w}}{\left(1 + \psi_2 \cdot k_{def.web}\right)} = 1.048 \cdot 10^4$$

$$G_{mean.fin.2} := \frac{G_{web.mean}}{\left(1 + \psi_2 \cdot k_{def.t.f}\right)} = 500$$

<u>Calculation of stresses in final condition:</u>
according to Annex B in EC5-1-1

- Cross sectional parameters:

2nd moment of area:

$$I_1 := \frac{1}{12} \cdot b_{ef.t} \cdot h_{t.f}^{\ 3} = 6.069 \cdot 10^6$$

$$I_2 := \frac{1}{12} \cdot b_w \cdot h_w^{\ 3} = 3.654 \cdot 10^8$$

$$I_3 := \frac{1}{12} \cdot b_{ef.b} \cdot h_{b.f}^{\ 3} = 1.733 \cdot 10^7$$

Areas:

$$A_1 := b_{t.f} \cdot h_{t.f} = 2.43 \cdot 10^4$$

$$A_2 := b_w \cdot h_w = 2.673 \cdot 10^4$$

$$A_3 := b_{b.f} \cdot h_{b.f} = 3.447 \cdot 10^4$$

Gamma-values:
(Glued interfaces)

$\gamma_1 := 1 \qquad \gamma_2 := 1 \qquad \gamma_3 := 1$

- Steiner-distances:
Calculated acc. to Annex B in EC5.

The figure that shows the Steiner-distances as illustrated in EC5-1-1, Annex B can be found in the corresponding instantaneous check.

$$a_2 := \frac{\gamma_1 \cdot E_{mean.fin.1} \cdot A_1 \cdot \left(h_{t.f} + h_w\right) - \gamma_3 \cdot E_{mean.fin.3} \cdot A_3 \cdot \left(h_{b.f} + h_w\right)}{2 \cdot \left(\left(\gamma_1 \cdot E_{mean.fin.1} \cdot A_1\right) + \left(\gamma_2 \cdot E_{mean.fin.2} \cdot A_2\right) + \left(\gamma_3 \cdot E_{mean.fin.3} \cdot A_3\right)\right)}$$

$$a_2 = -27.695$$

$$a_1 := \frac{h_w + h_{t.f}}{2} - a_2 = 251.695$$

$$a_3 := \frac{h_w + h_{b.f}}{2} + a_2 = 205.305$$

Effective bending stiffness:
Acc. to Equation B.1 in EC5-1-1

$$EI_{ef.fin1} := E_{mean.fin.1} \cdot I_1 + \gamma_1 \cdot E_{mean.fin.1} \cdot A_1 \cdot {a_1}^2$$

$$EI_{ef.fin2} := E_{mean.fin.2} \cdot I_2 + \gamma_2 \cdot E_{mean.fin.2} \cdot A_2 \cdot {a_2}^2$$

$$EI_{ef.fin3} := E_{mean.fin.3} \cdot I_3 + \gamma_3 \cdot E_{mean.fin.3} \cdot A_3 \cdot {a_3}^2$$

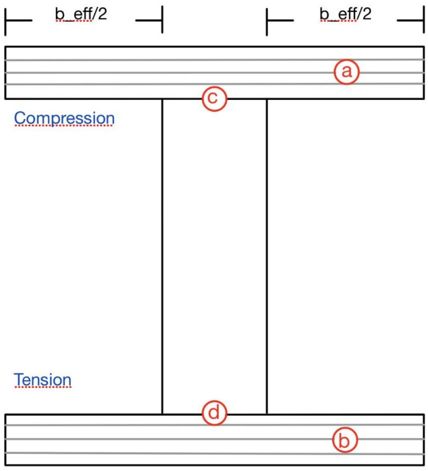
$$EI_{ef.fin} := EI_{ef.fin1} + EI_{ef.fin2} + EI_{ef.fin3} = 2.84 \cdot 10^{13}$$

Effective 2nd moment of inertia:

$$I_{ef.fin} := \left(I_1 + \left(A_1 \cdot {a_1}^2\right)\right) + \left(I_2 + \left(A_2 \cdot {a_2}^2\right)\right) + \left(I_3 + \left(A_3 \cdot {a_3}^2\right)\right) = 3.401 \cdot 10^9$$

- Calculation of stresses:

Axial stresses:
(Eq. B.7 in EC5-1-1)



The figure shows the position of the points in which the stresses are found.

Point a) Design compression stress

$$\sigma_1 := \frac{\left(\gamma_1 \cdot E_{mean.fin.1} \cdot a_1 \cdot M_{Ed.ULS}\right)}{EI_{ef.fin}} + \frac{\left(0.5 \cdot E_{mean.fin.1} \cdot h_{t.f} \cdot M_{Ed.ULS}\right)}{EI_{ef.fin}} = 3.925$$

Point b) Design tension stress

$$\sigma_3 := \frac{\left(\gamma_3 \cdot E_{mean.fin.3} \cdot a_3 \cdot M_{Ed.ULS}\right)}{EI_{ef.fin}} + \frac{\left(0.5 \cdot E_{mean.fin.3} \cdot h_{b.f} \cdot M_{Ed.ULS}\right)}{EI_{ef.fin}} = 3.388$$

Point c) Design bending + axial stress in the web (same as in point d)

$$\sigma_2 := \frac{\left(\gamma_2 \cdot E_{mean.fin.2} \cdot a_2 \cdot M_{Ed.ULS}\right)}{EI_{ef.fin}} + \frac{\left(0.5 \cdot E_{mean.fin.2} \cdot h_w \cdot M_{Ed.ULS}\right)}{EI_{ef.fin}} = 3.26$$

Shear stress:
(Eq. B.9 in EC5-1-1)

Distance from center of web to the place of zero normal stress:

$$h := \frac{h_w}{2} + a_2$$

$$\tau_{2.max} := \frac{\left(\gamma_3 \cdot E_{mean.fin.3} \cdot A_3 \cdot a_3 + 0.5 \cdot E_{mean.fin.2} \cdot b_w \cdot h^2\right)}{k_{cr.w} \cdot b_w \cdot EI_{ef.fin}} \cdot V_{Ed.ULS} = 1.074$$

Design checks: Utilizations:

- Normal stresses:

Point a)

$$\text{if } \sigma_1 < f_{c.0.d.tf} \;\; \| \text{“OK”} \;\; \text{else} \;\; \| \text{“Not OK”} = \text{“OK”}$$

$$\frac{\sigma_1}{f_{c.0.d.tf}} = 0.217$$

Point b)

$$\text{if } \sigma_3 < f_{t.0.d.bf} \;\; \| \text{“OK”} \;\; \text{else} \;\; \| \text{“Not OK”} = \text{“OK”}$$

$$\frac{\sigma_3}{f_{t.0.d.bf}} = 0.25$$

Point c)

$$\text{if } \sigma_2 < f_{c.0.d.w} \;\; \| \text{“OK”} \;\; \text{else} \;\; \| \text{“Not OK”} = \text{“OK”}$$

$$\frac{\sigma_2}{f_{c.0.d.w}} = 0.191$$

Point d)

$$\text{if } \sigma_2 < f_{t.0.d.w} \;\; \| \text{“OK”} \;\; \text{else} \;\; \| \text{“Not OK”} = \text{“OK”}$$

$$\frac{\sigma_2}{f_{t.0.d.w}} = 0.24$$

-Shear stress

$$\text{if } \tau_{2.max} < f_{vd.w} \;\; \| \text{“OK”} \;\; \text{else} \;\; \| \text{“Not OK”} = \text{“OK”}$$

$$\frac{\tau_{2.max}}{f_{vd.w}} = 0.441$$

Glue-line check:

We assume that the glue itself will be ok, but a check of the shear that arises in the LVL flanges along the grain in the interface will have to be done. The corresponding shear along the grain in the Glulam bottom flange can also be assumed to be ok because it will not be as critical as the shear in the Glulam web, and the strength will be the same for the two cases, fvk.

- Shear stress (assumed uniform) in the flange-area over the width of the web:
Acc. to eq. B.5 in EC5

$$\tau_{mean.d.fin} := \frac{E_{mean.fin.1} \cdot A_1 \cdot a_1}{EI_{ef.inst} \cdot k_{cr.w} \cdot b_w} \cdot V_{Ed.ULS} = 0.608$$

- Flatwise shear strength of LVL:

$$f_{v.0.flat.d.tf} = 0.904$$

- Check acc. to NS-EN 1995-1-1, §9.1.2(6):

Capacity:

$$f_{v.90d.check} := \begin{cases} f_{v.0.flat.d.tf} & \text{if } b_w \le 8 \cdot h_{t.f} \\ f_{v.0.flat.d.tf} \cdot \left(\frac{8 \cdot h_{t.f}}{b_w}\right)^{0.8} & \text{else} \end{cases} = 0.904$$

Check:

$$\begin{cases} \text{“OK”} & \text{if } f_{v.90d.check} \ge \tau_{mean.d.fin} \\ \text{“Not OK”} & \text{else} \end{cases} = \text{“OK”}$$

Utilization:

$$\frac{\tau_{mean.d.fin}}{f_{v.90d.check}} = 0.672$$

SLS: Instantaneous deformation

- Bending deformation: $$w_{inst.bending} := \frac{5}{384} \cdot \frac{q_{Ed.SLS1} \cdot l^4}{EI_{ef.inst}} = 7.349$$

-Shear deformation acc. to Timoshenko beam theory:

Shear correction factor: $\kappa := 0.83$

The shear correction factor is for rectangular cross sections, since we assume only web takes shear.

Shear stiffness:

$$S_T := \left(G_{t.f.mean} \cdot b_{ef.t} \cdot h_{t.f}\right) + \left(G_{web.mean} \cdot b_w \cdot h_w\right) + \left(G_{b.f.mean} \cdot b_{ef.b} \cdot h_{b.f}\right) = 7.453 \cdot 10^7$$

The shear deformation becomes:

$$w_{inst.shear} := q_{Ed.SLS1} \cdot \frac{l^2}{8} \cdot \frac{1}{\kappa \cdot S_T} = 0.577$$

-Total deformation:
Acc. to Timoshenko beam theory

$$w_{inst} := w_{inst.bending} + w_{inst.shear} = 7.926$$

- Allowed deformation: $w_{max} := \frac{l}{500} = 17$ (most conservative demand)
(Acc. to EC5-1-1, table 7.2)

$$\left. \begin{array}{l} \text{if } w_{inst} < w_{max} \\ \quad \text{"Ok"} \\ \text{else} \\ \quad \text{"Not ok"} \end{array} \right| = \text{"Ok"}$$

Utilization:

$$\frac{w_{inst}}{w_{max}} = 0.466$$

The instantaneous deformation is ok.

SLS: Final deformation

Since the time-property factors are not the same, we must use the elasticity-modulus for the final condition and then calculate the deformation in the same way as for instantaneous.

SLS characteristic load: $q_{Ed.SLS1} = 3.955$

- Bending deformation:
(formula from handbooks)

$$w_{fin.bending} := \frac{5}{384} \cdot \frac{q_{Ed.SLS1} \cdot l^4}{EI_{ef.fin}} = 9.466$$

-Shear deformation:

Shear correction factor: $\kappa := 0.83$

Shear stiffness:

$$S_T := \left(G_{mean.fin.1} \cdot b_{ef.t} \cdot h_{t.f}\right) + \left(G_{mean.fin.2} \cdot b_w \cdot h_w\right) + \left(G_{mean.fin.3} \cdot b_{ef.b} \cdot h_{b.f}\right) = 5.733 \cdot 10^7$$

Shear deformation:

$$w_{fin.shear} := q_{Ed.SLS1} \cdot \frac{l^2}{8} \cdot \frac{1}{\kappa \cdot S_T} = 0.751$$

-Total deformation: $w_{fin} := w_{fin.bending} + w_{fin.shear} = 10.216$

- Allowed deformation:
Acc. to EC5-1-1, table 7.2

$$w_{max} := \frac{l}{500} = 17$$

if $w_{fin} < w_{max}$
    "Ok"
else
    "Not ok"
= "Ok"

Utilization:

$$\frac{w_{fin}}{w_{max}} = 0.601$$

The final deformation is ok.

SLS: Vibration check
(Human induced vibr.)

EN1995-1-1, §7.3.1.(1)P: "It shall be ensured that the actions which can be reasonably anticipated on a member, component or structure, do not cause vibrations that can impair the function of the structure or cause unacceptable discomfort to the users."

Weight of the deck: $\gamma := 2$ $\left[\frac{kN}{m^2}\right]$

Dead-load: $q_{dead} := \gamma \cdot CC \cdot 10^{-3} = 1.13$ $\left[\frac{N}{mm}\right]$

Gravitational acceleration: $g := 9.81$ $\left[\frac{m}{s^2}\right]$

Mass: $m := \frac{q_{dead}}{g} = 0.115$ $\left[\frac{kg}{mm}\right]$

We calculate the mass based on the dead-load (EC5-1-1, §7.3)

No. of beam elements per meter: $n_{beams} := \frac{1000}{CC} = 1.77$

Equivalent bending stiffness: $EI_L := EI_{ef.inst} = 3.658 \cdot 10^{13}$ $\left[\frac{Nmm^2}{m}\right]$

Fundamental frequency: $f_{n.1} := \frac{\pi}{2 \cdot l^2} \cdot \sqrt[2]{\frac{EI_L}{m \cdot 10^{-3}}} = 12.252$ $[Hz]$

EN1995-1-1, §7.3.3.(1):

$$\begin{array}{l} \text{if } f_{n.1} > 8 \\ \quad \Big\| \text{“Simplified method allowed”} \\ \text{else} \\ \quad \Big\| \text{“Special investigation needed”} \end{array} \Bigg| = \text{“Simplified method allowed”}$$

1kN static deflection: $w_{static.1kN} := \dfrac{1000 \cdot l^3}{48 \cdot EI_L} = 0.35$

Hu & Chui criterion:

$$\begin{array}{l} \text{if } \dfrac{\left(\dfrac{f_{n.1}}{18.7}\right)^{2.27}}{w_{static.1kN}} \geq 1 \\ \quad \Big\| \text{“OK”} \\ \text{else} \\ \quad \Big\| \text{“Not OK”} \end{array} \Bigg| = \text{“OK”}$$

$$\frac{\left(\dfrac{f_{n.1}}{18.7}\right)^{2.27}}{w_{static.1kN}} = 1.095$$

By using the quasi-permanent load combination one would obtain more vibrations.

Effective stiffness to be used for the modelling of the shell elements: (using the instantaneous values)

Width of deck: $b = 2.4 \cdot 10^{3}$

Number of glulam elements over the width of the deck:

Middle beams: $n_{mid} := 3$

Edge beams: $n_{edge} := 2$

Total bending stiffness of the deck:

$$EI_{ef.tot.inst} := \left(EI_{ef.inst} \cdot n_{mid}\right) + \left(EI_{ef.inst.edge} \cdot n_{edge}\right) = 1.621 \cdot 10^{14}$$

Total 2nd moment of area of the deck:

$$I_{ef.tot.inst} := \left(I_{ef.inst} \cdot n_{mid}\right) + \left(I_{ef.inst.edge} \cdot n_{edge}\right) = 1.479 \cdot 10^{10}$$

Total elasticity modulus of the deck (grain direction):

$$E_{inst.1} := \frac{EI_{ef.tot.inst}}{I_{ef.tot.inst}} = 1.096 \cdot 10^{4}$$

Note: The stiffness values extracted here (both for instantaneous and final condition) are for the grain direction (direction 1). To get the values in the direction perpendicular to the grain, we may divide E1 by 4.

Total elasticity modulus of the deck (perpendicular to grain direction):

$$E_{inst.2} := \frac{E_{inst.1}}{4} = 2.74 \cdot 10^{3}$$

C2. Connection Calculations

Part 1 consists of the data needed for the calculation
Part 2 is the ULS checks.
Part 3 is the calculation of stiffness for the given configuration.

Formulas for structural connection design has been extrated from NS-EN 1995-1-1.

Units used in script:

- Dimensions/lengths: $[mm]$
- Forces: $[N]$
- Moments: $[Nmm]$
- Stresses/strengths: $\left[\dfrac{N}{mm^2}\right]$
- Areas: $\left[mm^2\right]$
- 2nd moment of inertia: $\left[mm^4\right]$
- Densities: $\left[\dfrac{kg}{m^3}\right]$

PART 1)

Geometry:

The geometry of the connection will now be presented.
The column is assigned the index 1, while the diagonal has the index 2.

Assumed dimensions of column and diagonal:

$b_{col} := 765$ $\quad h_{col} := 765$

$b_{diag} := 540$ $\quad h_{diag} := 585$

Lengths of members: $L_1 := 5000$

$L_2 := 11650$

Angle between diagonal and column: $\alpha := 42\ deg$

Diameter of all the dowels: $d := 12$

Angle between diagonal force and grain: $\alpha_1 := \alpha$

$\alpha_2 := 0$

Number of dowels: $n_{dowels.1} := 35$

$n_{dowels.2} := 66$

Number of rows: $n_{rows.1} := 5$

$n_{rows.2} := 6$

Number of steel plates in column and diagonal: $n_{plates} := 4$

Thickness of steel plates: $t_{plate} := 16$

Classification of steel plates according to NS-EN 1995-1-1, §8.2.3(1):

$$Plate\_id := \begin{array}{l} \text{if } t_{plate} \le 0.5 \cdot d \\ \quad \| \text{“Thin plate”} \\ \text{else if } t_{plate} \ge d \\ \quad \| \text{“Thick plate”} \\ \text{else} \\ \quad \| \text{“Not clear”} \end{array} = \text{“Thick plate”}$$

Note: For multiple internal slotted-in steel plate connections we can always assume to use thick plates.

Factors:

-Partial factors:

For glulam: $\gamma_{M.GL} := 1.15$

For connections: $\gamma_{M.con} := 1.3$

- Modification factors:

Factor for medium-duration loading: (EC5, Table 3.1) $k_{mod} := 1.1$ (since wind is included, ULS COMB1)

Bearing factor: (EC5, eq. (8.33)) $k_{90} := 1.35 + (0.015 \cdot d) = 1.53$

Material properties:

-Column and diagonal:

Density: $\rho_{m.1} := 480$

$\rho_{m.2} := 480$

$\rho_m := \sqrt[2]{\rho_{m.1} \cdot \rho_{m.2}} = 480$

Area: $A_1 := h_{col} \cdot b_{col}$

$A_2 := h_{diag} \cdot b_{diag}$

Strength: $f_{c.0.k} := 24.5$

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

$f_{v.k} := 3.5$

Youngs modulus: $E_{0.mean} := 13600$

2nd moment of area: $I_{col} := \frac{1}{12} \cdot b_{col} \cdot h_{col}{}^3 = 2.854 \cdot 10^{10}$

$I_{diag} := \frac{1}{12} \cdot b_{diag} \cdot h_{diag}{}^3 = 9.009 \cdot 10^{9}$

Embedment strength of timber:

(EC5, eq. 8.32) $f_{h.0.k} := 0.082 \cdot (1 - 0.01 \cdot d) \cdot \rho_m = 34.637$

(EC5, eq. 8.31) $f_{h.\alpha.k} := \frac{f_{h.0.k}}{k_{90} \cdot (\sin(\alpha))^2 + (\cos(\alpha))^2} = 27.994$

Column: $f_{h.1.k.col} := f_{h.\alpha.k} = 27.994$

$f_{h.2.k.col} := f_{h.\alpha.k} = 27.994$

$\beta_{col} := \frac{f_{h.2.k.col}}{f_{h.1.k.col}} = 1$

Diagonal: $f_{h.1.k.diag} := f_{h.0.k} = 34.637$

$f_{h.2.k.diag} := f_{h.0.k} = 34.637$

Ratio: $\beta_{diag} := \frac{f_{h.2.k.diag}}{f_{h.1.k.diag}} = 1$

-Fasteners (dowels):

Tensile strength: $f_{uk} := 650$

Yield moment: $M_{y.Rk} := 0.3 \cdot f_{uk} \cdot d^{2.6} = 1.247 \cdot 10^{5}$
(EC5, eq. 8.30)

- Steel plates:

Elasticity modulus: $E_s := 210000$

Safety factor: $\gamma_0 := 1.15$ $\gamma_{M2} := 1.25$

Dimensions: $l_{plate.1} := 0.8 \cdot h_{col}$ $b_{plate.1} := 0.8 \cdot b_{diag}$

$l_{plate.2} := 0.8 \cdot h_{diag}$ $b_{plate.2} := b_{plate.1}$

The dimensions of the steel plates are assumed to be 80% of the cross-sectional heights/widths as a conservative simplification.

Yielding strength: $f_y := 355$

Fracture strength: $f_u := 510$ (Stålprofiler håndbok)

$f_{ub} := 0.9 \cdot f_u = 459$

Design loads:

- Diagonal: $F_{diag} := 2385 \cdot 10^3$

- Column: $F_{col} := 4000 \cdot 10^3$

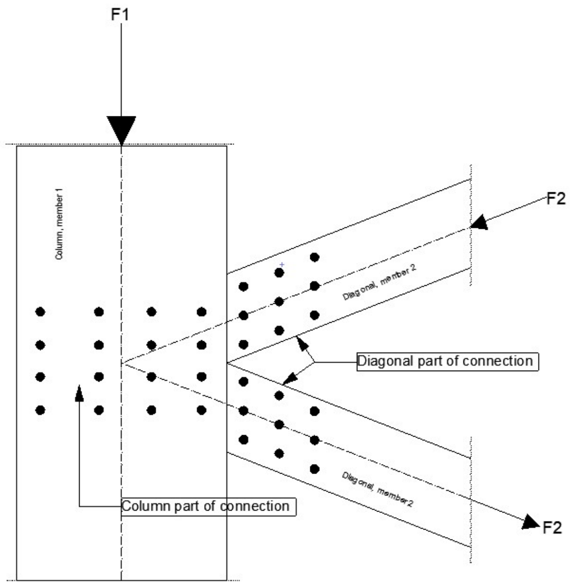
- Total load in column connection:

$$F_{Ed.col} := F_{diag} \cdot \sin(\alpha) = 1.596 \cdot 10^6$$

- Total load in diagonal connection:

$$F_{Ed.diag} := F_{diag} = 2.385 \cdot 10^6$$

In the connection it is assumed to be no eccentricities



Concentric connection

PART 2)
Capacity of Connection

In this part the ULS checks of the connection will be presented.
First the column part of the connection, then the diagonal part.

Column part:

Embedment strength:

Acc. to NS-EN 1995-1-1, §8.2

In connections with multiple shear planes the load-carrying capacity is determined by assuming that the external members are in single shear and the middle members in double shear. The total load-carrying capacity is determined by adding the contributions of compatible failure modes. Some of the failure modes cannot occur simultaneously due to deformation compatibility, meaning that they occur at different deformation levels: either small ('brittle') or large ('ductile').

Number of middle members: $n_{mid} := n_{plates} - 1 = 3$

Total nr. of shear planes for middle members: $n_{sp.mid} := n_{mid} \cdot 2 = 6$

Total nr. of shear planes for outer members: $n_{sp.out} := 2$

Thickness of middle members: $t_2 := \dfrac{b_{col}}{\left(n_{plates} + 1\right)} = 153$

Thickness of outer members: $t_1 := \dfrac{\left(b_{col} - \left(n_{mid} \cdot t_2\right) - \left(n_{rows.1} \cdot t_{plate}\right)\right)}{2} = 113$

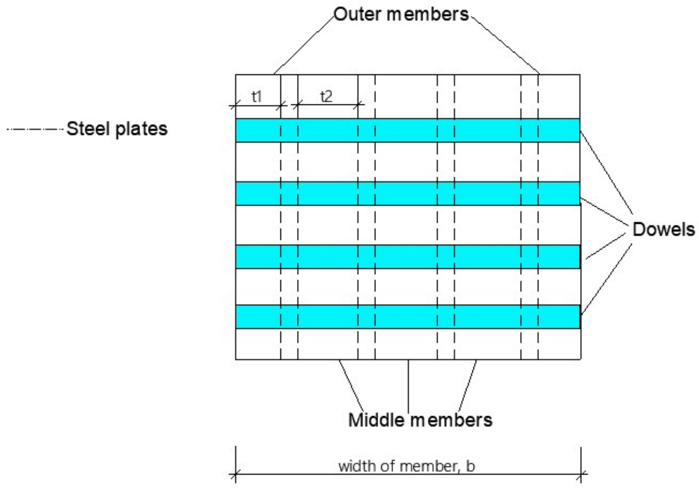


Figure: Section showing the outer and middle members of a connection with multiple slotted-in steel plates and dowels. Representative for both the column part and the diagonal part of the connection.

Outer members:

Formulas for single shear failure modes, NS-EN 1995-1-1, §8.2.3. Only thick plate modes considered.

Failure mode c) $F_{v.Rk.c} := f_{h.1.k.col} \cdot t_1 \cdot d$

Failure mode d) $F_{v.Rk.d} := f_{h.1.k.col} \cdot t_1 \cdot d \cdot \left(\sqrt[2]{2 + \frac{4 \cdot M_{y.Rk}}{f_{h.1.k.col} \cdot d \cdot {t_1}^2}} - 1 \right)$

Failure mode e) $F_{v.Rk.e} := 2.3 \cdot \sqrt[2]{M_{y.Rk} \cdot f_{h.1.k.col} \cdot d}$

We note that dowels have no axial capacity. Therefor, no rope effect is included in the transverse capacity.

Total capacity per fastener per shear plane for outer members:

$$F_{v.Rk.outer} := min\left(F_{v.Rk.c}, F_{v.Rk.d}, F_{v.Rk.e}\right) = 1.489 \cdot 10^4$$

<u>Middle members:</u>

Formulas for double shear failure modes with external plates, NS-EN 1995-1-1, §8.2.3. Only thick plate modes considered.

Failure mode f) $F_{v.Rk.f} := f_{h.1.k.col} \cdot t_1 \cdot d$

Failure mode g) $F_{v.Rk.g} := f_{h.1.k.col} \cdot t_1 \cdot d \cdot \left(\sqrt[2]{2 + \frac{4 \cdot M_{y.Rk}}{f_{h.1.k.col} \cdot d \cdot {t_1}^2}} - 1 \right)$

Failure mode h) $F_{v.Rk.h} := 2.3 \cdot \sqrt[2]{M_{y.Rk} \cdot f_{h.1.k.col} \cdot d}$

Failure mode l) $F_{v.Rk.l} := 0.5 \cdot f_{h.2.k.col} \cdot t_2 \cdot d$

Failure mode m) $F_{v.Rk.m} := 2.3 \cdot \sqrt[2]{M_{y.Rk} \cdot f_{h.2.k.col} \cdot d}$

We note that dowels have no axial capacity. Therefor, no rope effect is included in the transverse capacity.

Total capacity <u>per fastener per shear plane</u> for middle members:

$$F_{v.Rk.middle} := min\left(F_{v.Rk.f}, F_{v.Rk.g}, F_{v.Rk.h}, F_{v.Rk.l}, F_{v.Rk.m}\right) = 1.489 \cdot 10^4$$

<u>Total capacity per fastener per shear plane:</u>

We can only combine compatible failure modes.
For the practical purpose of being able to program the comparison, the failure modes will be identified as 1, 2, 3 etc. instead of a, b ,c etc.

Failure mode for middle members will hereafter be named "mode_mid".
Failure mode for outer members will hereafter be named "mode_outer".
It must also be noted that $\wedge$ is the logical operator "AND", and $\vee$ is the logical operator "OR", which will both be used in the if-else-statements below.

Failure mode for middle members:

$$
mode\_mid := \left. \begin{array}{l} \text{if } F_{v.Rk.middle} = F_{v.Rk.f} \\ \quad \Vert\ 6 \\ \text{else if } F_{v.Rk.middle} = F_{v.Rk.g} \\ \quad \Vert\ 7 \\ \text{else if } F_{v.Rk.middle} = F_{v.Rk.h} \\ \quad \Vert\ 8 \\ \text{else if } F_{v.Rk.middle} = F_{v.Rk.l} \\ \quad \Vert\ 12 \\ \text{else if } F_{v.Rk.middle} = F_{v.Rk.m} \\ \quad \Vert\ 13 \end{array} \right| = 8
$$

Failure mode for outer members:

$$
mode\_outer := \left. \begin{array}{l} \text{if } F_{v.Rk.outer} = F_{v.Rk.c} \\ \quad \Vert\ 3 \\ \text{else if } F_{v.Rk.outer} = F_{v.Rk.d} \\ \quad \Vert\ 4 \\ \text{else if } F_{v.Rk.outer} = F_{v.Rk.e} \\ \quad \Vert\ 5 \end{array} \right| = 5
$$

Comaptibility check:
Acc. to EC5, §8.1.3.2

Now we must check if the failure mode for outer members are the same type as for the middle members. We can do this through an if-else statement, as shown below.

$$
Failure := \begin{array}{l} \text{if } (mode\_outer = 3) \wedge ((mode\_mid = 6) \vee (mode\_mid = 12)) \\ \quad \| \text{“Brittle modes”} \\ \text{else if } ((mode\_outer = 4) \vee (mode\_outer = 5)) \wedge ((mode\_mid = 7) \vee (mode\_mid = 8) \vee (mode\_mid = 13)) \\ \quad \| \text{“Ductile modes”} \\ \text{else} \\ \quad \| \text{“Incompatible modes”} \end{array}
$$

$$Failure = \text{“Ductile modes”}$$

As we can see, both the outer failure modes and the inner failure modes are ductile, which is what we want. In situations where both ductile and brittle types are possible it is good practice to try to ensure that the design condition is based on the ductile failure mechanism. Therefor, we will not proceed until ductile compatibility is achieved in the above code.

Total capacity per fastener for the column part is:

Shear planes for outer members: $n_{sp.out} = 2$

Shear planes for middle members: $n_{sp.mid} = 6$

$$F_{v.Rk.col} := n_{sp.out} \cdot F_{v.Rk.outer} + n_{sp.mid} \cdot F_{v.Rk.middle} = 1.191 \cdot 10^5$$

Capacity per fastener in ULS:

$$F_{v.Rd.col} := \frac{F_{v.Rk.col}}{\gamma_{M.con}} \cdot \left(k_{mod}\right) = 1.008 \cdot 10^5$$

Amount of dowels needed in the column part of the connection:

$$n_{col} := \left(\frac{F_{Ed.col}}{F_{v.Rd.col}}\right) = 15.836$$

Check if the chosen amount of dowels (in the start of the script) is sufficient:

$$\begin{array}{l|l} \text{if } n_{dowels.1} \geq n_{col} & = \text{“OK”} \\ \quad \Vert \text{ “OK”} & \\ \text{else} & \\ \quad \Vert \text{ “Must increase nr. of dowels”} & \end{array}$$

Chosen configuration for column part (member 1):

$a_{1.col} := 100$ $a_{2.col} := 141$

$a_{3.t.col} := 1000$ $a_{3.c.col} := 1000$

$a_{4.t.col} := 1000$ $a_{4.c.col} := 100$

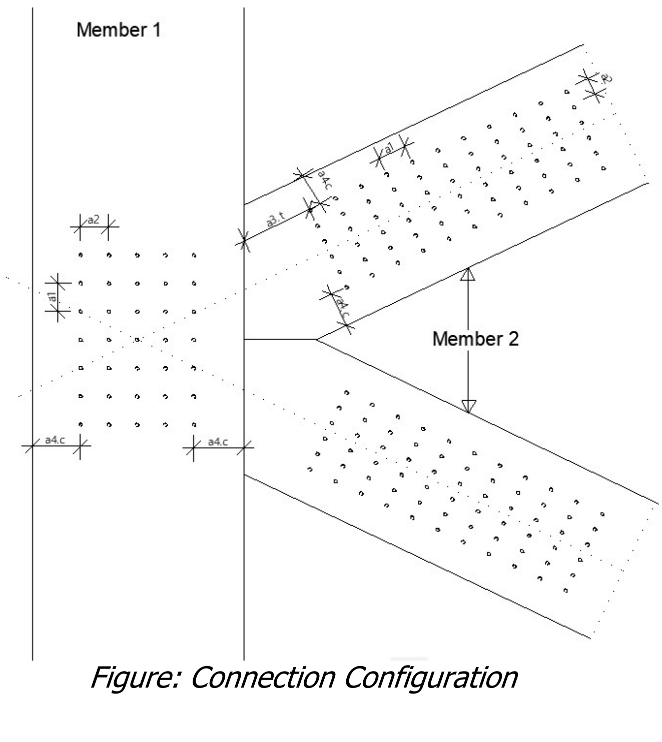


Figure: Connection Configuration

The a3 distances (and a4.t) are actually bigger, but there is no point in measuring them because they will satisfy the distance-demand, as seen in the figure. Therefor a random number (which is big enough to make the code run) has been implemented.

Check of minimum distances:

The minimum spacings given in EC5 have been derived to prevent splitting failure when connection is subjected to lateral load. With too small spacings we get increasing tension perpendicular to grain.

EC5, table 8.5:

$$\text{if } a_{1.col} \geq (3+(2 \cdot \cos(\alpha))) \cdot d \;\Vert\; \text{“OK”} = \text{“OK”}$$

$$\text{if } a_{2.col} \geq 3 \cdot d \;\Vert\; \text{“OK”} = \text{“OK”}$$

$$\text{if } a_{3.t.col} \geq \max(7 \cdot d, 80) \;\Vert\; \text{“OK”} = \text{“OK”}$$

$$\text{if } a_{3.c.col} \geq \max(\sin(\alpha) \cdot d, 3 \cdot d) \;\Vert\; \text{“OK”} = \text{“OK”}$$

$$\text{if } a_{4.t.col} \geq \max((2+2 \cdot \sin(\alpha)) \cdot d, 3 \cdot d) \;\Vert\; \text{“OK”} = \text{“OK”}$$

$$\text{if } a_{4.c.col} \geq 3 \cdot d \;\Vert\; \text{“OK”} = \text{“OK”}$$

All the minimum distances are fulfilled!

Splitting check: (Parallell to grain)

a) The first thing that needs to be sorted is the distances between the fasteners. This has been verified.

b) The second thing that must be verified is that the effective number of fasteners in a row has sufficient capacity to carry the load parallell to grain. This will be controlled in accordance with NS-EN 1995-1-1, §8.1.2(5):

Total amount of fasteners in one row in grain direction:

$$n_{row.col} := \frac{n_{dowels.1}}{n_{rows.1}} = 7$$

The effective number of fasteners in one row in grain direction:

EC5, eq. (8.34) $$n_{ef.row.col} := min\left(n_{row.col}, n_{row.col}^{0.9} \cdot \sqrt[4]{\frac{a_{1.col}}{13 \cdot d}}\right) = 5.156$$

The effective load-carrying capacity of each row then becomes:

$$F_{v.ef.Rk.col} := F_{v.Rk.col} \cdot n_{ef.row.col}$$

For entire connection:

$$F_{v.ef.Rk.col.tot} := F_{v.ef.Rk.col} \cdot n_{rows.1} = 3.07 \cdot 10^{6}$$

Design capacity in ULS:

$$F_{v.ef.Rd.col} := \frac{F_{v.ef.Rk.col.tot} \cdot k_{mod}}{\gamma_{M.con}} = 2.598 \cdot 10^{6}$$

Control of splitting parallell to grain:

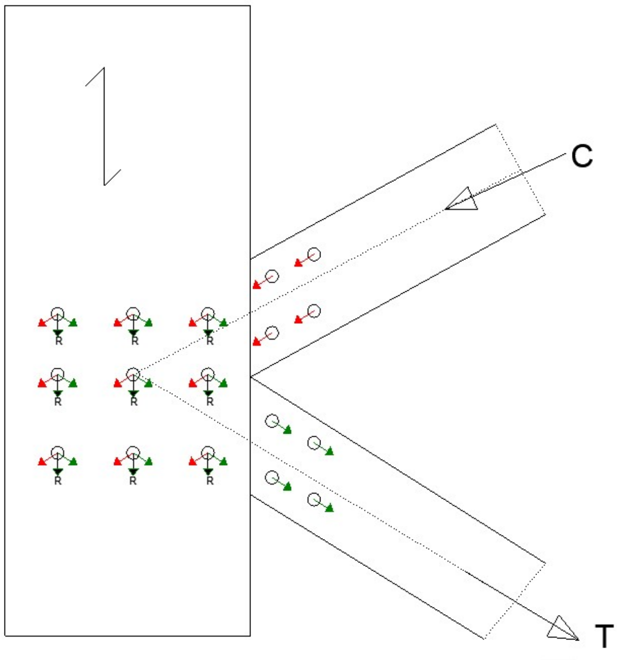
if $F_{Ed.col} \le F_{v.ef.Rd.col}$ = "OK"
  "OK"
else
  "Not OK"

Utility:

$$\frac{F_{Ed.col}}{F_{v.ef.Rd.col}} = 0.614$$

Splitting check: (Perpendicular to grain)

Since the connection considered has two diagonals hitting the column it means that there will be one compression force from one of the diagonals, and a tension force from the other. Looking at the force resultants on the dowels from the diagonal forces, we see that they point in the direction parallell to grain. This means that there will not be any forces perpendicular to grain, and we will not have to check it.



Demonstration of resultant force from diagonal forces

Control of compression of net cross section:

Net area: $A_{net.col} := A_1 - \left(n_{plates} \cdot h_{col} \cdot t_{plate}\right) - \left(d \cdot \frac{n_{dowels.1}}{n_{rows.1}} \cdot b_{col}\right) = 4.72 \cdot 10^5$

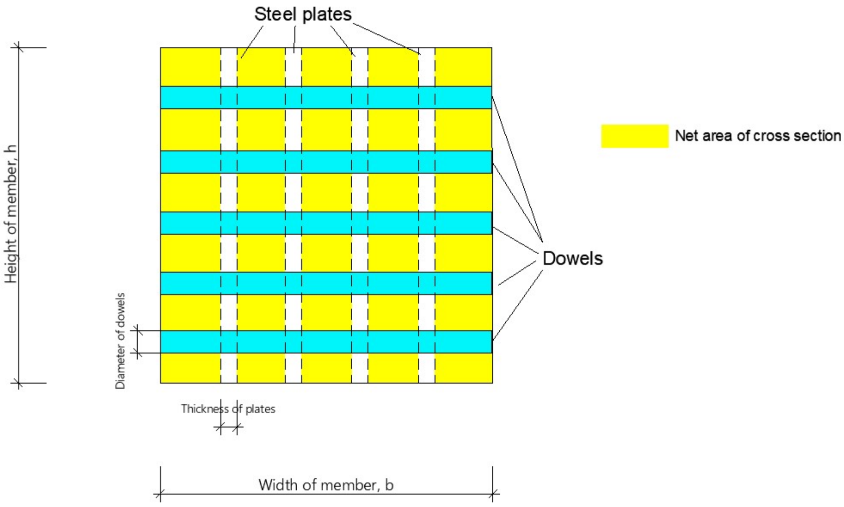


Figure: Net area of cross section.

Strength: $f_{c.0.d} := \frac{f_{c.0.k} \cdot k_{mod}}{\gamma_{M.GL}} = 23.435$

Compr. stress: $\sigma_{compr.col} := \frac{F_{col}}{A_{net.col}} = 8.474$

Utility: $u := \frac{\sigma_{compr.col}}{f_{c.0.d}} = 0.362$

Check:
```
if u ≤ 1
  || "OK"
else
  || "Not OK"
                 = "OK"
```

Block failure:
Acc. to EC5, §8.2.3(5)

For dowel type connections we should verify that block failure will not arise, by use of Annex A in EC5.

For the column part of the connection this will not be a problem since there is no tension force.

Check of steel plates:
Acc. to EC5, 8.2.3(2)

The capacity of the steel plates will be found according to EC3-1-1.

- Cross section classification:

Acc. to EC3-1-1, table 5.2

Factor: $\varepsilon := 0.81$

Length: $C := b_{plate.1} = 432$

Thickness: $t := t_{plate} = 16$

Slenderness: $\lambda := \dfrac{C}{t \cdot \varepsilon} = 33.333$

Classification:

$$\begin{array}{l} \text{if } \lambda \le 33 \\ \quad \text{"Class 1"} \\ \text{else if } \lambda \le 38 \\ \quad \text{"Class 2"} \\ \text{else if } \lambda \le 42 \\ \quad \text{"Class 3"} \\ \text{else} \\ \quad \text{"Class 4"} \end{array} = \text{"Class 2"}$$

- Compression check:

Net area of steel plates: $A_{plate.net} := t \cdot \left(b_{plate.1} - n_{rows.1} \cdot d\right)$

Capacity of steel plates: $N_{c.Rd} := A_{plate.net} \cdot \dfrac{f_y}{\gamma_0} = 1.837 \cdot 10^6$

Control of compression capacity acc. to EC3-1-1, §6.2.4

$$\begin{array}{l} \text{if } n_{plates} \cdot N_{c.Rd} \geq F_{Ed.col} \\ \quad \text{“OK”} \\ \text{else} \\ \quad \text{“Not OK”} \end{array} = \text{“OK”}$$

Utility: $u := \dfrac{F_{Ed.col}}{n_{plates} \cdot N_{c.Rd}} = 0.217$

The compression capacity of the steel plates in the column is OK.

- Minimum distances:
Acc. to EC3-1-8, table 3.3

Edge distances: $e_1 := a_{3.t.col} = 1 \cdot 10^3$

$e_2 := a_{4.t.col} = 1 \cdot 10^3$

Spacings: $p_1 := a_{1.col} = 100$

$p_2 := a_{2.col} = 141$

Hole width for dowels: $d_0 := d + 2 = 14$

Checks:

$$\begin{array}{l} \text{if } e_1 \geq 1.2 \cdot d_0 \\ \quad \| \text{"OK"} \\ \text{else} \\ \quad \| \text{"Not OK"} \end{array} = \text{"OK"}$$

$$\begin{array}{l} \text{if } e_2 \geq 1.2 \cdot d_0 \\ \quad \| \text{"OK"} \\ \text{else} \\ \quad \| \text{"Not OK"} \end{array} = \text{"OK"}$$

$$\begin{array}{l} \text{if } p_1 \geq 2.2 \cdot d_0 \\ \quad \| \text{"OK"} \\ \text{else} \\ \quad \| \text{"Not OK"} \end{array} = \text{"OK"}$$

$$\begin{array}{l} \text{if } p_2 \geq 2.4 \cdot d_0 \\ \quad \| \text{"OK"} \\ \text{else} \\ \quad \| \text{"Not OK"} \end{array} = \text{"OK"}$$

- Control of bukling:
acc. to EC3-1-8, §3.5

Buckling will not occur if:

$$\begin{array}{l} \text{if } a_{1.col} \leq 9 \cdot t_{plate} \cdot \sqrt[2]{\dfrac{235}{f_y}} \\ \quad \| \text{"No buckling"} \\ \text{else} \\ \quad \| \text{"Buckling check must be performed"} \end{array} = \text{"No buckling"}$$

Control of bearing resistance in the plate:

Acc. to EC3-1-8, table 3.4

α-values:

$$\alpha_{d.end} := \frac{e_1}{3 \cdot d_0} \qquad \alpha_{d.inner} := \frac{p_1}{3 \cdot d_0} - \frac{1}{4}$$

$$\alpha_{b.end} := min\left(\alpha_{d.end}, \frac{f_{ub}}{f_u}, 1\right) = 0.9$$

$$\alpha_{b.inner} := min\left(\alpha_{d.inner}, \frac{f_{ub}}{f_u}, 1\right) = 0.9$$

k-values:

$$k_{1.edge} := min\left(2.8 \cdot \frac{e_2}{d_0} - 1.7, 2.5\right) = 2.5$$

$$k_{1.inner} := min\left(1.4 \cdot \frac{p_2}{d_0}, 2.5\right) = 2.5$$

Bearing resistance:

Parallell to force direction

End dowels: $$F_{b.Rd.par.end} := \frac{k_{1.edge} \cdot \alpha_{b.end} \cdot f_u \cdot d \cdot t_{plate}}{\gamma_{M2}} = 1.763 \cdot 10^5$$

Inner dowels: $$F_{b.Rd.par.inner} := \frac{k_{1.inner} \cdot \alpha_{b.inner} \cdot f_u \cdot d \cdot t_{plate}}{\gamma_{M2}} = 1.763 \cdot 10^5$$

Total: $$F_{b.Rd.tot} := min\left(F_{b.Rd.par.end}, F_{b.Rd.par.inner}\right) \cdot n_{dowels.1} = 6.169 \cdot 10^6$$

Utility: $$u := \frac{F_{Ed.col}}{F_{b.Rd.tot}} = 0.259$$

Check:

```
if u ≤ 1
  ‖ "OK"          = "OK"
else
  ‖ "Not OK"
```

Diagonal part:

We will check only the diagonal in tension since this one is more critical than the one in compression.

Embedment strength:

Acc. to NS-EN 1995-1-1, §8.2

In connections with multiple shear planes the load-carrying capacity is determined by assuming that the external members are in single shear and the middle members in double shear. The total load-carrying capacity is determined by adding the contributions of compatible failure modes. Some of the failure modes cannot occur simultaneously due to deformation compatibility, meaning that they occur at different deformation levels: either small ("brittle") or large ("ductile").

Number of middle members: $n_{mid} := n_{plates} - 1 = 3$

Total nr. of shear planes for middle members: $n_{sp.mid} := n_{mid} \cdot 2 = 6$

$n_{out} := 2$

Total nr. of shear planes for outer members: $n_{sp.out} := 2$

Thickness of middle members: $t_2 := \dfrac{b_{diag}}{\left(n_{plates} + 1\right)} = 108$

Thickness of outer members: $t_1 := \dfrac{\left(b_{diag} - \left(n_{mid} \cdot t_2\right) - \left(n_{rows.2} \cdot t_{plate}\right)\right)}{2} = 60$

<u>Middle members:</u>

Formulas for double shear failure modes with external plates, NS-EN 1995-1-1, §8.2.3. Only thick plate modes considered.

Failure mode f) $F_{v.Rk.f} := f_{h.1.k.diag} \cdot t_1 \cdot d$

Failure mode g) $F_{v.Rk.g} := f_{h.1.k.diag} \cdot t_1 \cdot d \cdot \left(\sqrt[2]{2 + \frac{4 \cdot M_{y.Rk}}{f_{h.1.k.diag} \cdot d \cdot {t_1}^2}} - 1 \right)$

Failure mode h) $F_{v.Rk.h} := 2.3 \cdot \sqrt[2]{M_{y.Rk} \cdot f_{h.1.k.diag} \cdot d}$

Failure mode l) $F_{v.Rk.l} := 0.5 \cdot f_{h.2.k.diag} \cdot t_2 \cdot d$

Failure mode m) $F_{v.Rk.m} := 2.3 \cdot \sqrt[2]{M_{y.Rk} \cdot f_{h.2.k.diag} \cdot d}$

We note that dowels have no axial capacity. Therefor, no rope effect is included in the transverse capacity.

Total capacity <u>per fastener per shear plane for middle members:</u>

$$F_{v.Rk.middle} := min\left(F_{v.Rk.f}, F_{v.Rk.g}, F_{v.Rk.h}, F_{v.Rk.l}, F_{v.Rk.m}\right) = 1.316 \cdot 10^4$$

<u>Outer members:</u>

Formulas for single shear failure modes, NS-EN 1995-1-1, §8.2.3. Only thick plate modes considered.

Failure mode c) $F_{v.Rk.c} := f_{h.1.k.diag} \cdot t_1 \cdot d$

Failure mode d) $F_{v.Rk.d} := f_{h.1.k.diag} \cdot t_1 \cdot d \cdot \left(\sqrt[2]{2 + \frac{4 \cdot M_{y.Rk}}{f_{h.1.k.diag} \cdot d \cdot {t_1}^2}} - 1 \right)$

Failure mode e) $F_{v.Rk.e} := 2.3 \cdot \sqrt[2]{M_{y.Rk} \cdot f_{h.1.k.diag} \cdot d}$

We note that dowels have no axial capacity. Therefor, no rope effect is included in the transverse capacity.

Total capacity per fastener per shear plane for outer members:

$$F_{v.Rk.outer} := min\left(F_{v.Rk.c}, F_{v.Rk.d}, F_{v.Rk.e}\right) = 1.316 \cdot 10^4$$

Total capacity per fastener per shear plane:

We can only combine compatible failure modes.
For the practical purpose of being able to program the comparison, the failure modes will be identified as 1, 2, 3 etc. instead of a, b ,c etc.

Failure mode for middle members will hereafter be named "mode_mid"
Failure mode for outer members will hereafter be named "mode_outer"
It must also be noted that $\wedge$ is the logical operator "AND", and $\vee$ is the logical operator "OR", which will both be used in the if-else-statements below.

Failure mode for middle members:

$$mode\_mid := \left| \begin{array}{l} \text{if } F_{v.Rk.middle} = F_{v.Rk.f} \\ \quad \| \ 6 \\ \text{else if } F_{v.Rk.middle} = F_{v.Rk.g} \\ \quad \| \ 7 \\ \text{else if } F_{v.Rk.middle} = F_{v.Rk.h} \\ \quad \| \ 8 \\ \text{else if } F_{v.Rk.middle} = F_{v.Rk.l} \\ \quad \| \ 12 \\ \text{else if } F_{v.Rk.middle} = F_{v.Rk.m} \\ \quad \| \ 13 \end{array} \right| = 7$$

Failure mode for outer members:

$$mode\_outer := \left| \begin{array}{l} \text{if } F_{v.Rk.outer} = F_{v.Rk.c} \\ \quad \| \ 3 \\ \text{else if } F_{v.Rk.outer} = F_{v.Rk.d} \\ \quad \| \ 4 \\ \text{else if } F_{v.Rk.outer} = F_{v.Rk.e} \\ \quad \| \ 5 \end{array} \right| = 4$$

Comaptibility check:
Acc. to EC5, §8.1.3.2:

Now we must check if the failure mode for outer members are the same type as for the middle members. We can do this through an if-else statement:

$$
Failure := \begin{aligned}
&\text{if } (mode\_outer = 3) \wedge ((mode\_mid = 6) \vee (mode\_mid = 12)) \\
&\quad \| \text{"Brittle modes"} \\
&\text{else if } ((mode\_outer = 4) \vee (mode\_outer = 5)) \wedge ((mode\_mid = 7) \vee (mode\_mid = 8) \vee (mode\_mid = 13)) \\
&\quad \| \text{"Ductile modes"} \\
&\text{else} \\
&\quad \| \text{"Incompatible modes"}
\end{aligned}
$$

$$Failure = \text{"Ductile modes"}$$

As we can see, both the outer failure modes and the inner failure modes are ductile, which is what we want. In situations where both ductile and brittle types are possible it is good practice to try to ensure that the design condition is based on the ductile failure mechanism. Therefor, we will not proceed until ductile compatibility is achieved in the above code.

Total capacity per fastener for the column part is:

Shear planes for outer members: $n_{sp.out} = 2$

Shear planes for middle members: $n_{sp.mid} = 6$

$$F_{v.Rk.diag} := n_{sp.out} \cdot F_{v.Rk.outer} + n_{sp.mid} \cdot F_{v.Rk.middle} = 1.052 \cdot 10^{5}$$

Capacity per fastener in ULS:

$$F_{v.Rd.diag} := \frac{F_{v.Rk.diag}}{\gamma_{M.con}} \cdot (k_{mod}) = 8.906 \cdot 10^{4}$$

Now that the capacity per fastener has been found in ULS, we can check it for the design load to see if enough dowels have been selected.

Min. amount of dowels needed in the diagonal part of the connection:

$$n_{diag} := \left(\frac{F_{Ed.diag}}{F_{v.Rd.diag}}\right) = 26.781$$

Check if the chosen amount of dowels (in the start of the script) is sufficient:

```
if n_dowels.2 ≥ n_diag
    "OK"
else
    "Must increase nr. of dowels"
```
= "OK"

Chosen configuration:

$a_{1.diag} := 100$ $a_{2.diag} := 101$

$a_{3.t.diag} := 330$ $a_{3.c.diag} := 1000$

$a_{4.t.diag} := 1000$ $a_{4.c.diag} := 40$

The figure of the configuration is found in the column part calculation.

The a3.c distance (and a4.t) are actually bigger, but there is no point in measuring them because they will satisfy the distance-demand. Therefor a random number (which is big enough) has been implemented.

Minimum distances:

The minimum spacings given in EC5 have been derived to prevent splitting failure when connection is subjected to lateral load. With too small spacings we get increasing tension perpendicular to grain.

EC5, table 8.5:

$$\begin{array}{l|l} \text{if } a_{1.diag} \geq (3+(2 \cdot \cos(\alpha))) \cdot d & = \text{“OK”} \\ \quad \Vert \text{“OK”} & \\ \text{else} & \\ \quad \Vert \text{“Not OK”} & \end{array}$$

$$\begin{array}{l|l} \text{if } a_{2.diag} \geq 3 \cdot d & = \text{“OK”} \\ \quad \Vert \text{“OK”} & \\ \text{else} & \\ \quad \Vert \text{“Not OK”} & \end{array}$$

$$\begin{array}{l|l} \text{if } a_{3.t.diag} \geq \max(7 \cdot d, 80) & = \text{“OK”} \\ \quad \Vert \text{“OK”} & \\ \text{else} & \\ \quad \Vert \text{“Not ok”} & \end{array}$$

$$\begin{array}{l|l} \text{if } a_{3.c.diag} \geq \max(\sin(\alpha) \cdot d, 3 \cdot d) & = \text{“OK”} \\ \quad \Vert \text{“OK”} & \\ \text{else} & \\ \quad \Vert \text{“Not OK”} & \end{array}$$

$$\begin{array}{l|l} \text{if } a_{4.t.diag} \geq \max((2+2 \cdot \sin(\alpha)) \cdot d, 3 \cdot d) & = \text{“OK”} \\ \quad \Vert \text{“OK”} & \\ \text{else} & \\ \quad \Vert \text{“Not OK”} & \end{array}$$

$$\begin{array}{l|l} \text{if } a_{4.c.diag} \geq 3 \cdot d & = \text{“OK”} \\ \quad \Vert \text{“OK”} & \\ \text{else} & \\ \quad \Vert \text{“Not OK”} & \end{array}$$

All the minimum distances are fulfilled!

Splitting check: (Parallell to grain)

1) The first thing that needs to be sorted is the distances between the fasteners. This has been verified.

2) The second thing that must be verified is that the effective number of fasteners in a row has sufficient capacity to carry the load parallell to grain. This will be controlled in accordance with NS-EN 1995-1-1, §8.1.2(5).

Total amount of fasteners in one row in grain direction:

$$n_{row.diag} := \frac{n_{dowels.2}}{n_{rows.2}} = 11$$

The effective number of fasteners in one row in grain direction:
EC5, eq. (8.34)

$$n_{ef.row.diag} := min\left(n_{row.diag}, {n_{row.diag}}^{0.9} \cdot \sqrt[4]{\frac{a_{1.diag}}{13 \cdot d}}\right) = 7.744$$

The effective load-carrying capacity of each row then becomes:

$$F_{v.ef.Rk.diag} := F_{v.Rk.diag} \cdot n_{ef.row.diag} = 8.151 \cdot 10^{5}$$

For entire connection: $F_{v.ef.Rk.diag.tot} := F_{v.ef.Rk.diag} \cdot n_{rows.2} = 4.89 \cdot 10^{6}$

Design load in ULS: $F_{v.ef.Rd.diag} := \dfrac{F_{v.ef.Rk.diag.tot} \cdot k_{mod}}{\gamma_{M.con}} = 4.138 \cdot 10^{6}$

Control of splitting parallell to grain:

```
if F_Ed.diag ≤ F_v.ef.Rd.diag    = "OK"
  ‖ "OK"
else
  ‖ "Not OK"
```

Utility: $\dfrac{F_{Ed.diag}}{F_{v.ef.Rd.diag}} = 0.576$

Splitting check: (Perpendicular to grain)

No force components perpendicular to grain for the diagonals as explained in the calculation of the column part.

Control of tension of net cross section:

Net area: $A_{net.diag} := A_2 - \left(n_{plates} \cdot h_{diag} \cdot t_{plate}\right) - \left(d \cdot \frac{n_{dowels.2}}{n_{rows.2}} \cdot b_{diag}\right) = 2.072 \cdot 10^5$

Strength: $f_{t.0.d} := \frac{f_{t.0.k} \cdot k_{mod}}{\gamma_{M.GL}} = 18.652$

Compr. stress: $\sigma_{tens.diag} := \frac{F_{Ed.diag}}{A_{net.diag}} = 11.512$

Utility: $u := \frac{\sigma_{tens.diag}}{f_{t.0.d}} = 0.617$

Check:

```
if u ≤ 1
  ‖ "OK"
else
  ‖ "Not OK"
= "OK"
```

Block failure:

Acc. to EC5, §8.2.3(5)

For dowel type connections we should verify that block failure will not arise, by use of Annex A in EC5.

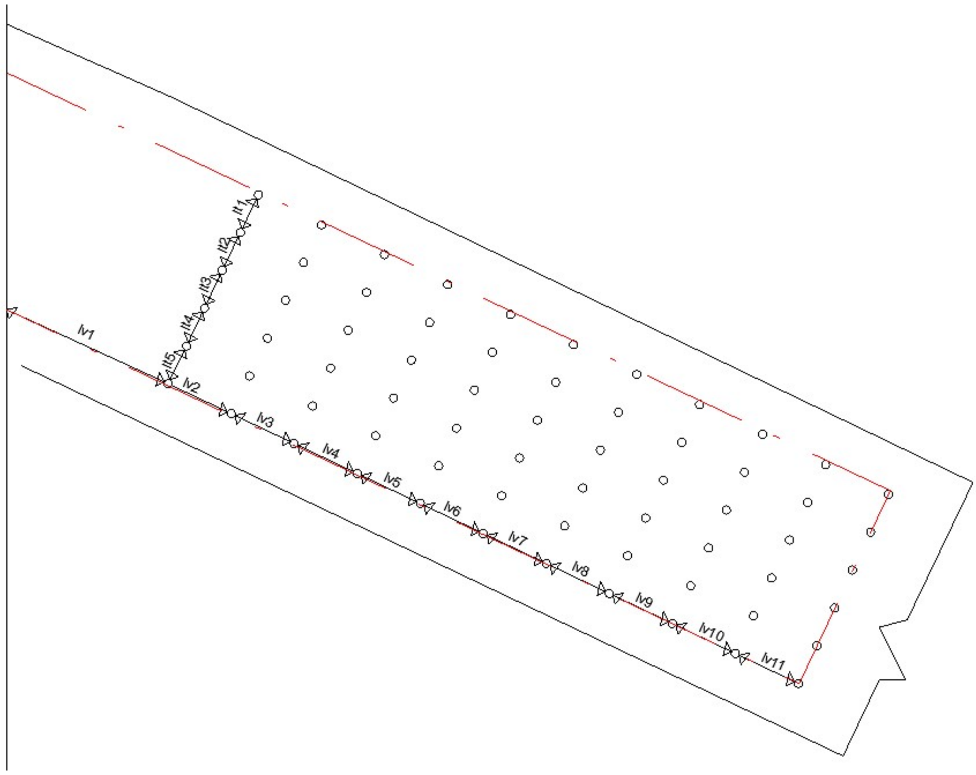
Lengths in v-direction:

$l_{v.1} := a_{3.t.diag}$ $l_{v.2} := a_{1.diag} - d = 88$ $l_{v.3} := l_{v.2}$ $l_{v.4} := l_{v.2}$ $l_{v.5} := l_{v.2}$

$l_{v.6} := l_{v.2}$ $l_{v.7} := l_{v.2}$ $l_{v.8} := l_{v.2}$ $l_{v.9} := l_{v.2}$ $l_{v.10} := l_{v.2}$ $l_{v.11} := l_{v.2}$

Lengths in t-direction:

$l_{t.1} := a_{2.diag} - d = 89$ $l_{t.2} := l_{t.1}$ $l_{t.3} := l_{t.1}$ $l_{t.4} := l_{t.1}$ $l_{t.5} := l_{t.1}$



Showing distances in t- and v-direction of the diagonal. The red line is the boundary of the block shear area.

Eq. A.4:

$$L_{net.v} := l_{v.1} + l_{v.2} + l_{v.3} + l_{v.4} + l_{v.5} + l_{v.6} + l_{v.7} + l_{v.8} + l_{v.9} + l_{v.10} + l_{v.11} = 1.21 \cdot 10^{3}$$

Eq. A.5: $L_{net.t} := l_{t.1} + l_{t.2} + l_{t.3} + l_{t.4} + l_{t.5} = 445$

Since we have failure modes (e) and (h) for outer and middle members, we get the following effective thickness (steel to timber connection with thick steel plates):

Eq. A.7 $t_{ef} := 2 \cdot \sqrt[2]{\frac{M_{y.Rk}}{f_{h.2.k.diag} \cdot d}} = 34.644$

Thickness of diagonal:

$$t_{1.outer} := t_1 = 60$$

$$t_{1.middle} := t_2 = 108$$

Net areas:

Eq. A.2:

Outer members: $A_{net.t.outer} := L_{net.t} \cdot t_{1.outer} \cdot n_{out} = 5.34 \cdot 10^{4}$

Middle members: $A_{net.t.middle} := L_{net.t} \cdot t_{1.middle} \cdot n_{mid} = 1.442 \cdot 10^{5}$

$$A_{net.t} := A_{net.t.outer} + A_{net.t.middle} = 1.976 \cdot 10^{5}$$

Eq. A.3:

Outer members: $A_{net.v.outer} := \frac{L_{net.v}}{2} \cdot \left(L_{net.v} + 2 \cdot t_{ef}\right) \cdot n_{out} = 1.548 \cdot 10^{6}$

Middle members: $A_{net.v.middle} := \frac{L_{net.v}}{2} \cdot \left(L_{net.v} + 2 \cdot t_{ef}\right) \cdot n_{mid} = 2.322 \cdot 10^{6}$

$$A_{net.v} := A_{net.v.outer} + A_{net.v.middle} = 3.87 \cdot 10^{6}$$

Block shear capacity:

Eq. A.1: $F_{bs.Rk} := \max\left(1.5 \cdot A_{net.t} \cdot f_{t.0.k}, 0.7 \cdot A_{net.v} \cdot f_{v.k}\right)$

ULS: $F_{bs.Rd} := \dfrac{F_{bs.Rk}}{\gamma_{M.con}} \cdot k_{mod} = 8.022 \cdot 10^{6}$

Control of block shear:

Utilization: $u := \dfrac{F_{Ed.diag}}{F_{bs.Rd}} = 0.297$

Check:

$$\begin{array}{l|l} \text{if } u \le 1 & = \text{"OK"} \\ \quad \text{"OK"} & \\ \text{else} & \\ \quad \text{"Not OK"} & \end{array}$$

Check of steel plates:

Acc. to EC5, 8.2.3(2)
The capacity of the steel plates will be found according to EC3.

Cross section classification:
EC3-1-1, table 5.2

Length: $C := b_{plate.2} = 432$

Thickness: $t := t_{plate} = 16$

Factor: $\varepsilon := 0.81$

Slenderness: $\lambda := \dfrac{C}{t \cdot \varepsilon} = 33.333$

$$\begin{array}{l|l} \text{if } \lambda \le 33 & = \text{"Class 2"} \\ \quad \text{"Class 1"} & \\ \text{else if } \lambda \le 38 & \\ \quad \text{"Class 2"} & \\ \text{else if } \lambda \le 42 & \\ \quad \text{"Class 3"} & \\ \text{else} & \\ \quad \text{"Class 4"} & \end{array}$$

- Tension check of steel plates:

Net area of steel plates: $A_{plate} := t \cdot (b_{plate.2} - n_{rows.2} \cdot d)$

Capacity of steel plate: $N_{t.Rd} := A_{plate} \cdot \frac{f_y}{\gamma_0} = 1.778 \cdot 10^6$

Control of compression capacity acc. to EC3-1-1, §6.2.4

$$\begin{array}{l} \text{if } n_{plates} \cdot N_{t.Rd} \geq F_{Ed.diag} \\ \quad \| \text{“OK”} \\ \text{else} \\ \quad \| \text{“Not OK”} \end{array} = \text{“OK”}$$

Utility: $u := \frac{F_{Ed.diag}}{n_{plates} \cdot N_{t.Rd}} = 0.335$

- Control of bearing resistance in the plate

Acc. to EC3-1-8, table 3.4:

α-values:

$$\alpha_{d.end} := \frac{e_1}{3 \cdot d_0}$$

$$\alpha_{d.inner} := \frac{p_1}{3 \cdot d_0} - \frac{1}{4}$$

$$\alpha_{b.end} := min\left(\alpha_{d.end}, \frac{f_{ub}}{f_u}, 1\right) = 0.9$$

$$\alpha_{b.inner} := min\left(\alpha_{d.inner}, \frac{f_{ub}}{f_u}, 1\right) = 0.9$$

k-values:

$$k_{1.edge} := min\left(2.8 \cdot \frac{e_2}{d_0} - 1.7, 2.5\right) = 2.5$$

$$k_{1.inner} := min\left(1.4 \cdot \frac{p_2}{d_0}, 2.5\right) = 2.5$$

Bearing resistance:
Parallell to force direction

End dowels: $F_{b.Rd.par.end} := \dfrac{k_{1.edge} \cdot \alpha_{b.end} \cdot f_u \cdot d \cdot t_{plate}}{\gamma_{M2}} = 1.763 \cdot 10^5$

Inner dowels: $F_{b.Rd.par.inner} := \dfrac{k_{1.inner} \cdot \alpha_{b.inner} \cdot f_u \cdot d \cdot t_{plate}}{\gamma_{M2}} = 1.763 \cdot 10^5$

Total: $F_{b.Rd.tot} := min\left(F_{b.Rd.par.end}, F_{b.Rd.par.inner}\right) \cdot n_{dowels.1} = 6.169 \cdot 10^6$

Utility: $u := \dfrac{F_{Ed.diag}}{F_{b.Rd.tot}} = 0.387$

Check:

```
if u ≤ 1
  || "OK"
else
  || "Not OK"
```
= "OK"

PART 3)
Connection Stiffness

Stiffness in SLS:
Acc. to NS-EN 1995-1-1, §7.1

Total number of shear planes: $n_{sp} := n_{sp.mid} + n_{sp.out} = 8$

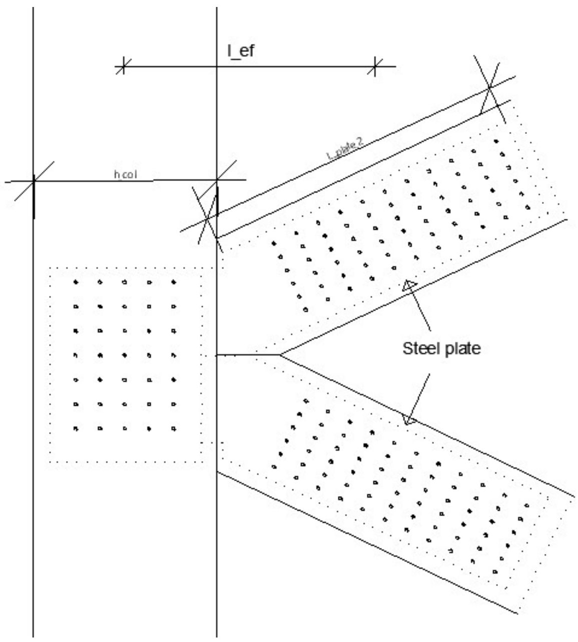
Stiffness per shear plane per fastener:

$$K_{ser} := \frac{{\rho_m}^{1.5} \cdot d}{23} = 5.487 \cdot 10^3$$

Slip modulus in column part: $K_{1.transl} := K_{ser} \cdot 2 \cdot n_{sp} \cdot n_{dowels.1} = 3.073 \cdot 10^6$

Slip modulus in diagonal part: $K_{2.transl} := K_{ser} \cdot 2 \cdot n_{sp} \cdot n_{dowels.2} = 5.794 \cdot 10^6$

Effective length of steel plates: $l_{plate.ef} := \dfrac{h_{col}}{2} + \dfrac{l_{plate.2}}{2} = 616.5$



The effective length of steel plates has been found by an approximation, based on the illustration shown in the figure.

$b_{plate.1} = 432 \qquad b_{plate.2} = 432$

Width of steel plates: $b_{plates} := b_{plate.1} = 432$

Net area of steel plates: $A_{net.plate} := t_{plate} \cdot \left(b_{plates} - n_{rows.1} \cdot d\right) = 5.952 \cdot 10^{3}$

Slip modulus of steel plates: $K_{3.transl} := E_s \cdot \dfrac{A_{net.plate}}{l_{plate.ef}} \cdot n_{plates} = 8.11 \cdot 10^{6}$

Total slip modulus:
The total slip modulus is calculated considering the different parts of the connection as springs in series.

$$K_{tot.con.SLS} := \left(\frac{1}{K_{1.transl}} + \frac{1}{K_{2.transl}} + \frac{1}{K_{3.transl}}\right)^{-1} = 1.609 \cdot 10^{6}$$

Check if the stiffness is sufficient:
Assuming same connection configuration in both ends of diagonals.

Effective length of diagonal: $L_{2.ef} := L_2 = 1.165 \cdot 10^{4}$

Stiffness of diagonal member: $K_{diag} := \dfrac{\left(h_{diag} \cdot b_{diag}\right) \cdot E_{0.mean}}{L_{2.ef}} = 3.688 \cdot 10^{5}$

System stiffness (translational):

$$K_{system.ser} := \frac{K_{diag} \cdot K_{tot.con.SLS}}{\left(K_{tot.con.SLS} + 2 \cdot K_{diag}\right)} = 2.529 \cdot 10^{5}$$

Ratio: $SLS\_ratio := \dfrac{K_{system.ser}}{K_{diag}} = 0.686$

Stiffness in ULS:

Since the members have the same time-dependent properties, we may use the mean stiffness values for the calculations.

Stiffness per plane per fastener: $K_u := \frac{2}{3} \cdot K_{ser} = 3.658 \cdot 10^3$

Slip modulus in part 1: $K_{transl.1} := K_u \cdot 2 \cdot n_{sp} \cdot n_{dowels.1} = 2.048 \cdot 10^6$

Slip modulus in part 2: $K_{transl.2} := K_u \cdot 2 \cdot n_{sp} \cdot n_{dowels.2} = 3.863 \cdot 10^6$

Slip modulus of steel plates: $K_{transl.3} := E_s \cdot \frac{A_{net.plate}}{l_{plate.ef}} \cdot n_{plates} = 8.11 \cdot 10^6$

Total stiffness (slip modulus):

$$K_{tot.con.uls} := \left(\frac{1}{K_{transl.1}} + \frac{1}{K_{transl.2}} + \frac{1}{K_{transl.3}} \right)^{-1} = 1.149 \cdot 10^6$$

Check if the stiffness is sufficient:

Assuming same connection configuration in both ends.

Effective length of diagonal: $L_{2.ef} := L_2 = 1.165 \cdot 10^4$

Stiffness of diagonal member: $K_{diag} := \frac{\left(h_{diag} \cdot b_{diag}\right) \cdot E_{0.mean}}{L_{2.ef}} = 3.688 \cdot 10^5$

System stiffness (translational): $K_{system.uls} := \frac{K_{diag} \cdot K_{tot.con.uls}}{\left(K_{tot.con.uls} + 2 \cdot K_{diag}\right)} = 2.246 \cdot 10^5$

Ratio: $ULS\_ratio := \frac{K_{system.uls}}{K_{diag}} = 0.609$

C3.1. Design of Beam

- Ultimate Limit State
- NS-EN-1995-1-1

Cross Section:

$H := 540 \quad [mm] \qquad B := 360 \quad [mm]$

Material: GL30c

Characteristic bending strength $f_{mk} := 30 \left[\frac{N}{mm^2}\right]$

Characteristic shear strength $f_{vk} := 3.5 \left[\frac{N}{mm^2}\right]$

Characteristic tension strength // grain $f_{t0k} := 19.5 \left[\frac{N}{mm^2}\right]$

Characteristic compression strength // grain $f_{c0k} := 24.5 \left[\frac{N}{mm^2}\right]$

Characteristic compression strength perpendicular to grain $f_{c90k} := 2.5 \left[\frac{N}{mm^2}\right]$

Characteristic tension strength perpendicular to grain $f_{t90k} := 0.5 \left[\frac{N}{mm^2}\right]$

Bucklling length about y-axis (strong axis) $L_{ky} := 9600 \quad [mm]$

Buckling length about z-axis (weak axis) $L_{kz} := 0 \quad [mm]$

Beams are assumed restrained in slabs

Modification factor (Tab. 3.1) $k_{mod} := 0.8$

Safety factor $\gamma := 1.15$

5% - fractile Elacticity modulus $E_{0.05} := 10800 \left[\frac{N}{mm^2}\right]$

Cross-section area $A := H \cdot B = 194400$ $[mm^2]$

Moment of Inertia $I_y := \frac{1}{12} \cdot B \cdot H^3 = 4.724 \cdot 10^9$ $[mm^4]$

$I_z := \frac{1}{12} \cdot B^3 \cdot H = 2.1 \cdot 10^9$ $[mm^4]$

Moment of Resistance $W_y := \frac{1}{6} \cdot B \cdot H^2 = 1.75 \cdot 10^7$ $[mm^3]$

$W_z := \frac{1}{6} \cdot B^2 \cdot H = 1.166 \cdot 10^7$ $[mm^3]$

Height Factor (EC5: 3.3)

$$k_h := \begin{array}{l} \text{if } H < 600 \\ \quad \left\| \, min\left(\left(\frac{600}{H}\right)^{0.1}, 1.1\right) \right. \\ \text{else if } H \geq 600 \\ \quad \left\| \, 1.0 \right. \end{array} = 1.011$$

Design strength

Design bending strength (y-axis)

$$f_{myd} := \frac{f_{mk} \cdot k_{mod}}{\gamma} \cdot k_h = 21.091 \quad \left[\frac{N}{mm^2}\right]$$

Design bending strength (z-axis)

$$f_{mzd} := f_{myd} = 21.091 \quad \left[\frac{N}{mm^2}\right]$$

Design tension strength // grain

$$f_{t0d} := \frac{f_{t0k} \cdot k_{mod}}{\gamma} \cdot k_h = 13.709 \quad \left[\frac{N}{mm^2}\right]$$

Design compression strength // grain

$$f_{c0d} := \frac{f_{c0k} \cdot k_{mod}}{\gamma} = 17.043 \quad \left[\frac{N}{mm^2}\right]$$

Design tension strength perpendicular to grain

$$f_{t90d} := \frac{f_{t90k} \cdot k_{mod}}{\gamma} = 0.348 \quad \left[\frac{N}{mm^2}\right]$$

Design compression strength perpendicular to grain

$$f_{c90d} := \frac{f_{c90k} \cdot k_{mod}}{\gamma} = 1.739 \quad \left[\frac{N}{mm^2}\right]$$

Design shear strength

$$f_{vd} := \frac{f_{vk} \cdot k_{mod}}{\gamma} = 2.435 \quad \left[\frac{N}{mm^2}\right]$$

Acting Forces (Extracted from *Robot Analysis*)

Moment: $M_{ed.y} := 325 \cdot 10^6 \quad [Nmm]$

$M_{ed.z} := 0 \cdot 10^6 \quad [Nmm]$

Bending stress (y-axis):

$$\sigma_{myd} := \frac{M_{ed.y}}{W_y} = 18.576 \left[\frac{N}{mm^2}\right]$$

Bending stress (z-axis):

$$\sigma_{mzd} := \frac{M_{ed.z}}{W_z} = 0$$

Shear: $V_{ed.z} := 114 \cdot 10^3 \quad [N]$

$V_{ed.y} := 0 \cdot 10^3 \quad [N]$

Shear stress:

$k_{cr} := 0.80$ (Glulam) (EC5: 6.1.7(2))

$b_{ef} := k_{cr} \cdot B = 288 \quad [mm]$ (6.13a)

$h_{ef} := k_{cr} \cdot H = 432 \quad [mm]$

$$\tau_{d.z} := \frac{3}{2} \cdot \frac{V_{ed.z}}{b_{ef} \cdot H} = 1.1 \quad \left[\frac{N}{mm^2}\right] \qquad \tau_{d.y} := \frac{3}{2} \cdot \frac{V_{ed.y}}{h_{ef} \cdot B} = 0 \quad \left[\frac{N}{mm^2}\right]$$

Axial: $N_{c.ed} := 0 \cdot 10^3 \quad [N]$

$N_{t.ed} := 0 \cdot 10^3 \quad [N]$

Axial stress:

$$\sigma_{c0d} := \frac{N_{c.ed}}{A} = 0 \quad \left[\frac{N}{mm^2}\right] \qquad \sigma_{t0d} := \frac{N_{t.ed}}{A} = 0 \quad \left[\frac{N}{mm^2}\right]$$

Bending - 6.1.6

Control check:

$k_m := 0.7$ (Glulam) (EC5 6.1.6(2))

$$\frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.11)$$

$$k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.12)$$

Utilization

$$\frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0.881 \qquad (6.11)$$

$$k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0.617 \qquad (6.12)$$

Shear - 6.1.7

Requirements:

$$\frac{\tau_d}{f_{vd}} \le 1 \qquad (6.13)$$

Utilization

$$\frac{\tau_{d.z}}{f_{vd}} = 0.452 \qquad \frac{\tau_{d.y}}{f_{vd}} = 0 \qquad (6.13)$$

Axial (Tension) - 6.1.2

Requirements:

$$\frac{\sigma_{t0d}}{f_{t0d}} \leq 1 \tag{6.1}$$

Utilization

$$\frac{\sigma_{t0d}}{f_{t0d}} = 0 \tag{6.1}$$

Axial (Compression) - 6.1.4

Requirements:

$$\frac{\sigma_{c0d}}{f_{c0d}} \leq 1 \tag{6.2}$$

Utilization

$$\frac{\sigma_{c0d}}{f_{c0d}} = 0 \tag{6.2}$$

Combination of Bending and Axial (tension) stress - 6.2.3

Requirements:

$k_m := 0.7$ (EC5: 6.1.6(2))

$$\frac{\sigma_{t0d}}{f_{t0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.17)$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.18)$$

Utilizations

$$\frac{\sigma_{t0d}}{f_{t0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0.881 \qquad (6.17)$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0.617 \qquad (6.18)$$

Combinations of Bending and Axial (compression) stress - 6.2.4

Requirements:

$k_m := 0.7$ (EC5: 6.1.6(2))

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.19)$$

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.20)$$

Utilizations

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0.881 \qquad (6.19)$$

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0.617 \qquad (6.20)$$

Stability - Buckling - 6.3.2

Buckling length (y-axis) $L_{ky}=9600\ [mm]$

Slenderness (y-axis)

$$\lambda_y := \frac{L_{ky}}{H} \cdot \sqrt[2]{12} = 61.584$$

$$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 0.934 \qquad (6.21)$$

Buckling length (z-axis) $L_{kz}=0\ [mm]$

Slenderness (z-axis)

$$\lambda_z := \frac{L_{kz}}{B} \cdot \sqrt[2]{12} = 0$$

$$\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 0 \qquad (6.22)$$

EC5: 6.3.2(3)

$\beta_c := 0.1$ Glulam (6.29)

$$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + {\lambda_{rel.y}}^2\right) = 0.968 \qquad (6.27)$$

$$k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + {\lambda_{rel.z}}^2\right) = 0.485 \qquad (6.28)$$

$$k_{cy} := \frac{1}{k_y + \sqrt[2]{{k_y}^2 - {\lambda_{rel.y}}^2}} = 0.819 \qquad (6.25)$$

$$k_{cz} := \frac{1}{k_z + \sqrt[2]{{k_z}^2 - {\lambda_{rel.z}}^2}} = 1.031 \qquad (6.26)$$

Control - Combination of Axial and Bending

Required:

$$k_m := 0.7$$ (EC5: 6.1.6(2))

$$\frac{\sigma_{c0d}}{k_{cy} \cdot f_{c0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \le 1$$ (6.23)

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \le 1$$ (6.24)

Utilizations

$$\frac{\sigma_{c0d}}{k_{cy} \cdot f_{c0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0.881$$ (6.23)

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0.617$$ (6.24)

Stability - LTB - 6.3.3

Checked when bending is acting alone or with compression

$$L := L_{ky} = 9600 \quad [mm]$$

$$l_{ef} := 0.9 \cdot L + 2 \cdot H = 9720 \quad [mm] \qquad \text{(Table 6.1)}$$

$$\sigma_{m.crit} := \frac{0.78 \cdot B^2}{H \cdot l_{ef}} \cdot E_{0.05} = 208 \quad \left[\frac{N}{mm^2}\right] \qquad (6.32)$$

$$\lambda_{rel.m} := \sqrt[2]{\frac{f_{mk}}{\sigma_{m.crit}}} = 0.38 \qquad (6.30)$$

$$k_{crit} := \begin{array}{l} \text{if } \lambda_{rel.m} \le 0.75 \\ \quad \| \; 1.0 \\ \text{else if } 0.75 < \lambda_{rel.m} \le 1.4 \\ \quad \| \; 1.56 - 0.75 \cdot \lambda_{rel.m} \\ \text{else if } 1.4 < \lambda_{rel.m} \\ \quad \| \; \dfrac{1.0}{\lambda_{rel.m}^{\ 2}} \end{array} = 1 \qquad (6.34)$$

Control - Bending

Requirements:

$$k_{crit} := 1.0 \qquad (6.34)$$

$$\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}} \le 1 \qquad (6.33)$$

Utilization

$$\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}} = 0.881 \qquad (6.33)$$

Control - Combination Bending and Axial

Requirements:

$$k_{crit} = 1 \qquad (6.34)$$

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + \left(\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}}\right)^2 \le 1 \qquad (6.35)$$

Utilization

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + \left(\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}}\right)^2 = 0.776 \qquad (6.35)$$

C3.2. Design of Columns

- Ultimate Limit State
- NS-EN1995-1-1

Cross Section:

$H := 720 \quad [mm] \qquad B := 720 \quad [mm] \qquad \alpha := 0 \quad [deg]$

Material: GL30C

Characteristic bending strength	$f_{mk} := 30$	$\left[\dfrac{N}{mm^2}\right]$
Characteristic shear strength	$f_{vk} := 3.5$	$\left[\dfrac{N}{mm^2}\right]$
Characteristic tension strength // grain	$f_{t0k} := 19.5$	$\left[\dfrac{N}{mm^2}\right]$
Characteristic compression strength // grain	$f_{c0k} := 24.5$	$\left[\dfrac{N}{mm^2}\right]$
Characteristic compression strength perpendicular to grain	$f_{c90k} := 2.5$	$\left[\dfrac{N}{mm^2}\right]$
Characteristic tension strength perpendicular to grain	$f_{t90k} := 0.5$	$\left[\dfrac{N}{mm^2}\right]$
Buckling about y-axis (strong axis)	$L_{ky} := 3500$	$[mm]$
Buckling about z-axis (weak axis)	$L_{kz} := 3500$	$[mm]$
Modification Factor (Tab. 3.1)	$k_{mod} := 0.8$	
Safety Factor	$\gamma := 1.15$	
5% - fractile Elasticity Modulus	$E_{0.05} := 10800$	$\left[\dfrac{N}{mm^2}\right]$

Cross Section Area $A := H \cdot B = 5.184 \cdot 10^{5}$ $\left[mm^{2} \right]$

Moment of Inertia (y-axis) $I_y := \frac{1}{12} \cdot B \cdot H^{3} = 2.239 \cdot 10^{10}$ $\left[mm^{4} \right]$

$I_z := \frac{1}{12} \cdot B^{3} \cdot H = 2.239 \cdot 10^{10}$ $\left[mm^{4} \right]$

Moment of Resistance (y-axis) $W_y := \frac{1}{6} \cdot B \cdot H^{2} = 6.221 \cdot 10^{7}$ $\left[mm^{3} \right]$

$W_z := \frac{1}{6} \cdot B^{2} \cdot H = 6.221 \cdot 10^{7}$ $\left[mm^{3} \right]$

Height Factor (EC5: 3.3)

$$k_h := \left| \begin{array}{l} \text{if } H < 600 \\ \quad \left\| \, min\left(\left(\frac{600}{H} \right)^{0.1}, 1.1 \right) \right. \\ \text{else if } H \geq 600 \\ \quad \left\| \, 1.0 \right. \end{array} \right| = 1$$

Design strength

Design bending strength (y-axis)

$$f_{myd} := \frac{f_{mk} \cdot k_{mod}}{\gamma} \cdot k_h = 20.87 \quad \left[\frac{N}{mm^2}\right]$$

Design bending strength (z-axis)

$$f_{mzd} := f_{myd} = 20.87 \quad \left[\frac{N}{mm^2}\right]$$

Design tension strength // grain

$$f_{t0d} := \frac{f_{t0k} \cdot k_{mod}}{\gamma} \cdot k_h = 13.565 \left[\frac{N}{mm^2}\right]$$

Design compression strength // grain

$$f_{c0d} := \frac{f_{c0k} \cdot k_{mod}}{\gamma} = 17.043 \quad \left[\frac{N}{mm^2}\right]$$

Design tension strength perpendiculat to grain

$$f_{t90d} := \frac{f_{t90k} \cdot k_{mod}}{\gamma} = 0.348 \quad \left[\frac{N}{mm^2}\right]$$

Design tension strength perpendiculat to grain

$$f_{c90d} := \frac{f_{c90k} \cdot k_{mod}}{\gamma} = 1.739 \quad \left[\frac{N}{mm^2}\right]$$

Design shear strength

$$f_{vd} := \frac{f_{vk} \cdot k_{mod}}{\gamma} = 2.435 \quad \left[\frac{N}{mm^2}\right]$$

Design tension strength angle to grain

$$f_{cad} := \frac{f_{c0d}}{\frac{f_{c0d}}{f_{c90d}} \cdot \sin(\alpha \cdot \boldsymbol{deg})^2 + \cos(\alpha \cdot \boldsymbol{deg})^2}$$

Acting Forces (Extracted from *Robot Structutal Analysis*)

Moment: $M_{ed.y} := 0 \cdot 10^6$ $[Nmm]$

$M_{ed.z} := 0 \cdot 10^6$ $[Nmm]$

Bending stress (y-axis):

$$\sigma_{myd} := \frac{M_{ed.y}}{W_y} = 0 \quad \left[\frac{N}{mm^2}\right]$$

Bending stress (z-axis):

$$\sigma_{mzd} := \frac{M_{ed.z}}{W_z} = 0$$

Shear: $V_{ed.z} := 0 \cdot 10^3$ $[N]$

$V_{ed.y} := 0 \cdot 10^3$ $[N]$

Shear stress:

$k_{cr} := 0.80$ (Glulam) (EC5: 6.1.7(2))

$b_{ef} := k_{cr} \cdot B = 576$ $[mm]$ (6.13a)

$h_{ef} := k_{cr} \cdot H = 576$ $[mm]$

$$\tau_{d.z} := \frac{3}{2} \cdot \frac{V_{ed.z}}{b_{ef} \cdot H} = 0 \quad \left[\frac{N}{mm^2}\right] \qquad \tau_{d.y} := \frac{3}{2} \cdot \frac{V_{ed.y}}{h_{ef} \cdot B} = 0 \quad \left[\frac{N}{mm^2}\right]$$

Axial: $N_{c.ed} := 7600 \cdot 10^3$ $[N]$

$N_{t.ed} := 0 \cdot 10^3$ $[N]$

Axial stress:

$$\sigma_{c0d} := \frac{N_{c.ed}}{A} = 14.66 \quad \left[\frac{N}{mm^2}\right] \qquad \sigma_{t0d} := \frac{N_{t.ed}}{A} = 0 \quad \left[\frac{N}{mm^2}\right]$$

Bending - 6.1.6

Requirements:

$k_m := 0.7$ (Rectangle cross-section, Glulam) (EC5: 6.1.6(2))

$$\frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.11)$$

$$k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.12)$$

Utilization

$$\frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0 \qquad (6.11)$$

$$k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0 \qquad (6.12)$$

Shear - 6.1.7

Requirements:

$$\frac{\tau_d}{f_{vd}} \le 1 \qquad (6.13)$$

Utilization

$$\frac{\tau_{d.z}}{f_{vd}} = 0 \qquad \frac{\tau_{d.y}}{f_{vd}} = 0 \qquad (6.13)$$

Axial (tensile) - 6.1.2

Requirements:

$$\frac{\sigma_{t0d}}{f_{t0d}} \leq 1 \qquad (6.1)$$

Utilization

$$\frac{\sigma_{t0d}}{f_{t0d}} = 0 \qquad (6.1)$$

Axial (compression) - 6.1.4

Requirements:

$$\frac{\sigma_{c0d}}{f_{c0d}} \leq 1 \qquad (6.2)$$

Utilization

$$\frac{\sigma_{c0d}}{f_{c0d}} = 0.86 \qquad (6.2)$$

Combined bending and axial tension - 6.2.3

Requirements:

$k_m := 0.7$ (EC5: 6.1.6(2))

$$\frac{\sigma_{t0d}}{f_{t0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.17)$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.18)$$

Calculations

$$\frac{\sigma_{t0d}}{f_{t0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0 \qquad (6.17)$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0 \qquad (6.18)$$

Combined bending and axial compression - 6.2.4

Requirements:

$k_m := 0.7$ (EC5: 6.1.6(2))

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.19)$$

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.20)$$

Utilization

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0.74 \qquad (6.19)$$

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0.74 \qquad (6.20)$$

Stability - Buckling - 6.3.2

Combined axial and bending

Buckling length (y-axis) $L_{ky} = 3500$ $[mm]$

Slenderness (y-axis)

$$\lambda_y := \frac{L_{ky}}{H} \cdot \sqrt[2]{12} = 16.839$$

$$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 0.255 \qquad (6.21)$$

Buckling length (z-axis) $L_{kz} = 3500$ $[mm]$

Slenderness

$$\lambda_z := \frac{L_{kz}}{B} \cdot \sqrt[2]{12} = 16.839$$

$$\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 0.255 \qquad (6.22)$$

EC5: 6.3.2(3)

$\beta_c := 0.1$ Glulam (6.29)

$$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + {\lambda_{rel.y}}^2\right) = 0.53 \qquad (6.27)$$

$$k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + {\lambda_{rel.z}}^2\right) = 0.53 \qquad (6.28)$$

$$k_{cy} := \frac{1}{k_y + \sqrt[2]{{k_y}^2 - {\lambda_{rel.y}}^2}} = 1.005 \qquad (6.25)$$

$$k_{cz} := \frac{1}{k_z + \sqrt[2]{{k_z}^2 - {\lambda_{rel.z}}^2}} = 1.005 \qquad (6.26)$$

Control - Combined bending and axial

Requirements:

$k_m := 0.7$ (EC5: 6.1.6(2))

$$\frac{\sigma_{c0d}}{k_{cy} \cdot f_{c0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.23)$$

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.24)$$

Utilization

$$\frac{\sigma_{c0d}}{k_{cy} \cdot f_{c0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0.856 \qquad (6.23)$$

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0.856 \qquad (6.24)$$

Stability - LTB - 6.3.3

Simply supported beams with uniform distributed load

$$L := L_{ky} = 3500 \quad [mm]$$

$$l_{ef} := 0.9 \cdot L + 2 \cdot H = 4590 \quad [mm]$$ (Tabell 6.1)

$$\sigma_{m.crit} := \frac{0.78 \cdot B^2}{H \cdot l_{ef}} \cdot E_{0.05} = 1.321 \cdot 10^3 \left[\frac{N}{mm^2}\right]$$ (6.32)

$$\lambda_{rel.m} := \sqrt[2]{\frac{f_{mk}}{\sigma_{m.crit}}} = 0.151$$ (6.30)

$$k_{crit} := \begin{array}{l} \text{if } \lambda_{rel.m} \le 0.75 \\ \quad \| \; 1.0 \\ \text{else if } 0.75 < \lambda_{rel.m} \le 1.4 \\ \quad \| \; 1.56 - 0.75 \cdot \lambda_{rel.m} \\ \text{else if } 1.4 < \lambda_{rel.m} \\ \quad \| \; \dfrac{1.0}{\lambda_{rel.m}^{\;2}} \end{array} = 1$$ (6.34)

Control - Bending
Requirements:

$$k_{crit} = 1$$ (6.34)

$$\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}} \le 1$$ (6.33)

Utilization

$$\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}} = 0$$ (6.33)

Control- Combination bending and compression

Requirements:

$$k_{crit} = 1 \tag{6.34}$$

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + \left(\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}}\right)^2 \leq 1 \tag{6.35}$$

Utilization

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + \left(\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}}\right)^2 = 0.856 \tag{6.35}$$

C3.3. Design of Diagonals

- Ultimate Limit State
- NS-EN1995-1-1

Cross Section:

$H := 585 \quad [mm] \qquad B := 540 \quad [mm]$

Material: GL30C

Characteristic bending strength $f_{mk} := 30 \quad \left[\frac{N}{mm^2}\right]$

Characteristic shear strength $f_{vk} := 3.5 \quad \left[\frac{N}{mm^2}\right]$

Characteristic tension strength // grain $f_{t0k} := 19.5 \quad \left[\frac{N}{mm^2}\right]$

Characteristic compression strength // grain $f_{c0k} := 24.5 \quad \left[\frac{N}{mm^2}\right]$

Characteristic compression strength perpendicular to grain $f_{c90k} := 2.5 \quad \left[\frac{N}{mm^2}\right]$

Characteristic tension strength perpendicular to grain $f_{t90k} := 0.5 \quad \left[\frac{N}{mm^2}\right]$

Bucklling length about y-axis (strong axis) $L_{ky} := 11650 \quad [mm]$

Buckling length about z-axis (weak axis) $L_{kz} := 0 \quad [mm]$

Diagonals are assumed restrained in weak axis

Modification factor (Tab. 3.1) $k_{mod} := 1.1$

Safety factor $\gamma := 1.15$

5% - fractile Elacticity modulus $E_{0.05} := 10800 \quad \left[\frac{N}{mm^2}\right]$

Cross-section area $A := H \cdot B = 315900 \quad [mm^2]$

Moment of Inertia $I_y := \frac{1}{12} \cdot B \cdot H^3 = 9.009 \cdot 10^9 \quad [mm^4]$

$I_z := \frac{1}{12} \cdot B^3 \cdot H = 7.676 \cdot 10^9 \quad [mm^4]$

Moment of Resistance $W_y := \frac{1}{6} \cdot B \cdot H^2 = 3.08 \cdot 10^7 \quad [mm^3]$

$W_z := \frac{1}{6} \cdot B^2 \cdot H = 2.843 \cdot 10^7 \quad [mm^3]$

Height Factor (EC5: 3.3)

$$k_h := \begin{array}{l} \text{if } H < 600 \\ \quad \left\| \, min\left(\left(\frac{600}{H}\right)^{0.1}, 1.1\right) \right. \\ \text{else if } H \geq 600 \\ \quad \left\| \, 1.0 \right. \end{array} = 1.003$$

Design strength

Design bending strength (y-axis)

$$f_{myd} := \frac{f_{mk} \cdot k_{mod}}{\gamma} \cdot k_h = 28.768 \quad \left[\frac{N}{mm^2}\right]$$

Design bending strength (z-axis)

$$f_{mzd} := f_{myd} = 28.768 \quad \left[\frac{N}{mm^2}\right]$$

Design tension strength // grain

$$f_{t0d} := \frac{f_{t0k} \cdot k_{mod}}{\gamma} \cdot k_h = 18.699 \quad \left[\frac{N}{mm^2}\right]$$

Design compression strength // grain

$$f_{c0d} := \frac{f_{c0k} \cdot k_{mod}}{\gamma} = 23.435 \quad \left[\frac{N}{mm^2}\right]$$

Design tension strength perpendicular to grain

$$f_{t90d} := \frac{f_{t90k} \cdot k_{mod}}{\gamma} = 0.478 \quad \left[\frac{N}{mm^2}\right]$$

Design compression strength perpendicular to grain

$$f_{c90d} := \frac{f_{c90k} \cdot k_{mod}}{\gamma} = 2.391 \quad \left[\frac{N}{mm^2}\right]$$

Design shear strength

$$f_{vd} := \frac{f_{vk} \cdot k_{mod}}{\gamma} = 3.348 \quad \left[\frac{N}{mm^2}\right]$$

Acting Forces extracted from *Robot Structural*

Moment: $M_{ed.y} := 0 \cdot 10^6$ $[Nmm]$

$M_{ed.z} := 0 \cdot 10^6$

Bending stress (y-axis):

$$\sigma_{myd} := \frac{M_{ed.y}}{W_y} = 0 \qquad \left[\frac{N}{mm^2}\right]$$

Bending stress (z-axis):

$$\sigma_{mzd} := \frac{M_{ed.z}}{W_z} = 0$$

Shear: $V_{ed.z} := 0 \cdot 10^3$ $[N]$

$V_{ed.y} := 0 \cdot 10^3$ $[N]$

Shear stress:

$k_{cr} := 0.80$ (Glulam) (EC5: 6.1.7(2))

$b_{ef} := k_{cr} \cdot B = 432$ $[mm]$ (6.13a)

$h_{ef} := k_{cr} \cdot H = 468$ $[mm]$

$$\tau_{d.z} := \frac{3}{2} \cdot \frac{V_{ed.z}}{b_{ef} \cdot H} = 0 \qquad \left[\frac{N}{mm^2}\right] \qquad\qquad \tau_{d.y} := \frac{3}{2} \cdot \frac{V_{ed.y}}{h_{ef} \cdot B} = 0 \qquad \left[\frac{N}{mm^2}\right]$$

Aksial: $N_{c.ed} := 2400 \cdot 10^3$ $[N]$

$N_{t.ed} := 2400 \cdot 10^3$

Assumed to be similar in both, compression and tension

Axial stress:

$$\sigma_{c0d} := \frac{N_{c.ed}}{A} = 7.597 \qquad \left[\frac{N}{mm^2}\right] \qquad\qquad \sigma_{t0d} := \frac{N_{t.ed}}{A} = 7.597 \qquad \left[\frac{N}{mm^2}\right]$$

Bending - 6.1.6

Control check:

$k_m := 0.7$ (Glulam) (EC5: 6.1.6(2))

$$\frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.11)$$

$$k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.12)$$

Utilization

$$\frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0 \qquad (6.11)$$

$$k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0 \qquad (6.12)$$

Shear - 6.1.7

Requirements:

$$\frac{\tau_d}{f_{vd}} \le 1 \qquad (6.13)$$

Utilization

$$\frac{\tau_{d.z}}{f_{vd}} = 0 \qquad \frac{\tau_{d.y}}{f_{vd}} = 0 \qquad (6.13)$$

Axial (Tension) - 6.1.2

Requirements:

$$\frac{\sigma_{t0d}}{f_{t0d}} \leq 1 \qquad (6.1)$$

Utilization

$$\frac{\sigma_{t0d}}{f_{t0d}} = 0.406 \qquad (6.1)$$

Axial (Compression) - 6.1.4

Requirements:

$$\frac{\sigma_{c0d}}{f_{c0d}} \leq 1 \qquad (6.2)$$

Utilization

$$\frac{\sigma_{c0d}}{f_{c0d}} = 0.324 \qquad (6.2)$$

Combination of Bending and Axial (tension) stress - 6.2.3

Requirements:

$k_m := 0.7$ (EC5: 6.1.6(2))

$$\frac{\sigma_{t0d}}{f_{t0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.17)$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.18)$$

Utilizations

$$\frac{\sigma_{t0d}}{f_{t0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0.406 \qquad (6.17)$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0.406 \qquad (6.18)$$

Combinations of Bending and Axial (compression) stress - 6.2.4

Requirements:

$k_m := 0.7$ (EC5: 6.1.6(2))

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \quad (6.19)$$

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \quad (6.20)$$

Utilizations

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0.105 \quad (6.19)$$

$$\left(\frac{\sigma_{c0d}}{f_{c0d}}\right)^2 + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0.105 \quad (6.20)$$

Stability - Buckling - 6.3.2

Buckling length (y-axis) $L_{ky} = 11650 \quad [mm]$

Slenderness (y-axis)

$$\lambda_y := \frac{L_{ky}}{H} \cdot \sqrt[2]{12} = 68.986$$

$$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 1.046 \qquad (6.21)$$

Buckling length (z-aksen) $L_{kz} = 0 \quad [mm]$

Slenderness (z-axis)

$$\lambda_z := \frac{L_{kz}}{B} \cdot \sqrt[2]{12} = 0$$

$$\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 0 \qquad (6.22)$$

EC5: 6.3.2(3)

$\beta_c := 0.1$ Glulam (6.29)

$$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + \lambda_{rel.y}^{\ 2}\right) = 1.084 \qquad (6.27)$$

$$k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + \lambda_{rel.z}^{\ 2}\right) = 0.485 \qquad (6.28)$$

$$k_{cy} := \frac{1}{k_y + \sqrt[2]{k_y^{\ 2} - \lambda_{rel.y}^{\ 2}}} = 0.73 \qquad (6.25)$$

$$k_{cz} := \frac{1}{k_z + \sqrt[2]{k_z^{\ 2} - \lambda_{rel.z}^{\ 2}}} = 1.031 \qquad (6.26)$$

Control - Combination of Axial and Bending

Required:

$$k_m := 0.7$$ (EC5: 6.1.6(2))

$$\frac{\sigma_{c0d}}{k_{cy} \cdot f_{c0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \leq 1$$ (6.23)

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \leq 1$$ (6.24)

Utilizations

$$\frac{\sigma_{c0d}}{k_{cy} \cdot f_{c0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} = 0.444$$ (6.23)

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} = 0.314$$ (6.24)

Stability - LTB - 6.3.3

Checked when bending is acting alone or with compression

$L := L_{ky} = 11650 \quad [mm]$

$l_{ef} := 0.9 \cdot L + 2 \cdot H = 11655 \quad [mm]$ (Table 6.1)

$$\sigma_{m.crit} := \frac{0.78 \cdot B^2}{H \cdot l_{ef}} \cdot E_{0.05} = 360.278 \quad \left[\frac{N}{mm^2}\right] \tag{6.32}$$

$$\lambda_{rel.m} := \sqrt[2]{\frac{f_{mk}}{\sigma_{m.crit}}} = 0.289 \tag{6.30}$$

$$k_{crit} := \begin{array}{l} \text{if } \lambda_{rel.m} \le 0.75 \\ \quad \| \; 1.0 \\ \text{else if } 0.75 < \lambda_{rel.m} \le 1.4 \\ \quad \| \; 1.56 - 0.75 \cdot \lambda_{rel.m} \\ \text{else if } 1.4 < \lambda_{rel.m} \\ \quad \| \; \dfrac{1.0}{\lambda_{rel.m}^{\;2}} \end{array} = 1 \tag{6.34}$$

Control - Bending

Requirements:

$k_{crit} := 1.0$ (Trykkdelen er fastholdt sideveis av takskivene) (6.34)

$$\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}} \le 1 \quad (6.33)$$

Utilization

$$\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}} = 0 \quad (6.33)$$

Control - Combination Bending and Axial

Requirements:

$$k_{crit} = 1 \quad (6.34)$$

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + \left(\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}}\right)^2 \le 1 \quad (6.35)$$

Utilization

$$\frac{\sigma_{c0d}}{k_{cz} \cdot f_{c0d}} + \left(\frac{\sigma_{myd}}{k_{crit} \cdot f_{myd}}\right)^2 = 0.314 \quad (6.35)$$

C4.1. Structural Fire Design - Beam

- EC5
- Reduced Cross Section Method

Dimension $[mm]$ $H:=540$ $B:=360$ $L:=9600$

Action Forces from *Robot Structural*

$M_{yd}:=325 \cdot 10^6$ $M_{zd}:=0 \cdot 10^6$ $[Nmm]$

$V_{zd}:=114 \cdot 10^3$ $V_{yd}:=0 \cdot 10^3$ $[N]$

$N_{cd}:=0 \cdot 10^3$ $N_{td}:=0 \cdot 10^3$ $[N]$

Buckling Length $[mm]$ $L_{ky}:=9600$ $L_{kz}:=9600$

Combination Factor $\psi_{fi}:=0.3$

Material Factor (NA.2.3) $\gamma_{M.fi}:=1.0$

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

Modification Factor (Glulam) Table 2.1 $k_{fi}:=1.15$

Reduction Factor $\eta_{fi}:=0.6$

Characteristic strength

Characteristic bending strength	$f_{mk} := 30 \quad \left[\frac{N}{mm^2}\right]$
Characteristic shear strength	$f_{vk} := 3.5 \quad \left[\frac{N}{mm^2}\right]$
Characteristic tension strength // grain	$f_{t0k} := 19.5 \quad \left[\frac{N}{mm^2}\right]$
Characteristic compression strength // grain	$f_{c0k} := 24.5 \quad \left[\frac{N}{mm^2}\right]$
Characteristic compression strength perpendicular to grain	$f_{c90k} := 2.5 \quad \left[\frac{N}{mm^2}\right]$
Characteristic tension strength perpendicular to grain	$f_{t90k} := 0.5 \quad \left[\frac{N}{mm^2}\right]$

Fire Design Strength

Design bending strength (y-axis)

$$f_{myd.fi} := \frac{f_{mk} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 34.5$$

Design bending strength (z-axis)

$$f_{mzd.fi} := f_{myd.fi} = 34.5$$

Design tension strength // grain

$$f_{t0d.fi} := \frac{f_{t0k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 22.425$$

Design compression strength // grain

$$f_{c0d.fi} := \frac{f_{c0k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 28.175$$

Design tension strength perpendicular to grain

$$f_{t90d.fi} := \frac{f_{t90k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 0.575$$

Design compression strength perpendicular to grain

$$f_{c90d.fi} := \frac{f_{c90k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 2.875$$

Design shear strength

$$f_{vd.fi} := \frac{f_{vk} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 4.025$$

Design Fire Load

$$M_{yd.fi} := \eta_{fi} \cdot M_{yd} = 1.95 \cdot 10^{8} \qquad M_{zd.fi} := \eta_{fi} \cdot M_{zd} \qquad [Nmm]$$

$$V_{zd.fi} := \eta_{fi} \cdot V_{zd} = 6.84 \cdot 10^{4} \qquad V_{yd.fi} := \eta_{fi} \cdot V_{yd} \qquad [N]$$

$$N_{cd.fi} := \eta_{fi} \cdot N_{cd} \qquad N_{td.fi} := \eta_{fi} \cdot N_{td} \qquad [N]$$

Parameters

$t_{req} := 90$ $[min]$

$\beta_0 := 0.65$ $\left[\dfrac{mm}{min}\right]$

$d_0 := 7$ $[mm]$

$k_0 := 1.0$

$d_{char.0} := \beta_0 \cdot t_{req}$ $[mm]$

$d_{ef} := d_{char.0} + k_0 \cdot d_0$ $[mm]$

Reduced Cross Section

$H_{ef} := H - d_{ef} = 474.5$ $[mm]$ $B_{ef} := B = 360$ $[mm]$

$A_{ef} := H_{ef} \cdot B_{ef}$ $[mm^2]$ $k_{cr} := 0.8$

$W_{y.fi} := \dfrac{1}{6} \cdot A_{ef} \cdot H_{ef}$ $[mm^3]$ $W_{z.fi} := \dfrac{1}{6} \cdot A_{ef} \cdot B_{ef}$ $[mm^3]$

Design Stresses in Fire $[MPa]$

Moment:

$\sigma_{my.fi} := \dfrac{M_{yd.fi}}{W_{y.fi}}$ $\sigma_{mz.fi} := \dfrac{M_{zd.fi}}{W_{z.fi}}$

Axial: $\sigma_{c.fi} := \dfrac{N_{cd.fi}}{A_{ef}}$ $\sigma_{t.fi} := \dfrac{N_{td.fi}}{A_{ef}}$

Shear: $\tau_{Vz.fi} := \dfrac{3}{2} \cdot \dfrac{V_{zd.fi}}{k_{cr} \cdot A_{ef}}$ $\tau_{Vy.fi} := \dfrac{3}{2} \cdot \dfrac{V_{yd.fi}}{k_{cr} \cdot A_{ef}}$

Design Check in Accordance with EC5

Bending - 6.1.6

Control check:

$k_m := 0.7$ (Glulam) (EC5: 6.1.6(2))

$$\frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \le 1 \qquad (6.11)$$

$$k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \le 1 \qquad (6.12)$$

Utilization

$$\frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.418 \qquad (6.11)$$

$$k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.293 \qquad (6.12)$$

Shear - 6.1.7

Requirements:

$$\frac{\tau_{V.fi}}{f_{vd.fi}} \le 1 \qquad (6.13)$$

Utilization

$$\frac{\tau_{Vz.fi}}{f_{vd.fi}} = 0.187 \qquad \frac{\tau_{Vy.fi}}{f_{vd.fi}} = 0 \qquad (6.13)$$

Axial (Tension) - 6.1.2

Requirements:

$$\frac{\sigma_{td.fi}}{f_{t0d.fi}} \leq 1 \qquad (6.1)$$

Utilization

$$\frac{\sigma_{t.fi}}{f_{t0d.fi}} = 0 \qquad (6.1)$$

Axial (Compression) - 6.1.4

Requirements:

$$\frac{\sigma_{cd.fi}}{f_{c0d.fi}} \leq 1 \qquad (6.2)$$

Utilization

$$\frac{\sigma_{c.fi}}{f_{c0d.fi}} = 0 \qquad (6.2)$$

Combination of Bending and Axial (tension) stress - 6.2.3

Requirements:

$k_m := 0.7$ (EC5: 6.1.6(2))

$$\frac{\sigma_{t0d}}{f_{t0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.17)$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.18)$$

Utilizations

$$\frac{\sigma_{t.fi}}{f_{t0d.fi}} + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.418 \qquad (6.17)$$

$$\frac{\sigma_{t.fi}}{f_{t0d.fi}} + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.293 \qquad (6.18)$$

Combinations of Bending and Axial (compression) stress - 6.2.4

Requirements:

$k_m := 0.7$ (EC5: 6.1.6(2))

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \le 1 \qquad (6.19)$$

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \le 1 \qquad (6.20)$$

Utilizations

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.418 \qquad (6.19)$$

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.293 \qquad (6.20)$$

Stability - Buckling - 6.3.2

$E_{0.05} := 10800 \quad [MPa]$

Buckling length (y-axis) $\quad L_{ky} = 9600 \quad [mm]$

Slenderness (y-axis)

$$\lambda_y := \frac{L_{ky}}{H} \cdot \sqrt[2]{12} = 61.584$$

$$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 0.934 \qquad (6.21)$$

Buckling length (z-aksen) $\quad L_{kz} = 9600 \quad [mm]$

Slenderness (z-axis)

$$\lambda_z := \frac{L_{kz}}{B} \cdot \sqrt[2]{12} = 92.376$$

$$\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 1.4 \qquad (6.22)$$

EC5: 6.3.2(3)

$\beta_c := 0.1$ Glulam (6.29)

$$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + {\lambda_{rel.y}}^2\right) = 0.968 \qquad (6.27)$$

$$k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + {\lambda_{rel.z}}^2\right) = 1.536 \qquad (6.28)$$

$$k_{cy} := \frac{1}{k_y + \sqrt[2]{{k_y}^2 - {\lambda_{rel.y}}^2}} = 0.819 \qquad (6.25)$$

$$k_{cz} := \frac{1}{k_z + \sqrt[2]{{k_z}^2 - {\lambda_{rel.z}}^2}} = 0.462 \qquad (6.26)$$

Control - Combination of Axial and Bending

Required:

$k_m := 0.7$ (EC5: 6.1.6(2))

$$\frac{\sigma_{c.fi}}{k_{cy} \cdot f_{c0d.fi}} + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \le 1 \qquad (6.23)$$

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \le 1 \qquad (6.24)$$

Utilizations

$$\frac{\sigma_{c.fi}}{k_{cy} \cdot f_{c0d.fi}} + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.418 \qquad (6.23)$$

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.293 \qquad (6.24)$$

Stability - LTB - 6.3.3

Checked when bending is acting alone or with compression

$$L := L_{ky} = 9600 \quad [mm]$$

$$l_{ef} := 0.9 \cdot L + 2 \cdot H = 9720 \quad [mm]$$ (Table 6.1)

$$\sigma_{m.crit} := \frac{0.78 \cdot B^2}{H \cdot l_{ef}} \cdot E_{0.05} = 208 \quad \left[\frac{N}{mm^2}\right]$$ (6.32)

$$\lambda_{rel.m} := \sqrt[2]{\frac{f_{mk}}{\sigma_{m.crit}}} = 0.38$$ (6.30)

$$k_{crit} := \begin{array}{l} \text{if } \lambda_{rel.m} \le 0.75 \\ \quad \| \; 1.0 \\ \text{else if } 0.75 < \lambda_{rel.m} \le 1.4 \\ \quad \| \; 1.56 - 0.75 \cdot \lambda_{rel.m} \\ \text{else if } 1.4 < \lambda_{rel.m} \\ \quad \| \; \dfrac{1.0}{\lambda_{rel.m}^{\;2}} \end{array} = 1$$ (6.34)

Control - Bending

Requirements:

$$k_{crit} := 1.0 \tag{6.34}$$

$$\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd}} \leq 1 \tag{6.33}$$

Utilization

$$\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd.fi}} = 0.418 \tag{6.33}$$

Control - Combination Bending and Axial

Requirements:

$$k_{crit} = 1 \tag{6.34}$$

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + \left(\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd.fi}}\right)^2 \leq 1 \tag{6.35}$$

Utilization

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + \left(\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd.fi}}\right)^2 = 0.175 \tag{6.35}$$

C4.2. Structural Fire Design - Column

- Eurocode 5 1-2
- Reduced Cross Section Method

Dimension	$[mm]$	$B := 720$	$H := 720$
Length	$[mm]$	$L := 9600$	
Buckling Length	$[mm]$	$L_{ky} := 9600$	$L_{kz} := 9600$

Action Forces from *Robot Structural*

$M_{yd} := 0$	$M_{zd} := 0$	$[Nmm]$
$V_{zd} := 0$	$V_{yd} := 0$	$[N]$
$N_{cd} := 7600 \cdot 10^3$	$N_{td} := 0$	$[N]$

Combination Factor	$\psi_{fi} := 0.3$
Material Factor (NA.2.3)	$\gamma_{M.fi} := 1.0$
Modification Factor	$k_{mod.fi} := 1.0$
Modification Factor (Glulam) Table 2.1	$k_{fi} := 1.15$
Reduction Factor	$\eta_{fi} := 0.6$

Characteristic strength

Characteristic bending strength	$f_{mk} := 30 \quad \left[\dfrac{N}{mm^2}\right]$
Characteristic shear strength	$f_{vk} := 3.5 \quad \left[\dfrac{N}{mm^2}\right]$
Characteristic tension strength // grain	$f_{t0k} := 19.5 \quad \left[\dfrac{N}{mm^2}\right]$
Characteristic compression strength // grain	$f_{c0k} := 24.5 \quad \left[\dfrac{N}{mm^2}\right]$
Characteristic compression strength perpendicular to grain	$f_{c90k} := 2.5 \quad \left[\dfrac{N}{mm^2}\right]$
Characteristic tension strength perpendicular to grain	$f_{t90k} := 0.5 \quad \left[\dfrac{N}{mm^2}\right]$

Fire Design Strength

Design bending strength (y-axis)

$$f_{myd.fi} := \frac{f_{mk} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 34.5$$

Design bending strength (z-axis)

$$f_{mzd.fi} := f_{myd.fi} = 34.5$$

Design tension strength // grain

$$f_{t0d.fi} := \frac{f_{t0k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 22.425$$

Design compression strength // grain

$$f_{c0d.fi} := \frac{f_{c0k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 28.175$$

Design tension strength perpendicular to grain

$$f_{t90d.fi} := \frac{f_{t90k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 0.575$$

Design compression strength perpendicular to grain

$$f_{c90d.fi} := \frac{f_{c90k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 2.875$$

Design shear strength

$$f_{vd} := \frac{f_{vk} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 4.025$$

Design Fire Load

$M_{yd.fi} := \eta_{fi} \cdot M_{yd} = 0$ $\quad$ $M_{zd.fi} := \eta_{fi} \cdot M_{zd} = 0$ $\quad$ $[Nmm]$

$V_{zd.fi} := \eta_{fi} \cdot V_{zd} = 0$ $\quad$ $V_{yd.fi} := \eta_{fi} \cdot V_{yd} = 0$ $\quad$ $[N]$

$N_{cd.fi} := \eta_{fi} \cdot N_{cd} = 4.56 \cdot 10^{6}$ $\quad$ $N_{td.fi} := \eta_{fi} \cdot N_{td} = 0$ $\quad$ $[N]$

Parameters

$t_{req} := 90$ $[min]$

$\beta_n := 0.7$ $\left[\dfrac{mm}{min}\right]$

$d_0 := 7$ $[mm]$

$k_0 := 1.0$

$d_{char.n} := \beta_n \cdot t_{req}$ $[mm]$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $[mm]$

Reduced Cross Section

$H_{ef} := H - 2 \cdot d_{ef} = 580$ $[mm]$ $B_{ef} := B - 2 \cdot d_{ef} = 580$ $[mm]$

$A_{ef} := H_{ef} \cdot B_{ef}$ $\left[mm^2\right]$ $k_{cr} := 0.8$ $\left[mm^2\right]$

$W_{y.fi} := \dfrac{1}{6} \cdot A_{ef} \cdot H_{ef}$ $\left[mm^3\right]$ $W_{z.fi} := \dfrac{1}{6} \cdot A_{ef} \cdot B_{ef}$ $\left[mm^3\right]$

Design Stresses in Fire $[MPa]$

Moment: $\sigma_{my.fi} := \dfrac{M_{yd.fi}}{W_{y.fi}}$ $\sigma_{mz.fi} := \dfrac{M_{zd.fi}}{W_{z.fi}}$

Axial: $\sigma_{c.fi} := \dfrac{N_{cd.fi}}{A_{ef}}$ $\sigma_{t.fi} := \dfrac{N_{td.fi}}{A_{ef}}$

Shear: $\tau_{Vz.fi} := \dfrac{3}{2} \cdot \dfrac{V_{zd.fi}}{k_{cr} \cdot A_{ef}}$ $\tau_{Vy.fi} := \dfrac{3}{2} \cdot \dfrac{V_{yd.fi}}{k_{cr} \cdot A_{ef}}$

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Design Check in Accordance to Eurocode 5

Bending - 6.1.6

Control check:

$k_m := 0.7$ (Glulam) (EC5: 6.1.6(2))

$$\frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \le 1 \qquad (6.11)$$

$$k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \le 1 \qquad (6.12)$$

Utilization

$$\frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0 \qquad (6.11)$$

$$k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0 \qquad (6.12)$$

Shear - 6.1.7

Requirements:

$$\frac{\tau_{V.fi}}{f_{vd}} \le 1 \qquad (6.13)$$

Utilization

$$\frac{\tau_{Vz.fi}}{f_{vd}} = 0 \qquad \frac{\tau_{Vy.fi}}{f_{vd}} = 0 \qquad (6.13)$$

Axial (Tension) - 6.1.2

Requirements:

$$\frac{\sigma_{td.fi}}{f_{t0d.fi}} \leq 1 \qquad (6.1)$$

Utilization

$$\frac{\sigma_{t.fi}}{f_{t0d.fi}} = 0 \qquad (6.1)$$

Axial (Compression) - 6.1.4

Requirements:

$$\frac{\sigma_{cd.fi}}{f_{c0d.fi}} \leq 1 \qquad (6.2)$$

Utilization

$$\frac{\sigma_{c.fi}}{f_{c0d.fi}} = 0.481 \qquad (6.2)$$

Combination of Bending and Axial (tension) stress - 6.2.3

Requirements:

$$k_m := 0.7$$ (EC5: 6.1.6(2))

$$\frac{\sigma_{t0d}}{f_{t0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.17)$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \le 1 \qquad (6.18)$$

Utilizations

$$\frac{\sigma_{t.fi}}{f_{t0d.fi}} + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0 \qquad (6.17)$$

$$\frac{\sigma_{t.fi}}{f_{t0d.fi}} + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0 \qquad (6.18)$$

Combinations of Bending and Axial (compression) stress - 6.2.4

Requirements:

$$k_m := 0.7$$ (EC5: 6.1.6(2))

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \le 1 \qquad (6.19)$$

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \le 1 \qquad (6.20)$$

Utilizations

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.231 \qquad (6.19)$$

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.231 \qquad (6.20)$$

Stability - Buckling - 6.3.2

$E_{0.05} := 10800 \quad [MPa]$

Buckling length (y-axis) $L_{ky} = 9600 \quad [mm]$

Slenderness (y-axis)

$$\lambda_y := \frac{L_{ky}}{H} \cdot \sqrt[2]{12} = 46.188$$

$$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 0.7 \qquad (6.21)$$

Buckling length (z-aksen) $L_{kz} = 9600 \quad [mm]$

Slenderness (z-axis)

$$\lambda_z := \frac{L_{kz}}{B} \cdot \sqrt[2]{12} = 46.188$$

$$\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 0.7 \qquad (6.22)$$

EC5: 6.3.2(3)

$\beta_c := 0.1$ Glulam (6.29)

$$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + \lambda_{rel.y}{}^2\right) = 0.765 \qquad (6.27)$$

$$k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + \lambda_{rel.z}{}^2\right) = 0.765 \qquad (6.28)$$

$$k_{cy} := \frac{1}{k_y + \sqrt[2]{k_y{}^2 - \lambda_{rel.y}{}^2}} = 0.931 \qquad (6.25)$$

$$k_{cz} := \frac{1}{k_z + \sqrt[2]{k_z{}^2 - \lambda_{rel.z}{}^2}} = 0.931 \qquad (6.26)$$

Control - Combination of Axial and Bending

Required:

$k_m := 0.7$ (EC5: 6.1.6(2))

$$\frac{\sigma_{c.fi}}{k_{cy} \cdot f_{c0d.fi}} + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \le 1 \qquad (6.23)$$

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \le 1 \qquad (6.24)$$

Utilizations

$$\frac{\sigma_{c.fi}}{k_{cy} \cdot f_{c0d.fi}} + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.517 \qquad (6.23)$$

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.517 \qquad (6.24)$$

Stability - LTB - 6.3.3

Checked when bending is acting alone or with compression

$$L := L_{ky} = 9600 \quad [mm]$$

$$l_{ef} := 0.9 \cdot L + 2 \cdot H = 10080 \quad [mm] \qquad \text{(Table 6.1)}$$

$$\sigma_{m.crit} := \frac{0.78 \cdot B^2}{H \cdot l_{ef}} \cdot E_{0.05} = 601.714 \quad \left[\frac{N}{mm^2}\right] \qquad (6.32)$$

$$\lambda_{rel.m} := \sqrt[2]{\frac{f_{mk}}{\sigma_{m.crit}}} = 0.223 \qquad (6.30)$$

$$k_{crit} := \begin{array}{l} \text{if } \lambda_{rel.m} \le 0.75 \\ \quad \| \; 1.0 \\ \text{else if } 0.75 < \lambda_{rel.m} \le 1.4 \\ \quad \| \; 1.56 - 0.75 \cdot \lambda_{rel.m} \\ \text{else if } 1.4 < \lambda_{rel.m} \\ \quad \| \; \dfrac{1.0}{{\lambda_{rel.m}}^2} \end{array} = 1 \qquad (6.34)$$

Control - Bending

Requirements:

$$k_{crit} := 1.0 \tag{6.34}$$

$$\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd}} \le 1 \tag{6.33}$$

Utilization

$$\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd.fi}} = 0 \tag{6.33}$$

Control - Combination Bending and Axial

Requirements:

$$k_{crit} = 1 \tag{6.34}$$

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + \left(\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd.fi}}\right)^2 \le 1 \tag{6.35}$$

Utilization

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + \left(\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd.fi}}\right)^2 = 0.517 \tag{6.35}$$

C4.3. Structural Fire Design - Diagonal

- EC5 1-2
- Reduced Cross Section Method

Dimension $[mm]$ $H := 585$ $B := 450$

Action Forces from *Robot Structural Analysis*

$M_{yd} := 0 \cdot 10^6$ $M_{zd} := 0 \cdot 10^6$ $[Nmm]$

$V_{zd} := 0 \cdot 10^3$ $V_{yd} := 0 \cdot 10^3$ $[N]$

$N_{cd} := 2400 \cdot 10^3$ $N_{td} := 2400 \cdot 10^3$ $[N]$

Buckling Length $[mm]$ $L_{ky} := 11650$ $L_{kz} := 0$ *Restrained in weak direction*

Combination Factor $\psi_{fi} := 0.3$

Material Factor (NA.2.3) $\gamma_{M.fi} := 1.0$

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

Modification Factor (Glulam) Table 2.1 $k_{fi} := 1.15$

Reduction Factor $\eta_{fi} := 0.6$

Characteristic strength

Characteristic bending strength	$f_{mk} := 30 \quad \left[\frac{N}{mm^2}\right]$
Characteristic shear strength	$f_{vk} := 3.5 \quad \left[\frac{N}{mm^2}\right]$
Characteristic tension strength // grain	$f_{t0k} := 19.5 \quad \left[\frac{N}{mm^2}\right]$
Characteristic compression strength // grain	$f_{c0k} := 24.5 \quad \left[\frac{N}{mm^2}\right]$
Characteristic compression strength perpendicular to grain	$f_{c90k} := 2.5 \quad \left[\frac{N}{mm^2}\right]$
Characteristic tension strength perpendicular to grain	$f_{t90k} := 0.5 \quad \left[\frac{N}{mm^2}\right]$

Fire Design Strength

Design bending strength (y-axis)

$$f_{myd.fi} := \frac{f_{mk} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 34.5$$

Design bending strength (z-axis)

$$f_{mzd.fi} := f_{myd.fi} = 34.5$$

Design tension strength // grain

$$f_{t0d.fi} := \frac{f_{t0k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 22.425$$

Design compression strength // grain

$$f_{c0d.fi} := \frac{f_{c0k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 28.175$$

Design tension strength perpendicular to grain

$$f_{t90d.fi} := \frac{f_{t90k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 0.575$$

Design compression strength perpendicular to grain

$$f_{c90d.fi} := \frac{f_{c90k} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 2.875$$

Design shear strength

$$f_{vd.fi} := \frac{f_{vk} \cdot k_{mod.fi}}{\gamma_{M.fi}} \cdot k_{fi} = 4.025$$

Design Fire Load

$$M_{yd.fi} := \eta_{fi} \cdot M_{yd} = 0 \qquad M_{zd.fi} := \eta_{fi} \cdot M_{zd} \qquad [Nmm]$$

$$V_{zd.fi} := \eta_{fi} \cdot V_{zd} = 0 \qquad V_{yd.fi} := \eta_{fi} \cdot V_{yd} \qquad [N]$$

$$N_{cd.fi} := \eta_{fi} \cdot N_{cd} = 1.44 \cdot 10^6 \qquad N_{td.fi} := \eta_{fi} \cdot N_{td} = 1.44 \cdot 10^6 \qquad [N]$$

Parameters

$t_{req} := 90$ $[min]$

$\beta_n := 0.7$ $\left[\dfrac{mm}{min}\right]$

$d_0 := 7$ $[mm]$

$k_0 := 1.0$

$d_{char.n} := \beta_n \cdot t_{req}$ $[mm]$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $[mm]$

Reduced Cross Section

$H_{ef} := H - 2 \cdot d_{ef} = 445$ $[mm]$ $B_{ef} := B - d_{ef} = 380$ $[mm]$

$A_{ef} := H_{ef} \cdot B_{ef}$ $[mm^2]$ $k_{cr} := 0.8$ $[mm^2]$

$W_{y.fi} := \dfrac{1}{6} \cdot A_{ef} \cdot H_{ef}$ $[mm^3]$ $W_{z.fi} := \dfrac{1}{6} \cdot A_{ef} \cdot B_{ef}$ $[mm^3]$

Design Stresses in Fire $[MPa]$

Moment: $\sigma_{my.fi} := \dfrac{M_{yd.fi}}{W_{y.fi}}$ $\sigma_{mz.fi} := \dfrac{M_{zd.fi}}{W_{z.fi}}$

Axial: $\sigma_{c.fi} := \dfrac{N_{cd.fi}}{A_{ef}}$ $\sigma_{t.fi} := \dfrac{N_{td.fi}}{A_{ef}}$

Shear: $\tau_{Vz.fi} := \dfrac{3}{2} \cdot \dfrac{V_{zd.fi}}{k_{cr} \cdot A_{ef}}$ $\tau_{Vy.fi} := \dfrac{3}{2} \cdot \dfrac{V_{yd.fi}}{k_{cr} \cdot A_{ef}}$

Design Check in Accordance to EC5

Bending - 6.1.6

Control check:

$k_m := 0.7$ (Glulam) (EC5: 6.1.6(2))

$$\frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \le 1 \qquad (6.11)$$

$$k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \le 1 \qquad (6.12)$$

Utilization

$$\frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0 \qquad (6.11)$$

$$k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0 \qquad (6.12)$$

Shear - 6.1.7

Requirements:

$$\frac{\tau_{V.fi}}{f_{vd.fi}} \le 1 \qquad (6.13)$$

Utilization

$$\frac{\tau_{Vz.fi}}{f_{vd.fi}} = 0 \qquad \frac{\tau_{Vy.fi}}{f_{vd.fi}} = 0 \qquad (6.13)$$

Axial (Tension) - 6.1.2

Requirements:

$$\frac{\sigma_{td.fi}}{f_{t0d.fi}} \leq 1 \tag{6.1}$$

Utilization

$$\frac{\sigma_{t.fi}}{f_{t0d.fi}} = 0.38 \tag{6.1}$$

Axial (Compression) - 6.1.4

Requirements:

$$\frac{\sigma_{cd.fi}}{f_{c0d.fi}} \leq 1 \tag{6.2}$$

Utilization

$$\frac{\sigma_{c.fi}}{f_{c0d.fi}} = 0.302 \tag{6.2}$$

Combination of Bending and Axial (tension) stress - 6.2.3

Requirements:

$k_m := 0.7$ (EC5: 6.1.6(2))

$$\frac{\sigma_{t0d}}{f_{t0d}} + \frac{\sigma_{myd}}{f_{myd}} + k_m \cdot \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \quad (6.17)$$

$$\frac{\sigma_{t0d}}{f_{t0d}} + k_m \cdot \frac{\sigma_{myd}}{f_{myd}} + \frac{\sigma_{mzd}}{f_{mzd}} \leq 1 \quad (6.18)$$

Utilizations

$$\frac{\sigma_{t.fi}}{f_{t0d.fi}} + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.38 \quad (6.17)$$

$$\frac{\sigma_{t.fi}}{f_{t0d.fi}} + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.38 \quad (6.18)$$

Combinations of Bending and Axial (compression) stress - 6.2.4

Requirements:

$k_m := 0.7$ (EC5: 6.1.6(2))

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \quad (6.19)$$

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \leq 1 \quad (6.20)$$

Utilizations

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.091 \quad (6.19)$$

$$\left(\frac{\sigma_{c.fi}}{f_{c0d.fi}}\right)^2 + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.091 \quad (6.20)$$

Stability - Buckling - 6.3.2

$E_{0.05} := 10800$

Buckling length (y-axis) $L_{ky} = 11650$ $[mm]$

Slenderness (y-axis)

$$\lambda_y := \frac{L_{ky}}{H} \cdot \sqrt[2]{12} = 68.986$$

$$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 1.046 \qquad (6.21)$$

Buckling length (z-aksen) $L_{kz} = 0$ $[mm]$

Slenderness (z-axis)

$$\lambda_z := \frac{L_{kz}}{B} \cdot \sqrt[2]{12} = 0$$

$$\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt[2]{\frac{f_{c0k}}{E_{0.05}}} = 0 \qquad (6.22)$$

EC5: 6.3.2(3)

$\beta_c := 0.1$ Glulam (6.29)

$$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + \lambda_{rel.y}^{2}\right) = 1.084 \qquad (6.27)$$

$$k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + \lambda_{rel.z}^{2}\right) = 0.485 \qquad (6.28)$$

$$k_{cy} := \frac{1}{k_y + \sqrt[2]{k_y^{2} - \lambda_{rel.y}^{2}}} = 0.73 \qquad (6.25)$$

$$k_{cz} := \frac{1}{k_z + \sqrt[2]{k_z^{2} - \lambda_{rel.z}^{2}}} = 1.031 \qquad (6.26)$$

Control - Combination of Axial and Bending

Required:

$$k_m := 0.7$$ (EC5: 6.1.6(2))

$$\frac{\sigma_{c.fi}}{k_{cy} \cdot f_{c0d.fi}} + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \le 1 \qquad (6.23)$$

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} \le 1 \qquad (6.24)$$

Utilizations

$$\frac{\sigma_{c.fi}}{k_{cy} \cdot f_{c0d.fi}} + \frac{\sigma_{my.fi}}{f_{myd.fi}} + k_m \cdot \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.414 \qquad (6.23)$$

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + k_m \cdot \frac{\sigma_{my.fi}}{f_{myd.fi}} + \frac{\sigma_{mz.fi}}{f_{mzd.fi}} = 0.293 \qquad (6.24)$$

Stability - LTB - 6.3.3

Checked when bending is acting alone or with compression

$$L := L_{ky} = 11650 \quad [mm]$$

$$l_{ef} := 0.9 \cdot L + 2 \cdot H = 11655 \quad [mm] \qquad \text{(Table 6.1)}$$

$$\sigma_{m.crit} := \frac{0.78 \cdot B^2}{H \cdot l_{ef}} \cdot E_{0.05} = 250.193 \quad \left[\frac{N}{mm^2}\right] \qquad (6.32)$$

$$\lambda_{rel.m} := \sqrt[2]{\frac{f_{mk}}{\sigma_{m.crit}}} = 0.346 \qquad (6.30)$$

$$k_{crit} := \begin{cases} 1.0 & \text{if } \lambda_{rel.m} \le 0.75 \\ 1.56 - 0.75 \cdot \lambda_{rel.m} & \text{else if } 0.75 < \lambda_{rel.m} \le 1.4 \\ \dfrac{1.0}{\lambda_{rel.m}^{\ 2}} & \text{else if } 1.4 < \lambda_{rel.m} \end{cases} = 1 \qquad (6.34)$$

Control - Bending

Requirements:

$$k_{crit} := 1.0 \tag{6.34}$$

$$\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd}} \le 1 \tag{6.33}$$

Utilization

$$\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd.fi}} = 0 \tag{6.33}$$

Control - Combination Bending and Axial

Requirements:

$$k_{crit} = 1 \tag{6.34}$$

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + \left(\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd.fi}}\right)^2 \le 1 \tag{6.35}$$

Utilization

$$\frac{\sigma_{c.fi}}{k_{cz} \cdot f_{c0d.fi}} + \left(\frac{\sigma_{my.fi}}{k_{crit} \cdot f_{myd.fi}}\right)^2 = 0.293 \tag{6.35}$$

C5. Peak Acceleration Calculation

- NS-EN1991-1-4

General remarks:

Units used in script:

- Length/height: $[m]$
- Force: $[N]$
- Velocity: $\left[\frac{m}{s}\right]$
- Density: $\left[\frac{kg}{m^3}\right]$

All equations- and chapter-references are from the EC1-1-4.

Geometry of the building:

Height: $h := 66$

Width: $b_x := 32$

Depth: $b_y := 19.2$

Fundamental values:

Damping coefficient: $\xi := \frac{1.9}{100} = 0.019$

Zeta faktor: $\zeta := 1.0$

Reference height: $z_s := 0.6 \cdot h$ (Figure 6.1)

Exact mode function value at top of building, extracted from *Robot:*

$\phi_{1.x}(h) := 1$

$\phi_{1.y}(h) := 1$

Exact mode function value at top floor of building, extracted from *Robot:*

$\phi_{1.x.tfl} := 0.97$

$\phi_{1.y.tfl} := 0.96$

Natural frequencies of building:

$n_{1.x} := 0.45$

$n_{1.y} := 0.55$

Air density: $\rho := 1.25$ (Chapter 4.5)

Basic wind velocity:

Since the probability factor for the calculation of acceleration is different from the probability factor in the calculation of static wind load we must calculate the basic wind velocity once again.

The fundamental value of basic wind velocity:

$v_{b.0} := 22$ (Table NA.4)

Probability factor:
For acceleration calculation we set return period T=1, which gives cprob=0,73.

$c_{prob} := 0.73$

Directional factor: $c_{dir} := 1$ (Chapter 4.2(2), NOTE2)

Season factor: $c_{season} := 1$ (Chapter 4.2(2), NOTE3)

Factor for the wind increasing with the height over the sea:

$c_{alt} := 1$ (Table NA.4(901.3))

Basic wind velocity: $v_b := c_{dir} \cdot c_{season} \cdot c_{alt} \cdot c_{prob} \cdot v_{b.0}$ (eq. NA.4.1)

$v_b = 16.06$

Mean wind velocity:

Since the reference height for the calculation of acceleration (zs = 0,6h) is different from the reference height in the calculation of static wind load (ze = h) we must calculate the mean wind velocity once again.

Terrain category: $TK := 4$ (Table 4.1)

Orography factor: $c_0 := 1$ (Chapter 4.3.1, NOTE 1)

Roughness length: (Chapter 4.3.2)

$$z_0 := \begin{array}{l} \text{if } TK = 0 \\ \quad 0.003 \\ \text{else if } TK = 1 \\ \quad 0.01 \\ \text{else if } TK = 2 \\ \quad 0.05 \\ \text{else if } TK = 3 \\ \quad 0.3 \\ \text{else if } TK = 4 \\ \quad 1 \end{array} = 1$$

Minimum height: (Table 4.1)

$$z_{min} := \begin{array}{l} \text{if } TK = 0 \\ \quad 2 \\ \text{else if } TK = 1 \\ \quad 2 \\ \text{else if } TK = 2 \\ \quad 4 \\ \text{else if } TK = 3 \\ \quad 8 \\ \text{else if } TK = 4 \\ \quad 16 \end{array} = 16$$

Max. height: $z_{max} := 200$ (Chapter 4.3.2)

Terrain factor: $k_r := 0.24$ (Table NA.4.1)

Roughness factor: $$c_r(z) := \text{if } (z \geq z_{min}) \wedge (z \leq z_{max}) \;\; k_r \cdot \ln\left(\frac{z}{z_0}\right) \;\; \text{else if } (z \leq z_{min}) \;\; c_r(z_{min})$$ (Eq. 4.4)

Mean wind velocity: $$v_m(z) := c_0 \cdot c_r(z) \cdot v_b$$ (Eq. 4.3)

$$v_m(z_s) = 14.18$$

Wind turbulence:

Turbulence factor: $$k_I := 1$$ (Eq. 4.7)

Standard deviation: $$\sigma_v := k_r \cdot v_b \cdot k_I$$ (Eq. 4.6)

Turbulence intensity: $$I_v(z) := \text{if } (z \geq z_{min}) \wedge (z \leq z_{max}) \;\; \frac{\sigma_v}{v_m(z)} \;\; \text{else if } (z \leq z_{min}) \;\; I_v(z_{min})$$ (Eq. 4.7)

Non-dimensional power spectral density function:

Roughness length: $z_0 = 1$

Reference height: $z_t := 200$ (eq. B.1)

Reference length scale: $L_t := 300$ (eq. B.1)

Factor: $\alpha := 0.67 + 0.05 \cdot \ln\left(z_0\right) = 0.67$ (eq. B.1)

Turbulent length scale:

$$L(z) := \begin{cases} L_t \cdot \left(\dfrac{z}{z_t}\right)^{\alpha} & \text{if } z \geq z_{min} \\ L\left(z_{min}\right) & \text{else} \end{cases}$$

Non-dimensional frequency: $f_L(z, n) := \dfrac{n \cdot L(z)}{v_m(z)}$

Non-dimensional power spectral density function:

$$S_L(z, n) := \frac{6.8 \cdot f_L(z, n)}{\left(1 + 10.2 \cdot f_L(z, n)\right)^{\frac{5}{3}}}$$

Aerodynamic admittance factors:

The aerodynamic admittance functions R_h and R_b for a fundamental mode shape may be approximated using Expressions (B.7) and (B.8).

x-direction:

$$\eta_{h.x} := 4.6 \cdot \frac{h}{L(z_s)} \cdot f_L(z_s, n_{1.x})$$

$$\eta_{b.x} := 4.6 \cdot \frac{b_y}{L(z_s)} \cdot f_L(z_s, n_{1.x})$$

(eq. B.7)

y-direction:

$$\eta_{h.y} := 4.6 \cdot \frac{h}{L(z_s)} \cdot f_L(z_s, n_{1.y})$$

$$\eta_{b.y} := 4.6 \cdot \frac{b_x}{L(z_s)} \cdot f_L(z_s, n_{1.y})$$

x-direction:

$$R_{h.x} := \frac{1}{\eta_{h.x}} - \frac{1}{2 \cdot \eta_{h.x}^{\ 2}} \cdot \left(1 - e^{-2 \cdot \eta_{h.x}}\right)$$

(eq. B.8)

$$R_{b.x} := \frac{1}{\eta_{b.x}} - \frac{1}{2 \cdot \eta_{b.x}^{\ 2}} \cdot \left(1 - e^{-2 \cdot \eta_{b.x}}\right)$$

y-direction:

$$R_{h.y} := \frac{1}{\eta_{h.y}} - \frac{1}{2 \cdot \eta_{h.y}^{\ 2}} \cdot \left(1 - e^{-2 \cdot \eta_{h.y}}\right)$$

$$R_{b.y} := \frac{1}{\eta_{b.y}} - \frac{1}{2 \cdot \eta_{b.y}^{\ 2}} \cdot \left(1 - e^{-2 \cdot \eta_{b.y}}\right)$$

Equivalent mass and dimensionless coefficient:

The equivalent mass per unit length will be calculated acc. to equation F.14. The dimensionless coefficient will be calculated acc. to equation B.11. They both depend on the modal shape of the building, which will be extracted from the modal analysis in Robot Structures. The mass of each floor, which is needed for the calculation of equivalent mass, will also be extracted from Robot. We place the degrees of freedom at the top of each floor. Since the modal values are found directly from the modal analysis in Robot, we can use summation instead of integration over the height of the building. These summations, both for equivalent mass and for the dimensionless coefficient, will be done in a separate excel sheet.

Mass from excel sheet (Appendix C5.1):

x-direction: $m_{e.x} := 53209$

y-direction: $m_{e.y} := 53218$

Dimensionless coefficient from excel sheet (Appendix C5.1):

x-direction: $K_x := 1.46$

y-direction: $K_y := 1.50$

Force coefficient:

Calculated according to chapter 7, eq. 7.9.

Force coefficient for rectangular cross section and sharp edges, and without free-end flow are given in figure 7.23:

Length-to-depth ratios:
(d/b in figure 7.23)

x-direction: $r_x := \dfrac{b_x}{b_y} = 1.667$

y-direction: $r_y := \dfrac{b_y}{b_x} = 0.6$

Force coefficients of rectangular cross sections with sharp corners and without free end flow:

We interpret the whole building as a rectangular cross section.

x-direction: $c_{f.0.x} := 1.8$

(Figure 7.23)

y-direction: $c_{f.0.y} := 2.35$

Reduction factor: $\psi_r := 1$ (NA.7)

Effective slenderness according to table 7.16:
For l=38 m we must interpolate by linear interpolation to find the slenderness.

Interpolation gives:

$$k(z) := \frac{1.4 \cdot z - 21}{35} - \frac{2 \cdot z - 100}{35}$$

$$\lambda_x := min\left(k(h) \cdot \frac{h}{b_x}, 70\right) = 2.322$$

$$\lambda_y := min\left(k(h) \cdot \frac{h}{b_y}, 70\right) = 3.87$$

Solidity ratio: $\varphi := 1$

End-effect factor:

The end effect factor accounts for reduced force caused by wind flow around the ends of a finite section. The force coefficients cf0 are based on measurements on structures without free-end flow away from the ground. The end-effect factor takes into account the reduced resistance of the structure due to the wind flow around the end (end-effect). We use figure 7.36 which is based on measurements in low-turbulent flow.

$\psi_{\lambda.x} := 0.63$

(Figure 7.36)

$\psi_{\lambda.y} := 0.65$

Force-factor:

$c_{f.x} := c_{f.0.x} \cdot \psi_{\lambda.x} \cdot \psi_r = 1.134$

(Eq. 7.19)

$c_{f.y} := c_{f.0.y} \cdot \psi_{\lambda.y} \cdot \psi_r = 1.528$

The force coefficients give overall loads on the whole structure. In effect, they represent the integration of the surface pressure distribution.

Logarithmic decrement of damping:

We will be putting the total damping of the structure equal to 1,9% based on the value given by Sweco. This damping is assumed to be the structural damping, meaning that we put aerodynamic damping and damping from special devices equal to zero.

Logarithmic decrement of structural damping:

$$\delta_s := 2 \cdot \pi \cdot \frac{\xi}{\sqrt[2]{1-\xi^2}} = 0.119$$

Logarithmic decrement of Aerodynamic damping:

$\delta_{a.x} := 0$

Ignored for this project

$\delta_{a.y} := 0$

Logarithmic decrement of damping from special devices:

$$\delta_d := 0$$

Logarithmic decrement of damping:

$$\delta_x := \delta_s + \delta_{a.x} + \delta_d = 0.119$$

(Eq. F.15)

$$\delta_y := \delta_s + \delta_{a.y} + \delta_d = 0.119$$

$$\delta := \delta_s = 0.119$$ (Same for both directions)

Resonance response factor:

The resonance response factor squared R allowing for turbulence in resonance with the considered vibration mode of the structure should be determined using Expression (B.6)

x-direction: $$R_x := \sqrt[2]{\left(\frac{\pi^2}{2 \cdot \delta} \cdot S_L\left(z_s, n_{1.x}\right) \cdot R_{h.x} \cdot R_{b.x}\right)}$$ (Eq. B.6)

y-direction: $$R_y := \sqrt[2]{\left(\frac{\pi^2}{2 \cdot \delta} \cdot S_L\left(z_s, n_{1.y}\right) \cdot R_{h.y} \cdot R_{b.y}\right)}$$ (Eq. B.6)

Background factor:

x-direction: $$B_x := \sqrt[2]{\left(\frac{1}{1 + 0.9 \cdot \left(\frac{b_y + h}{L\left(z_s\right)}\right)^{0.63}}\right)} = 0.744$$ (Eq. B.3)

y-direction: $$B_y := \sqrt[2]{\left(\frac{1}{1 + 0.9 \cdot \left(\frac{b_x + h}{L\left(z_s\right)}\right)^{0.63}}\right)} = 0.729$$ (Eq. B.3)

Up-crossing frequency:

We put the up-crossing frequency equal to the natural frequency of the building. (Annex B in EC1-1-4)

$\nu_x := n_{1.x} = 0.45$

$\nu_y := n_{1.y} = 0.55$

Peak factor:

Averaging time for the mean wind velocity:

$T := 600$

Peak factor:

x-direction: $k_{p1.x} := \sqrt[2]{2 \cdot \ln(\nu_x \cdot T)} + \dfrac{0.6}{\sqrt[2]{2 \cdot \ln(\nu_x \cdot T)}} = 3.525$

$k_{p2.x} := 3$

$k_{p.x} := \max(k_{p1.x}, k_{p2.x}) = 3.525$ (Eq. B.2)

y-direction: $k_{p1.y} := \sqrt[2]{2 \cdot \ln(\nu_y \cdot T)} + \dfrac{0.6}{\sqrt[2]{2 \cdot \ln(\nu_y \cdot T)}} = 3.582$

$k_{p2.y} := 3$

$k_{p.y} := \max(k_{p1.y}, k_{p2.y}) = 3.582$ (Eq. B.2)

Standard deviation:

Standard deviation for the top (roof) of the building: (Eq. B.10)

x-direction: $$\sigma_{a.x}(z) := \frac{c_{f.x} \cdot \rho \cdot b_y \cdot I_v(z_s) \cdot v_m(z_s)^2}{m_{e.x}} \cdot R_x \cdot K_x \cdot \phi_{1.x}(z)$$

$$\sigma_{a.x} := \sigma_{a.x}(h) = 0.011$$

y-direction: $$\sigma_{a.y}(z) := \frac{c_{f.y} \cdot \rho \cdot b_x \cdot I_v(z_s) \cdot v_m(z_s)^2}{m_{e.y}} \cdot R_y \cdot K_y \cdot \phi_{1.y}(z)$$

$$\sigma_{a.y} := \sigma_{a.y}(h) = 0.016$$

Standard deviation for the top floor of the building: (Eq. B.10)

x-direction: $$\sigma_{a.x.tfl}(z) := \frac{c_{f.x} \cdot \rho \cdot b_y \cdot I_v(z_s) \cdot v_m(z_s)^2}{m_{e.x}} \cdot R_x \cdot K_x \cdot \phi_{1.x.tfl}$$

$$\sigma_{a.x.tfl} := \sigma_{a.x.tfl}(h) = 0.011$$

y-direction: $$\sigma_{a.y.tfl}(z) := \frac{c_{f.y} \cdot \rho \cdot b_x \cdot I_v(z_s) \cdot v_m(z_s)^2}{m_{e.y}} \cdot R_y \cdot K_y \cdot \phi_{1.y.tfl}$$

$$\sigma_{a.y.tfl} := \sigma_{a.y.tfl}(h) = 0.015$$

Peak acceleration:

Peak acceleration on roof:

$$a_x := \sigma_{a.x} \cdot k_{p.x} = 0.039$$

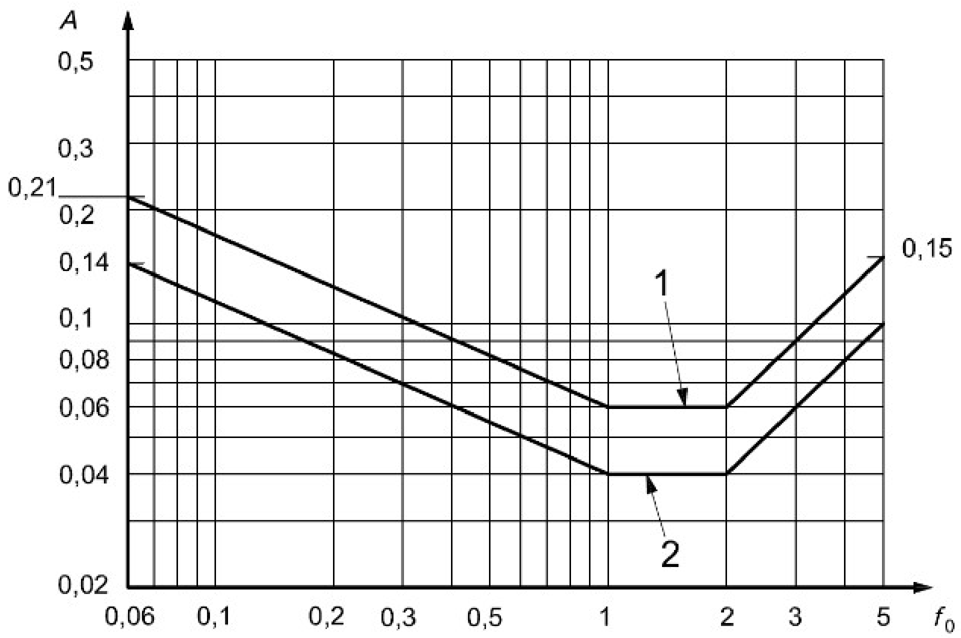
$$a_y := \sigma_{a.y} \cdot k_{p.y} = 0.058$$

Peak acceleration on top Floor :

$$a_{x.tfl} := \sigma_{a.x.tfl} \cdot k_{p.x} = 0.038$$

$$a_{y.tfl} := \sigma_{a.y.tfl} \cdot k_{p.y} = 0.055$$

Requirement from ISO10137



C5.1 Equivalent Mass and Dimensionless Coefficient

Equivalent Mass and Dimensionless Coefficient in x-direction

Dimensionless Coefficient

n	z	$\Phi_{1.x}(z)$	$v_m(z)$	$v_m^2 \cdot \Phi_{1.x}(z)$	$\Phi_{1.x}(z)^2$	$v_m(z_s)^2$
1,00	5,00	0,07	14,64	15,86	0,01	377,00
2,00	10,00	0,15	14,64	32,57	0,02	
3,00	13,50	0,21	14,64	44,79	0,04	
4,00	17,00	0,27	14,96	59,52	0,07	
5,00	20,50	0,32	15,95	81,13	0,10	
6,00	24,00	0,38	16,78	106,71	0,14	
7,00	27,50	0,44	17,50	135,35	0,20	
8,00	31,00	0,51	18,13	166,01	0,26	
9,00	34,50	0,57	18,70	198,54	0,32	
10,00	38,00	0,63	19,21	231,65	0,39	
11,00	41,50	0,68	19,67	264,70	0,47	
12,00	45,00	0,73	20,10	296,51	0,54	
13,00	48,50	0,79	20,50	329,74	0,62	
14,00	52,00	0,84	20,86	363,45	0,70	
15,00	55,50	0,88	21,21	397,12	0,78	
16,00	59,00	0,93	21,53	429,66	0,86	
17,00	62,50	0,97	21,83	460,51	0,93	
18,00	66,00	1,00	22,12	489,34	1,00	
			Sum:	*4103,17*	*7,45*	
				Kx =	**1,46**	

Equivalent mass

n	[m]	[kg]	[kg/m]	$\boldsymbol{\Phi_{1.x}(z)}$	$\Phi_{1.x}(z)^2$	$m(z) \cdot \Phi_{1.x}(z)^2$
1,00	5,00	352359,59	70471,92	0,07	0,01	385,90
2,00	5,00	201996,31	40399,26	0,15	0,02	933,38
3,00	3,50	185519,64	53005,61	0,21	0,04	2315,34
4,00	3,50	185519,64	53005,61	0,27	0,07	3750,47
5,00	3,50	185519,64	53005,61	0,32	0,10	5393,90
6,00	3,50	185519,64	53005,61	0,38	0,14	7613,78
7,00	3,50	185519,64	53005,61	0,44	0,20	10355,39
8,00	3,50	185519,64	53005,61	0,51	0,26	13517,76
9,00	3,50	185519,64	53005,61	0,57	0,32	17100,88
10,00	3,50	185519,64	53005,61	0,63	0,39	20904,57
11,00	3,50	185519,64	53005,61	0,68	0,47	24798,99
12,00	3,50	185519,64	53005,61	0,73	0,54	28557,09
13,00	3,50	185519,64	53005,61	0,79	0,62	32663,38
14,00	3,50	185519,64	53005,61	0,84	0,70	36956,84
15,00	3,50	185519,64	53005,61	0,88	0,78	41327,89
16,00	3,50	211493,72	60426,78	0,93	0,86	51926,48
17,00	3,50	206794,86	59084,25	0,97	0,93	55134,82
18,00	3,50	149321,94	42663,41	1,00	1,00	42663,41
				Sum:	*7,45*	*396300,28*
					me =	**53208,58**

Equivalent Mass and Dimensionless Coefficient in y-direction

Dimensionless Coefficient

n	z	$\Phi_{1.y}(z)$	$v_m(z)$	$v_m^2 \cdot \Phi_{1.y}(z)$	$\Phi_{1.y}(z)^2$	$v_m(z_s)^2$
1,00	5,00	-0,06	14,64	-11,79	0,00	377,00
2,00	10,00	-0,13	14,64	-27,00	0,02	
3,00	13,50	-0,18	14,64	-38,79	0,03	
4,00	17,00	-0,23	14,96	-52,14	0,05	
5,00	20,50	-0,29	15,95	-73,25	0,08	
6,00	24,00	-0,35	16,78	-97,42	0,12	
7,00	27,50	-0,41	17,50	-124,63	0,17	
8,00	31,00	-0,47	18,13	-153,52	0,22	
9,00	34,50	-0,53	18,70	-184,56	0,28	
10,00	38,00	-0,59	19,21	-216,53	0,34	
11,00	41,50	-0,65	19,67	-250,77	0,42	
12,00	45,00	-0,70	20,10	-283,99	0,49	
13,00	48,50	-0,76	20,50	-319,23	0,58	
14,00	52,00	-0,82	20,86	-354,74	0,66	
15,00	55,50	-0,87	21,21	-390,37	0,75	
16,00	59,00	-0,91	21,53	-423,64	0,84	
17,00	62,50	-0,96	21,83	-456,22	0,92	
18,00	66,00	-0,99	22,12	-484,45	0,98	
			Sum:	-3943,03	6,96	
				Ky =	**-1,50**	

Equivalent mass

n	[m]	[kN]	[kN/m]	$\Phi_{1.y}(z)$	$\Phi_{1.y}(z)^2$	$m(z) \cdot \Phi_{1.y}(z)^2$
1,00	5,00	352359,59	70471,92	-0,06	0,00	213,18
2,00	5,00	201996,31	40399,26	-0,13	0,02	641,38
3,00	3,50	185519,64	53005,61	-0,18	0,03	1736,52
4,00	3,50	185519,64	53005,61	-0,23	0,05	2877,62
5,00	3,50	185519,64	53005,61	-0,29	0,08	4396,50
6,00	3,50	185519,64	53005,61	-0,35	0,12	6345,62
7,00	3,50	185519,64	53005,61	-0,41	0,17	8780,33
8,00	3,50	185519,64	53005,61	-0,47	0,22	11559,94
9,00	3,50	185519,64	53005,61	-0,53	0,28	14777,12
10,00	3,50	185519,64	53005,61	-0,59	0,34	18264,09
11,00	3,50	185519,64	53005,61	-0,65	0,42	22257,27
12,00	3,50	185519,64	53005,61	-0,70	0,49	26195,85
13,00	3,50	185519,64	53005,61	-0,76	0,58	30616,04
14,00	3,50	185519,64	53005,61	-0,82	0,66	35207,65
15,00	3,50	185519,64	53005,61	-0,87	0,75	39935,70
16,00	3,50	211493,72	60426,78	-0,91	0,84	50480,29
17,00	3,50	206794,86	59084,25	-0,96	0,92	54112,25
18,00	3,50	149321,94	42663,41	-0,99	0,98	41814,41
				Sum:	*6,96*	*370211,74*
					me =	**53218,80**

Appendix D

Note – *Økern Sentrum*

SWECO

NOTAT – ØKERN SENTRUM

KUNDE / PROSJEKT	PROSJEKTLEDER	DATO
Steen & Strøm	Daniel Adolfsson	01.07.2020
PROSJEKTNUMMER	OPPRETTET AV	REV. DATO
10214111	Jonas Johnstad og Cathrine Hafnor Revidert av Daniel Adolfsson	20.11.2020

Bakgrunn

I forbindelse med utvikling av område på Økern, er det bedt om et estimat av muligheter ang. antall etasjer som kan bygges over flere tunneler og kulverter på Økern. Det vises til et Premissnotat fra «Aas-Jakobsen» for Statens vegvesen (DOK.nr B014), som legger føringer for mulig ovenforliggende bebyggelse over Økerntunnelen, Lørentunnelen, rampe Grorud – Sinsen, samt en flere tekniske- og VA kulverter. Dette notatet tar for seg et grovt estimat over mulig antall etasjer over de ulike tunneler mht. fundamentering direkte på tunnelenes vegger og/eller tak med de kapasiteter gitt i Aas-Jakobsen sitt notat.

Forutsetninger

For beregningene er det gjort følgende forutsetninger:

- For konstruksjoner med bærende betongvegger (ikke søyler) settes det at veggkonstruksjonene opptar 6 % og massivtreveggene 9 % av BYA. Det er tatt utgangspunkt i et bygg med dimensjoner lik 20 m x 30 m for beregning av andel bærende vegger.
- Det er lagt inn restriksjonssoner som gjelder området over tunneler og arealer inntil ca.10 m utenfor tunneler. For å få et fornuftig areal i kjeller, må denne hensynssonen revurderes. Sweco har ansvar for plan-prosjektering av bebyggelse i sone 1, 2, 3 og 4. I disse sonene skal all byggeaktivitet godkjennes av Statens vegvesen før den kan igangsettes. Videre er krav, kapasiteter og andre forutsetninger hentet fra notat av «Aas-Jakobsen» for Statens vegvesen. (DOK.nr B014).
- I dette notat er det tatt utgangspunkt i at byggets last fordeles ned på tuneller i form av flatelast. Der det er behov og mulig, kan det ved hjelp av bjelker føres laster ned på utsiden av tunell/kulvert som kan gi en høyere kapasitet. Dette tas i utgangspunktet ikke med i dette notatet.

Laster

For beregningene er det brukt følgende laster:
Nyttelast: 5 kN/m2 for næring, 3 kN/m2 for kontor, og 2kN/m2 for bolig. Det antas 2 etasjer med næring, og resterende i bolig eller kontor.
Egenlaster: Hulldekker 4 kN/m2, bærevegg betong 25 kN/m3, lettvegger 0,5 kN/m2, teknisk + himling 1 kN/m2.
For tak det er beregnet 3 alternativer:

- Takkonstruksjon med ordinær taktekking (1,5kN/m2)
- Takkonstruksjon med 0,7m vannmettet «lettjord» fra Bergknapp (10kN/m2)

- Takkonstruksjon med 0,7m vannmettet ordinær jord (16,5kN/m2)

Snølast: 2,8 kN/m2 (dimensjonerende). Det er i tillegg medregnet en teknisk etasje (på tak) med 5kN/m2 nyttelast.

Materialer og bæresystemer

Det er gjort et overslag over antall mulige etasjer over de ulike tunnelene/kulvertene. Det er tatt utgangspunkt i 2 etasjer med næringslokaler og resterende etasjer som bolig eller kontorer (det er gjort beregninger for begge tilfeller). Disse 2 mulighetene er igjen kontrollert med ulike byggematerialer:

- **Betong**: komplett bæresystem i betongelementer, hulldekker og innvendige lettvegger.
- **Betong/stål**: betongvegger i trappehus/heissjakt, stålsøyler med lette vegger som yttervegger, samt hulldekker.
- **Massivtre**: massivtre veggelementer i yttervegger og bærevegger, massivtredekker, 4-5 betongdekker. For massivtre er det antatt flere bærevegger grunnet begrensning i spenn på massivtredekker (maks 7,5m).

Område 1

- Fra Aas-Jakobsens notat fremkommer det at lastene som påføres tunnelens fundamenter fra overliggende bebyggelse må påføres som uavhengige stripelaster over tunnelens vegger. Typisk verdi av stripelastene er 1000-1370 kN/m, avhengig av tunnelbredden. Laster fra fremtidig bebyggelse kan kun settes på oppstikkende betongvegger over tunneltaket. I våre beregninger brukes det da 11m lastbredde, og en maksimal linjelast på 1370kN/m.
- Teknisk kulvert er fundamentert i løsmasser, og er ikke dimensjonert for overliggende bygg. Kulverten er også sensitiv for setninger og det må vises varsomhet ved fundamentering rundt. Dvs. at ovenforeliggende bebyggelse må fundamenteres ned på hver side av kulvert, uten belastning på kulverts tak.
- Det kan være mulig å fundamentere på fjell ved siden av kulvert. Lastbredden over kulvert til tunnelfundament ser ut til å være omtrent like stor som tunnelveggene. Det tas derfor utgangspunkt i samme maksimale karakteristisk nyttelast som i område direkte over tunnel; 1370 kN/m.

Felt F9

Felt F9 er plassert over setningssensitiv kulvert, og det må vises varsomhet ved fundamentering rundt denne. Ellers kan ingen laster føres ned på kulvert. Her vil evt. utveksling av konstruksjon til siden av tunnel og kulvert være nødvendig, for å føre laster til siden for kulvert og ned gjennom fundamenter direkte til berg. Krav fra Statens Vegvesen for mulig lastnedføring rundt dette feltet vil være førende for hva som kan bygges. I tillegg er tilgjengelig plass mellom kulvert og andre konstruksjoner samt ovenforliggende bebyggelse av vesentlig betydning med tanke på tilstrekkelig høyde for bæresystem. Dette bør dog være løsbart slik at planlagt høyde på felt kan bygges. Adkomster til kulverter må ivaretas ved overbygging.

Område 2 - Økerntunnelen:

Fra notat av Statens vegvesen, er det ved dimensjonering av Økerntunnelen gjort antagelser om plasstøpte bygg med maksimalt 8 etg. Der er det beregnet egenvekt på 10kPa/etg og en nyttelast på 5kPa/etg. Ved å bygge i annet materiale og/eller bruke annen nyttelast, kan denne begrensningen økes. Se videre resultater.

Forutsetninger for beregninger:

- Det tas utgangspunkt i karakteristisk linjelast på 1370 kN/m når maksimalt antall etasjer beregnes.
- Det benyttes en lastbredde på 11 m, hvilket er verst tenkelige tilfelle. I enkelte tilfeller er det kanskje mulig å nedjustere denne da bygget ikke står fullstendig over tunnel. Her vil det også bli ekstra utfordringer med å overføre lasten ned på tunnelens fundamenter.

Felt F2 og F3

Hjørne på F3 er så vidt plassert over setningssensitiv kulvert, og det må vises varsomhet ved fundamentering rundt denne. Ellers kan ingen laster føres ned på kulvert. Her vil evt. utvekslingskonstruksjon være nødvendig, for å føre laster til siden for kulvert og ned gjennom fundamenter direkte til berg. Krav fra Statens Vegvesen rundt mulig lastnedføring rundt denne vil være førende for hva som kan bygges. I tillegg er tilgjengelig høyde mellom kulvert og ovenforliggende bebyggelse vesentlig mtp. plass til bæresystem. Dette bør dog være løsbart slik at planlagt høyde kan bygges.

Hjørnet av F3 og F2 mot rundkjøring er til dels plassert over teknisk bygg, der både ventilasjon og adkomst må ivaretas. Hjørne F3 er i tillegg plassert over adkomst til kulvert som også må ivaretas.

For øvrige bygg i dette området er det den maksimale karakteristiske stripelasten som begrenser høyden på bygget.

Område 3, 4 og 5 - Lørentunnelen, rampe Grorud – Sinsen

Lørentunnelen er ikke dimensjonert for overliggende bebyggelse. Det ser også ut til at det er lite sannsynlig at Statens vegvesen tillater overliggende bebyggelse pga. hindret adkomst fra oversiden ved en hendelse i tunnelen som krever vedlikehold/utbedring.

Grorud – Sinsen rampe er ifølge Aas-Jakobsens rapport dimensjonert for en karakteristisk terrenglast på 40kPa, og er generelt sensitiv for skjevlast pga. det sirkulære tverrsnittet. På bakgrunn av dette, bør trolig last fra overliggende bebyggelse føres ut på hver side av tunnelveggene, evt. bygge lave bygg i lette materialer. Ved lavere utgravninger på en side enn den andre, må det etableres spuntvegg langs tunnelen. Ifølge rapporten er det er fra Statens Vegvesens side ikke stilt spesifikke krav til tilkomst for vedlikehold av denne tunnelen. Dersom det mot formodning skulle skje en hendelse der det blir behov for tilkomst for tyngre vedlikehold vil dette kunne medføre at overliggende bebyggelse må rives. Sannsynligheten for en slik hendelse er svært liten.

Felt F1

Felt F1 er plassert over påkjøringsrampe til Lørentunnelen, og det bør derfor kontrolleres mulighet for bygg her med Statens Vegvesen i tidlig fase.

Utenfor definerte soner

Felt F7 og F8

F8 vil bygges over innkjøring til tunnel samt ved siden av og vil måtte godkjennes av Statens Vegvesen for å kunne bygges. Det gjelder også forsiktighet med fundamentering rundt VA-kulvert. Det vil også her være bæresystem over kulvert som blir dimensjonerende for antall mulige etasjer, og tilgjengelig høyde mellom øvre del av kulvert og ovenforeliggende bygg er essensielt. Dette bør dog være løsbart slik at planlagt høyde kan bygges.

F7 vil bli inneklemt mellom Østre Aker Vei og Økernveien. Feltet ser ut til å ligge over en undergang/trapper ned til t-bane. Dette er ikke nevnt i notat fra Statens Vegvesen, men det antas at adkomst her også må ivaretas, og at bygget må fundamenteres utenfor dette.

Felt F4 og F10

Felt F4 og F10 antas i mindre grad å være begrenset av konstruksjoner i grunnen.

Konklusjon

For å komme i nærheten av de bygningshøyedene som er planlagt over Økern-området anbefales det benyttes en kombinasjon av massivtre og betongdekker. I sone 1 og 2 kan det da være mulig å oppnå omtrent 21 etasjer uten grønt tak, 17 etasjer med grønt tak og 19 etasjer dersom det benyttes «lettjord». For å oppnå disse høydene forutsettes det som tidligere nevnt at kun de to nederste etasjene er næring og at resterende etasjer benyttes til boligformål. Dersom etasjene over næringslokalene skal benyttes som kontor vil maksimalt antall etasjer være omtrent 18 uten grønne tak og 15 med grønne tak med lettjord. Byggene i sone 3 og 5 er planlagt over Lørentunnelen og/eller rampen ned til Grorud-Sinsen tunnelen. I disse sonene tåler fundamentet svært lave laster og med massivtre vil det maksimalt kunne bygges 3 etasjer. Ifølge notatet av «Aas-Jakobsen» for Statens vegvesen vil det trolig ikke være mulig å bygge noe som helst over Lørentunnelen.

Det må allikevel presiseres at dette kun er et estimat gjort ut ifra gitte forutsetninger. Dersom man optimaliserer byggene og har tilstrekkelig med tid og økonomiske midler er det sannsynligvis mulig å bygge høyere enn disse estimatene.