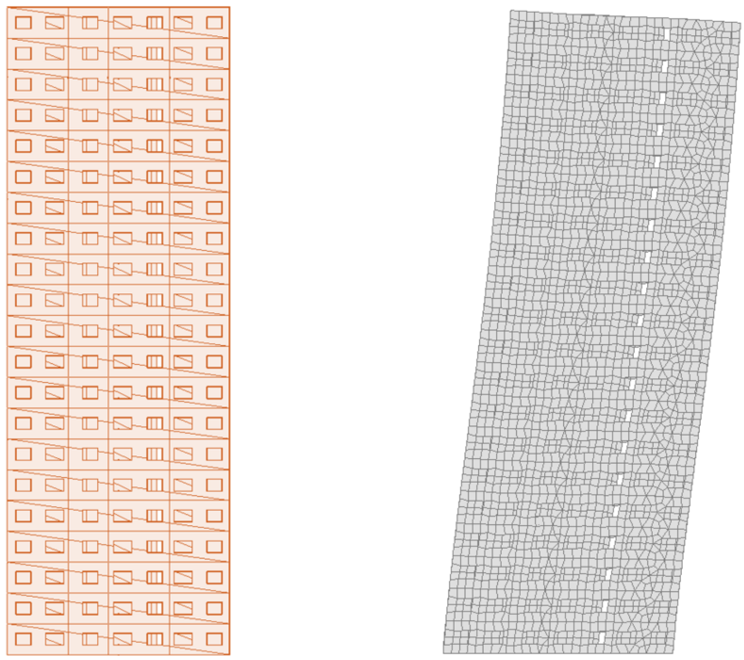

# Dynamic Optimization of High Rise Timber-Hybrid Buildings

## A FEM study of the effect on the dynamic response of combining timber and concrete in a high rise building

Master's thesis in Structural Engineering and Building Technology


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CHALMERS UNIVERSITY OF TECHNOLOGY



Gothenburg, Sweden 2023
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Master's thesis ACEX30

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Chalmers tekniska högskola, 2023

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Cover: FEM-Design model of a timber building and the building's first eigenmode.

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# Abstract

In recent years, the popularity of building high rise timber buildings has increased. Some reasons for using timber in high rise structural systems are for environmental, architectural and marketing aspects. Timber is a light-weight building material which can result in large dynamic effects due to wind, causing comfort problems in the buildings. One solution is to build timber-hybrid structures that utilize other structural materials in combination with timber. A study is done with focus on dynamic responses in high rise timber and timber-hybrid buildings to optimize the combination of timber and concrete.

The aim of the project is to get a better understanding of the effect on added stiffness and mass by utilizing concrete as material in high rise timber-hybrid buildings and how it influences the dynamic response. The work is carried out by using FE analyses to determine the dynamic properties of different timber-hybrid buildings. The FEM study is performed using the software FEM-Design. Using results from the FEM study, calculations methods from EKS 12 and Eurocode 1-4 are used to determine the accelerations in buildings caused by wind loads. The results are compared to the human comfort requirements of ISO 10137 and ISO 6897.

The results show that replacing timber structural elements with concrete is an effective way of improving dynamic properties in high rise timber buildings. It is shown that concrete floors are more efficient at the top of the building, where the mass has the largest influence. It is also shown that concrete cores are effective throughout the full height of the building. The most optimal solutions are found when a concrete core and concrete floors were utilized in the structural systems. The CLT structural system can be built with less concrete than the Glulam column system, however the CLT building requires more timber. A building with both systems combined can be used to increase the building height and reduce concrete volume compared to just using one system. A tuned mass damper is shown to be a potential solution to dynamic problems in high rise timber hybrid buildings.

Keywords: Structural dynamics, Timber, Timber-hybrid, Tall timber building, Concrete volume, Wind, Damping, TMD, Modal mass, Mode shape, Acceleration, CLT, Glulam.

Dynamisk optimering av höga trä-hybridbyggnader
En FEM-studie av effekten på den dynamiska responsen med kombinerat trä och betong i ett höghus
*Examensarbete inom konstruktionsteknik och byggnadsteknologi*

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# Sammanfattning

Under de senaste åren har populariteten av höghus i trä ökat. Några skäl till att använda trä i höghuskonstruktioner är på grund av miljö, arkitekt och markadsföringsaspekter. Trä är ett lätt byggnadsmaterial, vilket kan leda till stora dynamiska effekter på grund av vind. Detta kan orsaka komfortproblem i byggnaderna. En lösning är att bygga trä-hybridstrukturer som använder andra byggmaterial i kombination med trä. En studie har gjorts med fokus på dynamisk respons i höga träbyggnader och trä-hybridbyggnader för att optimera kombinationen av trä och betong.

Målet med projektet är att få en bättre förståelse för hur ökad styvhet och massa påverkar den dynamiska responsen genom att använda betong som material i höga trä-hybridbyggnader. Arbetet utförs genom att använda FE-analyser för att bestämma de dynamiska egenskaperna hos olika trä-hybridbyggnader. FEM-studien utförs i programvaran FEM-Design. Med hjälp av resultat från FEM-studien används beräkningsmetoder från EKS 12 och Eurocode 1-4 för att bestämma accelerationerna i byggnader orsakade av vindbelastningar. Resultaten jämförs med kraven för mänsklig komfort enligt ISO 10137 och ISO 6897.

Resultaten visar att ersätta träbyggnadsdelar med betong är en effektiv metod för att förbättra de dynamiska egenskaperna i höga trähus. Betongbjälklag är mer effektiva högst upp i byggnaden, där massan har störst påverkan. Betongkärnor är effektiva längs med hela byggnadens höjd. De mest optimala lösningarna använder en kombination av betongkärna och betonggolv i de strukturella systemen. KL-träsystemet kan byggas med mindre betong än Limträ-pelarsystemet, men KL-träsystemet kräver mer trä. Genom att kombinera båda systemen i en byggnad kan byggnadens höjd ökas och betongvolymen minskas jämfört med att bara använda ett system. En TMD kan vara en potentiell lösning på dynamiska problem i höga trä-hybridhus.
Nyckelord: Strukturdynamik, Trä, Trä-hybrid, Höga träbyggnader, Betong volym, Vind, Dämpning, TMD, Modmassa, Ekvivalent massa, Modform, Acceleration, KL-trä, Limträ.

# Preface

This study is the final part of the master program Structural engineering and building technology at Chalmers university of technology. The dynamic behavior of timber and timber-hybrid buildings due to wind loads have been investigated in order to optimize their dynamic responses and reduce the use of concrete. FEM Analyses have been done using FEM-Design. The project has taken place at COWI's Gothenburg office. COWI have contributed with a computer and software for the project. They have also contributed with a desk at the office and other materials to make the project possible. We would like to thank our supervisor from COWI, Thomas Hallgren, who has been involved and helpful. We especially would like to thank you for our many interesting discussions during the project. We also want to thank our examiner Robert Jockwer for continously holding meetings and supporting us with new ideas throughout the project.

Johan Dahlén and Jack Niemi-Impola, Gothenburg, May 2023

# Notations

## Roman uppercase letters

| | |
|---|---|
| $A_{ref}$ | Reference area of the structure |
| $B^2$ | Background factor |
| $E$ | Stiffness modulus |
| $F$ | Von Kármáns wind energy spectrum |
| $G$ | Shear modulus |
| $K_{ser,i}$ | Slip modulus of joint |
| $H$ | Horizontal resultant of wind load |
| $\mathbf{I}$ | Unit matrix |
| $I_v(z)$ | Turbulence intensity factor |
| $\mathbf{K}$ | Stiffness matrix |
| $\mathbf{M}$ | Mass matrix |
| $R^2$ | Resonance response |
| $S_i$ | Shear stiffness of bracing |
| $S_{joint}$ | Joint stiffness |
| $T$ | Torsional moment |
| $T_k$ | Time period |
| $\mathbf{V}$ | Damping matrix |
| $\ddot{\mathrm{X}}_{max}$ | Peak acceleration |

## Roman lowercase letters

| | |
|---|---|
| $a_i$ | Position of bracing member center in x-direction |
| $b_i$ | Position of bracing member center in y-direction |
| $c_0$ | Topography factor |
| $c_O$ | Orography factor |
| $c_f$ | Force coefficient |
| $c_{pe}$ | Pressure coefficient for external pressure |
| $c_sc_d$ | Structural factor |
| $e$ | Eccentricity |
| $f(t)$ | External force on the system at time $t$ |
| $f_k$ | k:th eigenfrequency [Hz] |
| $h$ | Height |
| $h_{ref}$ | Reference height |
| $k$ | Stiffness |
| $k_p$ | Peak factor |
| $k_r$ | Terrain factor |
| $k_s$ | Horizontal stiffness of the building |
| $k_{TMD}$ | Stiffness of spring connecting the building to the TMD |
| $m$ | Mass |

| | |
|---|---|
| $\overline{m}$ | Mass ratio |
| $m_e$ | Equivalent mass |
| $m_s$ | Modal mass of building |
| $m_{TMD}$ | Mass of the TMD |
| $r_i$ | Radial distance from rotational centre |
| $t$ | Thickness |
| u | Displacement |
| $\dot{\text{u}}$ | Velocity |
| $\ddot{\text{u}}$ | Acceleration |
| $v$ | Viscous damping coefficient |
| $q$ | Line load |
| $v_b$ | Basic wind velocity |
| $v_{\text{m}}$ | Mean wind velocity |
| $v_s$ | Structural damping in the original building |
| $v_{TMD}$ | Viscous damper connecting the building to the TMD |
| $\mathbf{x}$ | Eigenvector |
| $x_t$ | X-coordinate of rotational center |
| $y_C$ | Non-dimensional frequency |
| $y_t$ | Y-coordinate of rotational centre |
| $z_0$ | Roughness length (Table 4.1 EC1) |
| $z_{0,II}$ | Roughness length terrain category II |
| $z_e$ | Reference height |
| $z_{max}$ | Maximum reference height |
| $z_{min}$ | Minimum reference height |
| $z_s$ | Reference height for structural factor |

## Greek letters

| | |
|---|---|
| $\delta_a$ | Aerodynamic damping |
| $\kappa_s$ | Shear factor in timber walls |
| $\kappa_b$ | Bending factor in timber walls |
| $\lambda$ | Eigenvalue |
| $\mu$ | Modal mass |
| $\nu$ | Up-crossing frequency |
| $\xi$ | Damping ratio |
| $\rho$ | Density |
| $\sigma_{\ddot{X}}$ | r.m.s acceleration |
| $\phi$ | Eigenvector |
| $\Phi$ | Modal matrix |
| $\phi_h$ | Size factor in respct to height |
| $\phi_b$ | Size factor in respect to witdh |
| $\phi_1(z)$ | Mode shape |
| $\omega$ | Frequency [rad/s] |

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# 1 Introduction

## 1.1 Background

There is an increase in popularity of high rise buildings with timber as main structural material. These buildings can be categorised into timber and timber-hybrid buildings. Where timber-hybrid buildings also include other materials in the strucutal system. According to SCB (Landel, 2022), the percentage of newly produced multi storey timber buildings have increased from 5% to 20% among multi storey residential buildings from 2010 to 2020. An argument for using timber as building material instead of more conventional materials such as steel and concrete is the environmental aspect. Timber has a lower environmental impact than materials such as steel or concrete that are more frequently used when building high-rise buildings. To produce steel and concrete, a higher amount of energy is used than for the production of timber products. The production process of these also releases higher amounts of emissions. Since production of building materials stands for approximately 11% of the total emissions in the building sector(IEA, 2019), replacement of concrete and steel with timber has potential to lower the environmental impact.

Despite the recent increase in popularity, it is still uncommon that high rise buildings are built with timber. One reason for this is that the dynamic response from wind load on tall timber buildings usually becomes an issue when considering comfort demands of the building. This is because compared to steel and concrete, timber is more lightweight and has a lower stiffness, which makes timber buildings more affected by external loads such as wind and also more sensitive to vibrations. Solutions to this problem, that are being used in existing timber-hybrid buildings, is to replace timber floors with reinforced concrete slabs in order to increase the mass of the structure, since more mass leads to increased inertia in the building against wind. Another solution is increasing the stiffness in the horizontal bracing of the building is another way to improve the dynamic response. One solution used in some high-rise buildings is to use tuned mass dampers.

If the design of high rise timber-hybrid structures can be optimized with regards to dynamic effects, then timber can become more competitive building material and solution for high rise buildings when comparing to reinforced concrete, which has the potential of leading to an increase in production of timber-hybrid buildings.

## 1.2 Aim and objectives

The aim of the project is to get a better understanding of the effect of added stiffness and mass in high-rise timber-hybrid buildings from replacing timber elements with reinforced concrete. This understanding should guide the optimization of the structure to better withstand the dynamic effects of wind loads.

The objectives are to calculate the dynamic properties of high rise timber-hybrid buildings by using FEM-analysis and hand calculations based on EKS12 and Eurocode 1-4. Results of the analysis should be assessed to find alternatives that are suitable as structural solutions that reduce the use of concrete.

## 1.3 Method

The project begins with a literature study. In the literature study, already existing tall timber-hybrid buildings are studied. Furthermore, Swedish engineering guidelines, ISO-standards, building materials theory and structural dynamics theory are studied to support calculation choices during the project.

When the literature study is done and the necessary information is collected, modelling of the structure the numerical FEM analysis is done. Modelling choices are made based on the literature study and discussions with the supervisor and examiner.

Using the model, the dynamic FEM analysis of the structure is done in many iterations while using different parameters, the most substantial being changes to mass and stiffness. The mass and location of mass is changed by replacing timber floors with reinforced concrete floors. The stiffness is changed by modifying stabilizing elements in timber and by replacing them with reinforced concrete, resulting in an increase in the stiffness of the building. This is done using the software FEM-Design. The FEM model is then verified using analytical calculations based on structural dynamics theory. Accelerations due to wind loads are calculated analytically using methods described in EKS 12 and Eurocode 1-4.

After analyses are done, concrete volume is compared between different solutions. Concrete and timber volumes are compared to estimated values from existing high rise timber-hybrid buildings. A TMD model is developed for post-processing of results and tested on different buildings in order to understand the feasibility of using dampers in high rise timber-hybrid structures. Recommendations are derived for the conceptual design from the results of the studies.

## 1.4 Limitations

This thesis only covers the dynamic response in the building caused by wind loads. Other loads on the building which may produce dynamic effects are not accounted for in the analyses. The studied structural response studied is limited to the dynamic response in the building. The building that is used as a model will keep the same footprint throughout the project and changes in width or length will not be done to optimize dynamic response in the structure. Different types of building foundation conditions are not compared in the project, but instead the foundation is modelled as line or point supports. Longterm effects such as creep and shrinkage will not be considered in this thesis. The location of the the building is Gothenburg and environmental factors and aspects will be used for that location.

## 1.5 Ethical, social and Ecological aspects

A main objective on this project is the dynamic response optimization of hybrid timber structures to find the most suitable solution for the structural system. By working towards material optimisation between timber and reinforced concrete a more sustainable suggestions can be made in order to reduce the environmental impact in the construction phase of the building.

Improvements to efficiency of new production of buildings supports UN's sustainability goal 9:

*"Build resilient infrastructure, promote inclusive and sustainable industrialization and foster innovation"*

Work towards a higher percentage of new production in timber supports UN's sustainability goal 11:

*"Make cities and human settlements inclusive, safe, resilient and sustainable"*
(United Nations, n.d.)

# 2 Theory

## 2.1 Timber

Timber has been used as construction material for thousands of years. In Sweden, timber is used with a long tradition due to its local availability and the experience of craftsmen. Timber is used in buildings as structural material in the load-bearing system and as non-load bearing material such as facade cladding to protect against the surrounding environment.

### 2.1.1 Properties of timber

Wood is a naturally grown material which has adapted to manage its own demands. The properties of the material is therefore optimized to resist the loads that acts on the standing tree. The strength properties of a tree is dependent on its growth characteristic. The tree is adapted to the local conditions of where it is growing, which will give different material properties depending on where it is grown. When wood is processed in order to be used in construction, then it is referred to as timber.

Timber is an anisotropic material which means that the properties of the timber is different in different directions relative to the wood fibres. The strength of timber is much higher parallel to the fibres than perpendicular to the fibres, according to Thelandersson and Larsen (2003), the difference is between 30-50 times smaller perpendicular to the grain. The difference in strength makes it important to consider the directions of loading on the timber member in the design (Thelandersson & Larsen, 2003).

Timber is also a hygroscopic material that can exchange its moisture content with the ambient environment. The mechanical properties of the wood is highly dependent on the moisture content of the wood. Below the saturation point of the timber the strength and stiffness properties of the timber increases (Thelandersson & Larsen, 2003). The change of the moisture content in timber also leads to shrinkage or swelling of the timber (Svenskt Trä, 2015), this will induce stresses in the constraints of timber members. Parallel to the grain where the stiffness is high, these stresses are negligible but perpendicular to the grain where the stiffness is low the stresses can be considerably higher and must be taken into consideration.

### 2.1.2 Timber products

Traditionally, sawn timber is the most commonly used timber product in the building industry. Since the beginning of the nineteen-hundreds, many new types of timber engineered wood products (EWP) have been developed (Svenskt Trä, 2015). The benefit of EWPs are that they can be manufactured to overcome the limited sizes of sawn timber. EWPs can also be manufactured with higher quality and different properties to sawn timber products. In this section timber products suitable as structural elements for high rise timber buildings will be discussed, with glulam, CLT and LVL being considered engineered wood products.

#### 2.1.2.1 Sawn timber

Sawn timber is a solid member that is sawn out of a timber log. The sizes of the sawn timber has natural size limit from the log, but also limitations in size from the industrial processes in the sawing of timber. For Swedish industries, the maximum sizes for sawn timber is generally 245 mm high with a maximum length of 5.5 m (Svenskt Trä, 2015). After the timber is swan its gets classified by its strength quality(Al-Emrani et al., 2013). By either machine grading or visual grading, a strength class of structural timber is determined. The classification of the sawn timber is highly dependent on defects and the growth characteristics of the tree such as distribution of knots and straightness of fibres. The classes of sawn timber varies between C14 and C40, where the number represents the characteristic bending strength in the material in MPa. In Sweden, the most common classes are C14 and C24.

#### 2.1.2.2 Glue-Laminated timber (Glulam)

Glulam is produced by gluing together multiple lamellas of sawn timber into a rectangular cross section, where the lamellas are normally up to 45 mm in thickness. The lamellas are oriented with fibres in the same axial direction as the final glulam member (Al-Emrani et al., 2013). An important benefit of glulam compared to sawn timber is that it can overcome the limited manufacturing size, making glulam a suitable option for columns and beams that necessitate larger dimensions. The height of a glulam element is limited by industrial machinery to 2 meters at a constant cross section. Multiple lamellas can be glued together along the element's width, leading to a maximum width of 800 mm. The length of a glulam element is limited to 40 m, however it is ideally produced up to an length of 24-30 m which is the longest length of elements that can easily be transported by road traffic (Svenskt Trä, 2016).

To optimize glulam beams, higher quality timber is used in the outer lamellas where bending stresses are higher, and a lower quality timber in the middle lamellas. Strength classes of glulam exist from GL22c to GL32c, where the number represents the characteristic bending strength in the material similar to how sawn timber classes are named. The last letter in the name refers to the type of element c = combined, h = homogenous, s = split. In Sweden, GL30c is the most common class used (Svenskt Trä, 2015). By gluing together lamellas of sawn timber the variation of defects will be spread out in the elements rather than being locally concentrated. Therefore glulam elements provide the benefits of lower variability in strength and higher Young's modulus than for the sawn timber (Svenskt Trä, 2015). Glued-laminated beams can also be produced in curved shapes which is done by putting the lamellas into form work before being pressed and glued together (Thelandersson & Larsen, 2003).

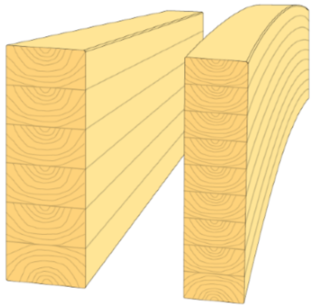


**Figure 2.1:** Example of two glulam beams, one straight and one curved. Illustration: Svenskt Trä

#### 2.1.2.3 Cross laminated timber (CLT)

CLT is a relatively new product that been more commonly used in recent years (Svenskt Trä, 2019). CLT are structural timber panel products that are utilized as wall, floor and roof elements. As floor elements, CLT panels alone can span up to 7.4 m, but can be combined with glulam beams to span up to 16 m (Martinsons, 2022). The CLT is manufactured by gluing together several layers of sawn timber, with strength classes commonly between C14-C30. The layers of sawn timber are often glued together perpendicularly in up to 9 layers. This is done due to the anisoptropic properties of the wood. The high strength of wood parallel to the fibres is utilized in both directions, thus a CLT panel efficiently transfers loads in both in-plane directions. If one direction is intended to be the load carrying direction, lower strength class timber can be used perpendicularly to the load, thus saving the high quality material (Svenskt Trä, 2019). The number of layers is often odd so that the outer layers are oriented in the same direction (Svenskt Trä, 2015) which become the main load-bearing directions. In some applications, there are two parallel outer layers on each side of the board, making the panel considerably stronger in that direction. The combination of several sawn layers also has the benefit of spreading out local defects in the material, providing the same improvements as glulam compared to sawn timber. The size of the CLT-panels depend on the capacity of the manufacturer but it can have lengths up to 24 m, a width up to 3 m and a thickness up to 500 mm. The possibility of large cross sections makes CLT useful for load bearing and stabilizing elements in buildings and can compete with traditional building materials such as steel and concrete in high-rise buildings (Svenskt Trä, 2019).

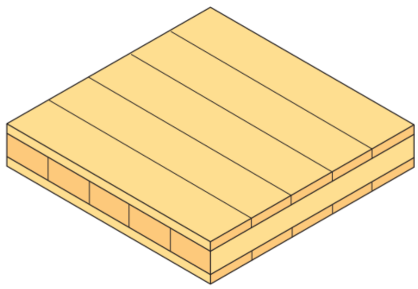


**Figure 2.2:** Example of a 3 layer CLT panel. Illustration: Svenskt Trä

#### 2.1.2.4 Laminated veneer lumber (LVL)

The veneers are produced in a process where raw lumber is heated with steam and then turned into layers with a thickness of 2-4 mm. The veneers are then glued together, most often with the grain aligned in the same orientation. The final thickness of the LVL is often 20-90 mm. By gluing different veneers together the defects will be spread out similarly to in the glulam beams, which leads to a lower variability of the product. LVL is used as material in beams and panels (Svenskt Trä, 2015).

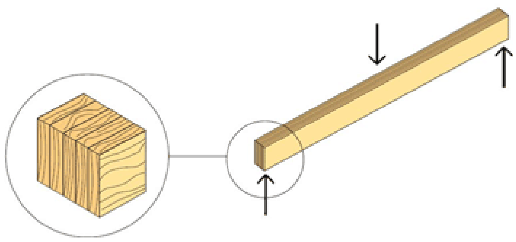


**Figure 2.3:** Example of an LVL beam. Illustration: Svenskt Trä

## 2.2 Concrete

Concrete is the most commonly used building material in the world. Concrete is a mix of cement, water and aggregates, such as gravel and sand. Main characteristics of concrete is the relatively high stiffness of the material and that the strength in compression is considerably higher than in tension. Material properties of concrete can be adjusted using additives and admixtures. The composition of material in the concrete highly influences the strength and stiffness of concrete. A higher amount of cement in the mix generally creates stronger concrete.

Additives are commonly used as cement replacements in the concrete due to the high carbon dioxide emissions in the calcination process of cement. The additives are divided into two different types. Type 1, nearly inert additions and type 2, pozzolanic or latent hydrated addition. The most commonly used type 1 addition is limestone. By adding limestone, physical improvements of the fresh properties can be achieved e.g. better cohesiveness and consistence. Type 2 additions increase the formation of calcium silicate hydrate (C-S-H) which improves the long term strength of concrete (Domone & Soutsos, 2017). Examples of type 2 additions are fly ash, slag and micro silica.

Admixtures such as superplasticizers, accelerators and retarders have an influence on the fresh properties of the concrete. The effect on fresh concrete is changes in the workability and curing time of fresh concrete. Other admixture such as Air-entrancing and water-resistant admixtures improves the long-term durability of the concrete (Domone & Soutsos, 2017).

Concrete members exposed to tensile stresses are often reinforced due the relatively low tensile strength of the concrete. The reinforced concrete utilizes the composite interaction between the reinforcement steel and concrete which allows for a higher capacity in tension than for the pure concrete.

## 2.3 Environmental impact of different building materials

The building material used in buildings has a considerable impact on the global emissions which affect the climate. The building sector is responsible for 38% of global $CO_2$ emissions (Tupenaite et al., 2021). In the building sector, 40-50% of a buildings energy consumption during its lifetime comes from the production and transport of the building materials and construction of the building (Skullestad et al., 2016). Therefore, choosing an energy efficient building material is an approach to reduce $CO_2$ emissions.

Building with timber is associated with a less negative impact on the environment than building with other materials such as concrete and steel. Timber is the only widely used renewable construction material. It is reusable and recyclable and emits less $CO_2$ than steel and concrete in the production process (Tupenaite et al., 2021). A life cycle assessment (LCA) study by Skullestad et al. (2016) showed that a timber building can produce a 34-84% lower climate change impact than an equivalent reinforced concrete building. The study also showed that the reduction in climate change impact per $m^2$ floor area decreases slightly with building height up to 12 storeys, but increase from 12 to 21 storeys.

## 2.4 Horizontal stabilisation

Wind that is acting on a building results in a horizontal pressure on the building's facade which is distributed to the structural system. When a structure is loaded horizontally the resulting effects on the building is a possibility of horizontal deflections, warping and twisting deflections. A common way to move wind loads from the facade to the horizontal bracing system is through the floor and roof structures (Svenskt Trä, 2015). The floors that distribute forces to the bracing members are called diaphragms. Diaphragms can be categorized into two different simplified models, flexible or rigid, depending on their stiffness. Flexible diaphragms distribute forces depending on the tributary area of the facade between the stabilizing members. A rigid diaphragm is assumed to be infinitely stiff and will distribute forces depending on the relative stiffness in the bracing members and the position of the load resultant of wind load (Thelandersson & Larsen, 2003).

For a rigid diaphragm, the rotational centre of the building is used to calculate the forces in each horizontal element and the resulting global torsional moment in the structure. Coordinates for the rotation center is calculated using the stiffness of each bracing member in the horizontal direction and the position of the member:

$$x_t = \frac{\Sigma(a_i S_{yi})}{\Sigma S_{yi}}, \qquad y_t = \frac{\Sigma(b_i S_{xi})}{\Sigma S_{xi}} \tag{2.1}$$

where
$S_{xi}$ = stiffness of bracing member $i$ in x-direction
$S_{yi}$ = stiffness of bracing member $i$ in y-direction
$a_i$ = position of bracing member center in x-direction
$b_i$ = position of bracing member center in y-direction

The stiffness in a wall $i$ is given by

$$\frac{1}{S_i} = \frac{1}{S_{si}} + \frac{1}{S_{bi}} \qquad [(N/m)^{-1}] \tag{2.2}$$

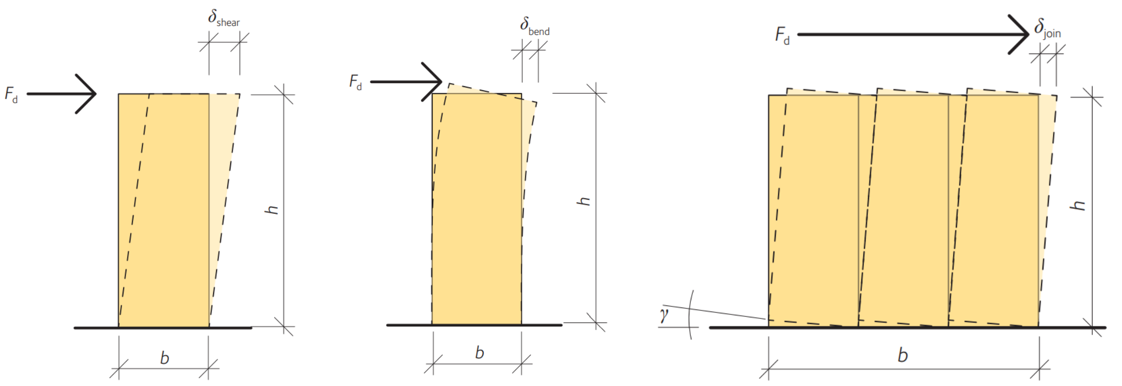
where
$S_{si}$ = shear stiffness of wall $i$ $\quad [N/m]$
$S_{bi}$ = bending stiffness of wall $i$ $\quad [N/m]$

$$S_{si} = \kappa_s \frac{E_i A_i}{h_i} \tag{2.3}$$

$$S_{bi} = \kappa_b \frac{E_i I_i}{h_i^3} \tag{2.4}$$

for bracing shear walls, coefficients for horizontal loads applied to top of the walls are
$\kappa_s = 1/3$
$\kappa_b = 3$

For CLT panels as horizontal bracing walls, Svenskt Trä (2019) suggests using three types of displacements from horizontal loads. These modes of deformation in CLT shear walls are shown in Figure 2.4. Three types of stiffness can be derived from the stiffness in each wall. Furthermore, the shear stiffness can be defined by the shear modulus $G$




**(a)** **(b)** **(c)**

**Figure 2.4:** Modes of displacement in CLT shear walls in (a) shear (b) bending (c) joints (Svenskt Trä, 2019).

$$S_{si,CLT} = \frac{G_i\ A_i}{h_i} \tag{2.5}$$

and the stiffness against joint displacement is

$$S_{joint} = \frac{b_i\ K_{ser,i}}{h_i} \tag{2.6}$$

where $K_{ser,i}$ = stiffness of the joint

The equation for bending stiffness, $S_{bi}$ is unchanged for a CLT panel compared to the general case,

If the resultant of the wind load acts directly in the line of the rotational center, there is no resulting global torsional moment in the stabilising system. When the resultant of the wind load does not align with the rotational center, then a torsional moment is acting on the structure. The torsional moment in calculated using

$$T = H \cdot e \tag{2.7}$$

where
$H$ = horizontal resultant of wind load
$e$ = eccentricity of the resultant relative to rotational center of the structure

A global torsional moment causes an additional horizontal resultant force in the bracing members. The resultant on one member $i$ is given by

$$H_{i,C} = T \frac{S_i \cdot r_i}{\Sigma(S_i \cdot r_i^2)} \tag{2.8}$$

where $r_i$ = radial distance from rotation centre.
The horizontal force resultant from torsion is combined with the force from translation on a bracing member

$$H_{i,B} = H \frac{S_i}{\Sigma S_i} \tag{2.9}$$

By adding (2.8) and (2.9) which is shown in Figure 2.5, the resulting force in one member $i$ is given by

$$H_i = H_{i,B} + H_{i,C} \tag{2.10}$$

(Engström, n.d)

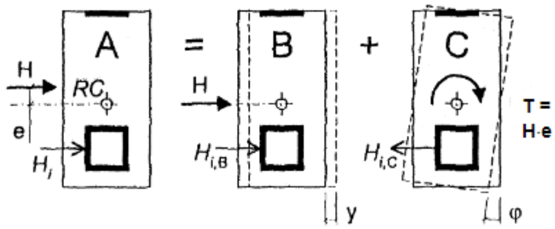



**Figure 2.5:** Force with an eccentricity relative to the rotational center (Engström, n.d).

The distribution of horizontal forces should be taken into account when deciding where to add horizontal bracing since uneven force distributions and torsional moment in the structure should be avoided.

### 2.4.1 Stabilizing elements

The elements that make up a horizontally stable member in a building can be done in different ways

#### Diagonally braced frames

By adding diagonal bracings in a frame structure, horizontal loads can effectively be resisted. The bracing member can work in either tension or compression. The bracings together with the frame forms a stiff vertical truss. Bracing is effective in resisting horizontal loads and creating a stiff structure with minimum material (Smith & Coull, 1992). A main drawback when using bracing is the risk of obstructing in the architectural planning of windows, doors etc. But if well planned it can be integrated in the design for a pleasant aesthetic view.

#### Rigid frame structures

Rigid frames are frame structures without bracing elements that are designed to be stable against horizontal forces. In a rigid frame the beams-column connections are designed to be moment resistant. The moment resistance is achieved through adding rigidity to the connection by utilizing more material locally or increasing stiffness. Because of demands of rigid connections, stiffer material such as reinforced concrete and steel are more suitable to use than timber for this type of frame structure. Unlike a diagonally braced frame, a rigid frame structure does not obstruct the architectural planning to the same extent.

**Shear wall**

A wall that functions as a horizontal bracing member in a structure is called a shear wall. A shear wall transfers horizontal forces down to the foundation through shear stresses in the wall. The moment caused by horizontal forces in the shear wall are counteracted by vertical forces in the foundation. The wall must therefore be anchored to the foundation to stop an uplifting effect of the wall. The benefit of a shear wall is its potential to simultaneously work as vertical and horizontal load-bearing while functioning as an architectural wall. The disadvantage is that it loses much of its function where openings are made in the wall.

**Core structure**

A core structure is a bracing element that often consists of four bracing members that forms a rectangular shape and works as a horizontal stabilization in two directions. A core structure can alone resist torsional moments in the structure. Examples of material used in a core are CLT-panels, reinforced concrete or a steel truss. The benefit of a core is that it generally also functions as walls to a staircase and elevator shaft. The disadvantage is that there is less freedom in placement of a core and that it takes a large amount of space in a building.

## 2.5 Structural systems in timber-hybrid structures

Timber-hybrid buildings can be designed using different variations of structural systems. The structural systems relevant to high rise timber-hybrid building are discussed in this chapter.

### 2.5.1 Framed system

A frame system is another type of structural system that can be used as building system. The frame system can consist of rigid frames, braced frames or a combination of the two. The members in a timber frame system are often made out of glulam. The frame, which supports the building, is often connected to a core that contributes to the horizontal bracing. Despite the risks of obstructing windows and doors, the frame system is versatile and it allows the designer to have more freedom in terms of designs and layout (Green & Taggart, 2017) . It allows for more open areas inside the building, making it valuable from an architectural perspective. This make it possible to use for both commercial buildings were open space is generally desired, but also as a residential building there i flexibility in floor planning.

**Figure 2.6:** Construction of a timber frame system used in Bullit Center, Seattle, USA. The glulam columns and beams are connected to a diagonally braced steel frame core (Stamets 2012).

### 2.5.2 Panel system

A timber-hybrid structure can be built using a panel system, in which the load-bearing structure is made of timber panels, typically as covered timber stud frames, LVL or CLT. These panels are able to transfer vertical loads and also act as shear walls to provide stability against wind loads. To enhance the overall stability of the structure, both horizontally and vertically, the panel system can be supported by core structures, which often surround the staircase and elevator shafts. One of the benefits of building with a panel system is that structural elements and modules can be easily prepared with doors, windows, and insulation in a factory, and lifted into place on the construction site (Green & Taggart, 2017). The disadvantage of a panel system is that it creates few open spaces, which generally limits the building to hotel or residential function.

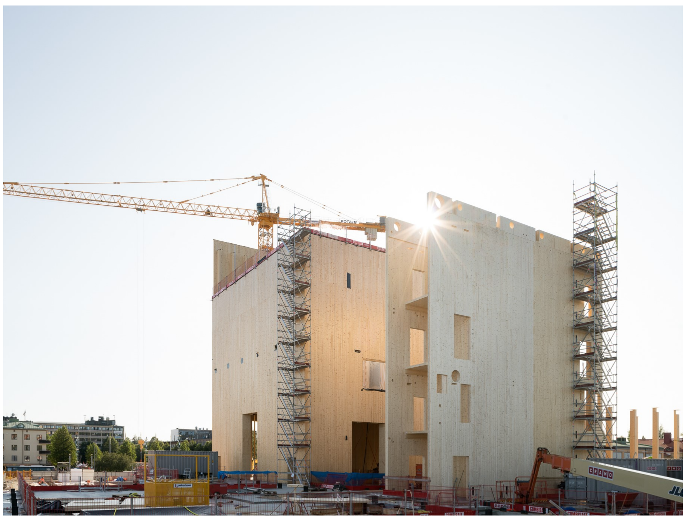


**Figure 2.7:** A panel system using CLT panels in many parts of Sara Kulturhus, Skellefteå Photo: Martinsons / Jonas Westling.

### 2.5.3 Hybrid system

A hybrid structural system uses both panel and frame types of structural elements. By using a combination of the two element types, the structure can be optimized with the benefit of both systems. Hybrid systems are a common way to design structural systems in timber. This is due to multiple reasons, including structural, architectural, production and environmental considerations (Green & Taggart, 2017). Using a combination of different types of structural timber elements can allow for a more efficient use of materials and improved freedom for the designer. The tallest timber-hybrid buildings today utilize hybrid structural systems, which suggests that hybrid systems are effective and versatile.

**Figure 2.8:** Hybrid system of Brock Commons Tallwood house, Vancouver, Canada. CLT floors and glulam columns are braced by concrete cores (Credit: naturallywood.com, photographer: KK Law).

## 2.6 Examples of existing high rise timber and timber-hybrid structures

The tallest existing timber-hybrid buildings in the world have been finished in the past ten years. An increased interest of reducing the environmental impact in construction has made timber a more appealing alternative in high-rise buildings. It has simultaneously become possible to increase the height of timber buildings because of availability of EWP with suitable structural properties combined with an improved knowledge of fire safety. This chapter aims to study the tallest timber-hybrid buildings today to create a better understanding of different structural solutions.

The Brock Commons Tallwood house in University of British Columbia, Vancouver Canada is an 18 storey timber-hybrid structure with a height of 54 m completed in 2017. The building is built to serve as a student apartments complex. The bottom storey is constructed using concrete columns which support a concrete slab floor. Above the first floor, the building is constructed with glulam columns and

CLT floors. These structural elements are connected to two concrete cores that act as horizontal bracing units, both of which reach to the top of the building (Naturally:wood, 2017). The concrete cores provide stiffness against horizontal wind loads. The building uses a hybrid structural system since it uses both glulam column frame elements and CLT floor panel elements.

Mjøstårnet is a tall timber-hybrid building that was completed in Brumunddal, Norway in the year 2019. The building has 18 floors which make up 68.2 m and functions as a combination of commercial and apartment building. The structure's architectural height reaches 85.4 m if the pergola glulam construction on top of the building is included. The structural system of Mjøstårnet is a hybrid system with a frame of glulam beams which support LVL panel floors. Mjøstårnet's frame is built as a truss system that resist the vertical and horizontal forces by providing stiffness for the building. Floors 2-11 are LVL floors made of timber. Floors 12-18 are reinforced concrete slabs floors (Abrahamsen, 2018). Adding concrete floors improves dynamic responses by increasing mass. The building has a core of CLT walls that provides vertical load bearing for staircases and elevators but does not function as a horizontal stabilizing elements.

Sara kulturhus is a culture and hotel building in Skellefteå. It was finished in 2021 with a height of 80 m (Martinsons, n.d.). Sara kulturhus is 20 storeys tall with a culture house in the lower storeys of the building and a hotel tower on the floors above. It is a timber-hybrid that is built with a hybrid structural system of CLT panels combined with glulam and steel trusses. The hotel part, which makes up the largest height is built as a panel system using CLT timber modules (Whitearkitekter, n.d.). Concrete floors are placed in the three highest storeys in order to reduce the wind induced oscillations. Furthermore, a concrete floor on the fifth storey works in combination with a steel truss to support the timber modules and distribute loads to supporting structures on the lower storeys (Brandt, 2021).

HoHo Tower, Vienna is a residential and commercial timber-hybrid building that was completed in 2020. It is an 84 meter tall building with 24 storeys. It is a timber-hybrid structure where timber and concrete are used building materials with 75% of construction material being timber (HASSLACHER group, n.d.). The structural system is built surrounding three concrete towers that work as bracing units. Around the cores is a timber-hybrid system using a frame of glulam columns and concrete beams. The concrete beams support prefabricated diaphragm floors made of composite elements of CLT and concrete (Woschitz, 2015).

Treet is a 14 floor apartment building in Bergen, Norway. It was completed in 2015 and was at that time the tallest timber building in the world (Abrahamsen, 2015). It is constructed using prefabricated timber modules made as framework, supported by glulam trusses at the facade. The floors on storeys 5, 10, 13 and the roof consists of concrete slabs. The function of these concrete slabs is partly to increase the building's mass to improve its dynamic response. The concrete slabs also function as diaphragms by acting as connections between glulam trusses (Abrahamsen, 2015).

The Ascent, Milwaukee was completed in 2022 with a height of 86.4 m. It is currently the tallest timber-hybrid tower. The building is a mixed-use building with 25 storeys, where the bottom six storeys are built with reinforced concrete and the 19 storeys above are built with timber. The concrete part is built as a column and slab system. The timber part is a frame system with glulam columns and beams in the frame that support one way spanning CLT floors. Two concrete cores extend from the bottom storey to the top storey. Their functions are to add stiffness to the structural system (Gonchar, 2022).

(a) (b) (c)

(d) (e) (f)

**Figure 2.9:** (a) Mjöstornet, Bumunddal Source: Wikimedia Commons (b) Sara Kulturhus, Skellefteå Source: Wikimedia Commons (c) Brock Commons, Vancouver Source: UBC Media Relations (d) Treet, Bergen Source: Wikimedia Commons (e) HoHo Tower, Vienna Source: Wikimedia Commons (f) The Ascent, Milwaukee Source: Wikimedia Commons.

## 2.7 Structural dynamics

When forces act on a structure, dynamic effects are induced in the building. These effects are oscillations and vibrations which for a tall slender building cannot be neglected because of the high levels of dynamic response these buildings have. Structural dynamics is the theory of mathematically modelling a structure and to analyze its dynamic behaviour. A structural dynamics problem considers time varying excitation on a structure, and the effect of acceleration in the structure (Craig Jr. & Kurdila, 2006). These effects are neglected in a static problem, or are approximated by introducing dynamic factors to analyses.

### 2.7.1 Dynamic modelling of mass and stiffness

In order to do dynamic calculations for a structure, mass and stiffness in the structure must be modelled. A structure can be modelled for calculations using a continuous model. It can also be discretized into nodes with motion variables called degrees-of-freedom (dofs). A structural system can be modelled either as one single degree-of-freedom (SDOF), or a model with multiple degrees-of-freedom (MDOF). An SDOF model is an approximation of a structure that is applicable for simplified dynamic calculations of high-rise buildings (Landel, 2022). Undamped SDOF systems can be described as a mass with a spring which is shown in Figure 2.10.

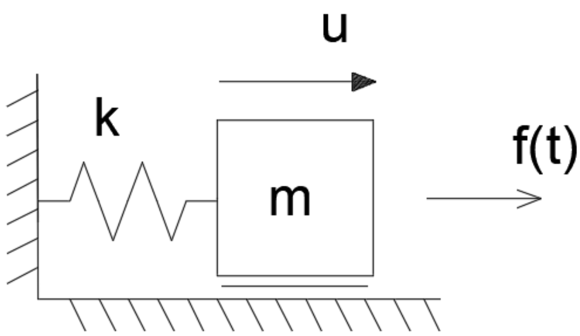


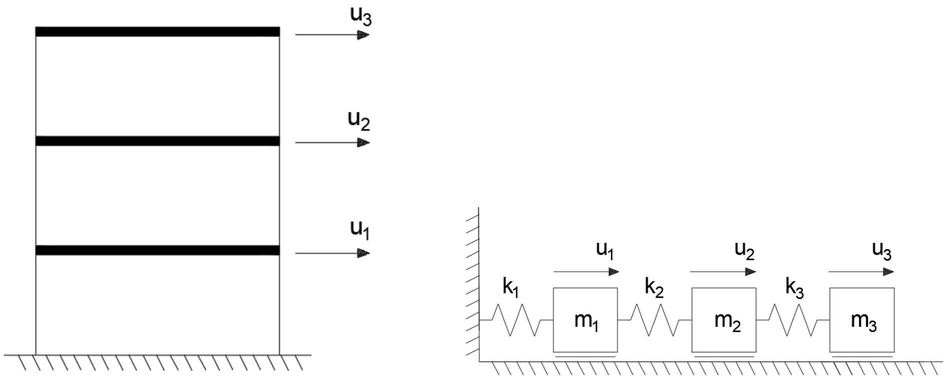

**Figure 2.10:** Model of an SDOF mass-spring system with displacement $u(t)$, loaded by a force $f(t)$.

The degree of freedom has the spring stiffness $k$ and the mass $m$. Newtons 2nd law for the SDOF system gives:

$$m\ \ddot{\mathrm{u}} + k\ \mathrm{u} = f(t) \tag{2.11}$$

where
u = displacement
ü = acceleration (second time derivative of u)
$f(t)$ = external force on the system at time $t$

Many structures have a complex geometry that can not be modelled using an SDOF system. An a alternative is to model the system using a multiple degree-of-freedom (MDOF) model. An MDOF system can be used to model an approximation of a multi-storey building as a mass spring system. An example of how to model a 3-dof multi-storey building is by using a lumped-parameter model where the parameters of the structure such as mass and stiffness are assumed to be concentrated in the dofs. This is illustrated in Figure 2.11 The three dofs are the horizontal degrees-of-freedom on each floor of the building.




**(a)** **(b)**

**Figure 2.11:** Example of three storey building modelled as an MDOF mass-spring system.

Looking at the free body diagram in Figure 2.12, Newtons second law can be written for each element, while the spring force can be described as a stiffness constant multiplied with the relative displacement of each element.

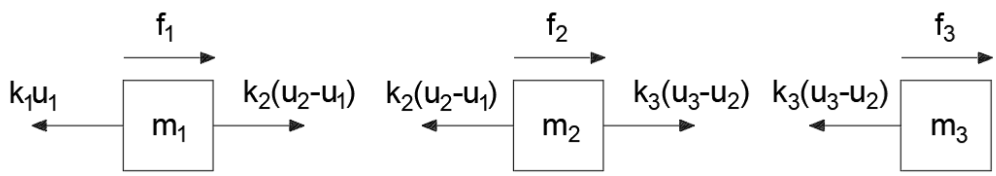



**Figure 2.12:** Free-body diagram of Figure 2.11 (b).

$$\begin{cases} m_1\ddot{\mathrm{u}}_1 + (k_1 + k_2)\mathrm{u}_1 - k_2\mathrm{u}_2 = f_1(t) \\ m_2\ddot{\mathrm{u}}_2 - k_2\mathrm{u}_1 + (k_2 + k_3)\mathrm{u}_2 - k_3\mathrm{u}_3 = f_2(t) \\ m_3\ddot{\mathrm{u}}_3 - k_3\mathrm{u}_2 + k_3\mathrm{u}_3 = f_3(t) \end{cases} \tag{2.12}$$

In matrix form (2.12) can be written as

$$\begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_3 \end{bmatrix} \begin{bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \\ \ddot{u}_3 \end{bmatrix} + \begin{bmatrix} k_1 + k_2 & -k_2 & 0 \\ -k_2 & k_2 + k_3 & -k_3 \\ 0 & -k_3 & k_3 \end{bmatrix} \begin{bmatrix} u_1 \\ u_2 \\ u_3 \end{bmatrix} = \begin{bmatrix} f_1(t) \\ f_2(t) \\ f_3(t) \end{bmatrix}$$

This system of equations is one example of the structural dynamic equation and can be generalized to $n$ degrees-of-freedom.

$$\mathbf{M}\ddot{\mathbf{u}}(t) + \mathbf{K}\mathbf{u}(t) = \mathbf{f}(t) \tag{2.13}$$

where:

$$\mathbf{M} = \begin{bmatrix} m_{11} & \cdots & m_{1n} \\ \vdots & \ddots & \\ m_{n1} & & m_{nn} \end{bmatrix} \qquad \mathbf{K} = \begin{bmatrix} k_{11} & \cdots & k_{1n} \\ \vdots & \ddots & \\ k_{n1} & & k_{nn} \end{bmatrix}$$

Are the $(n\ x\ n)$ mass and stiffness matrices, and:

$$\ddot{\mathbf{u}}(t) = \begin{bmatrix} \ddot{u}_1 \\ \vdots \\ \ddot{u}_n \end{bmatrix} \qquad \mathbf{u}(t) = \begin{bmatrix} u_1 \\ \vdots \\ u_n \end{bmatrix} \qquad \mathbf{f}(t) = \begin{bmatrix} f_1 \\ \vdots \\ f_n \end{bmatrix}$$

Are the $(1\ x\ n)$ displacement, acceleration and force vectors (Craig Jr. & Kurdila, 2006).

### 2.7.2 Eigenmodes and eigenfrequencies

In structural dynamics, frequency $\omega$ is a measurement of oscillation in a structure. The unit of measurement of $\omega$ is usually [rad/s]. Using radians as a measurement for displacement is beneficial because a freely vibrating system generally follows harmonic motion which has a sinusoidal displacement with time. The movement of a dof can therefore be described as a sinusoidal period of $2\pi$ radians.

A systems eigenfrequencies, also known as natural/resonance frequencies, are frequencies where a system tends to oscillate with the same amplitude without external forces, when released from an offset position. To determine eigenfrequencies of an undamped MDOF system, free vibration with no external force can be solved using the structural dynamics equation (2.13)

$$\mathbf{M}\ddot{\mathbf{u}}(t) + \mathbf{K}\mathbf{u}(t) = \mathbf{0} \tag{2.14}$$

The displacement of each dof in free vibration can be assumed to be sinusoidal with an amplitude $\hat{\mathbf{u}}$. The displacement over time can then be described as

$$\mathbf{u}(t) = \hat{\mathbf{u}} \cdot \sin(\omega t) \tag{2.15}$$

$$\ddot{\mathbf{u}}(t) = \frac{\partial^2}{\partial t^2}\Big(\hat{\mathbf{u}} \cdot \sin(\omega t)\Big) = -\hat{\mathbf{u}}\ \omega^2 \cdot \sin(\omega t) \tag{2.16}$$

(2.15), (2.16) into (2.14) gives

$$[\mathbf{K} - \omega^2\mathbf{M}]\hat{\mathbf{u}} \cdot \sin(\omega t) = \mathbf{0} \tag{2.17}$$

The non-trivial solution to this equation exists for a system where

$$\det(\mathbf{K} - \omega^2\mathbf{M}) = 0 \tag{2.18}$$

Assuming the stiffness $\mathbf{K}$ and mass $\mathbf{M}$ remain fixed, the eigenvalue solutions to the problem can be defined as $\omega_k^2, \quad k = 1, 2, 3, ..., n$. Where $n$ is the number of system dofs. A vector $\boldsymbol{\phi}_k$ can be defined as the vector which for the eigenvalue $\omega_k^2$ is the amplitude of displacements $\boldsymbol{\phi}_k = \hat{\mathbf{u}}_k$. This vector is given by the $k$:th solution to the free vibration equation (2.17) which can be written as

$$[\mathbf{K} - \omega_k^2\ \mathbf{M}]\boldsymbol{\phi}_k = \mathbf{0} \tag{2.19}$$

Multiplying (2.19) with $\mathbf{M}^{-1}$ results in

$$[\mathbf{M}^{-1}\mathbf{K} - \omega_k^2\ \mathbf{I}]\boldsymbol{\phi}_k = \mathbf{0} \tag{2.20}$$

This can be compared to the standard form of an eigenvalue problem with eigenvalues and eigenvectors for a matrix $\mathbf{A}$

$$[\mathbf{A} - \lambda\mathbf{I}]\mathbf{x} = \mathbf{0} \tag{2.21}$$

where
$\mathbf{I}$ = unit matrix $(n\ x\ n)$
$\lambda$ = eigenvalue
$\mathbf{x}$ = eigenvector $(n\ x\ 1)$

By solving the eigenvalue equation (2.20), eigenvector $\boldsymbol{\phi}_k$ and eigenvalue $\omega_k^2$ are determined. The eigenfrequency, which is the frequency at mode $k$ is then calculated with

$$\omega_k = \sqrt{w_k^2} \qquad \left[\frac{rad}{s}\right] \tag{2.22}$$

The eigenvector $\boldsymbol{\phi}_k$ at mode $k$ is the relative amplitude of displacements in a sturucture between the dofs at that eigenfrequency. (Williams, 2016) . The eigenfrequency can also be expressed as a frequency of oscillation cycles per second [Hz]. This can be a more useful unit than radians per second in some applications.

$$f_1 = \frac{\omega_k}{2\pi} \qquad [Hz] \tag{2.23}$$

The time period can be described by the frequency of a system. The time period is the length of time it takes for one cycle of oscillation at a frequency $f_1$

$$T_k = \frac{1}{f_1} \qquad [s] \tag{2.24}$$

Another method of determining the eigenvalues of an MDOF system is by using Rayleigh's quotient (2.25). When using Rayleigh's quotient to calculate an eigenvalue, mode shape $\boldsymbol{\phi}_k$, mass $\mathbf{M}$, and stiffness $\mathbf{K}$ of the system must be known or approximated.

$$\omega_k^2 = \frac{\boldsymbol{\phi}_k^T \mathbf{K} \boldsymbol{\phi}_k}{\boldsymbol{\phi}_k^T \mathbf{M} \boldsymbol{\phi}_k} \tag{2.25}$$

To understand the effect of changing a structure, Rayleigh's theorems on added stiffness and added mass can be used to understand the dynamic behaviour. Rayleigh's theorems state how mass and stiffness influence an increase or decrease in resonance frequencies.

**Rayleigh's theorem on added stiffness:** For a system with stiffness $\mathbf{K}$ and mass $\mathbf{M}$, adding stiffness $\Delta\mathbf{K} > 0$, eigenfrequencies either stay put or <u>increase</u>.

**Rayleigh's theorem on added mass:** For a system with stiffness $\mathbf{K}$ and mass $\mathbf{M}$, adding mass $\Delta\mathbf{M} > 0$, eigenfrequencies either stay put or <u>decrease</u>.

*Proof*: The Rayleigh quotent (2.25) which is proven in Abrahamsson,(2019) gives a new eigenvalue $\bar{\omega}^2$ when increasing mass or stiffness:

$$\bar{\omega}^2 = \frac{\boldsymbol{\phi}^T(\mathbf{K}+\Delta\mathbf{K})\boldsymbol{\phi}}{\boldsymbol{\phi}^T\mathbf{M}\boldsymbol{\phi}} \geq \frac{\boldsymbol{\phi}^T\mathbf{K}\boldsymbol{\phi}}{\boldsymbol{\phi}^T\mathbf{M}\boldsymbol{\phi}} = \omega^2, \qquad \bar{\omega}^2 = \frac{\boldsymbol{\phi}^T\mathbf{K}\boldsymbol{\phi}}{\boldsymbol{\phi}^T(\mathbf{M}+\Delta\mathbf{M})\boldsymbol{\phi}} \leq \frac{\boldsymbol{\phi}^T\mathbf{K}\boldsymbol{\phi}}{\boldsymbol{\phi}^T\mathbf{M}\boldsymbol{\phi}} = \omega^2$$

Rayleigh's theorems on added mass and added stiffness demonstrate that any resulting increase or decrease in resonance frequencies in an undamped dynamic system are caused by changes in mass or stiffness in the structure.

### 2.7.3 Damping

Damping is the loss of energy in a system that prevents a system from oscillating indefinitely once set in motion. Damping in a building occurs due to many different reasons: Coulomb damping is caused by friction between surfaces in a structure. Radiation damping is caused by the dissipation of energy into the ground below the building. Aerodynamic damping comes from the friction between the structure and the surrounding flow of air. A building can also be constructed to intentionally increase the level of damping. A method of doing this is to include a tuned mass damper or viscous damper in the system (Williams, 2016).

For an SDOF-system with added damping, the damping can be modelled as a viscous damper that connects the particle to a rigid wall. This is illustrated in Figure 2.13. The force from the damper which acts on the system is proportional to the relative velocity of the particle to the wall (Williams, 2016).

$$F_D = v \cdot \dot{u} \tag{2.26}$$

where
$v$ = viscous damping coefficient [Ns/m]
$\dot{u}$ = velocity (first time derivative of u)

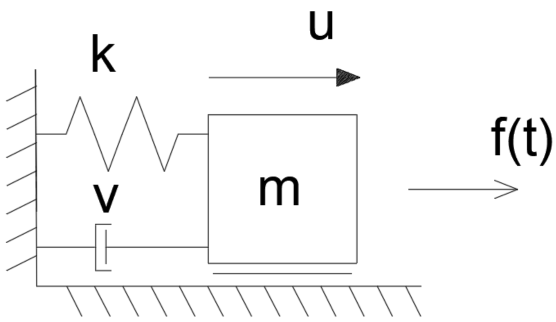



**Figure 2.13:** Damped SDOF mass-spring system with displacement $u(t)$, loaded by a force $f(t)$.

Newtons second law for the damped SDOF-system can be expressed as

$$m\ddot{u} + v\dot{u} + k u = f(t) \tag{2.27}$$

The viscous damping coefficient in a system is often difficult to derive analytically. Therefore, a common parameter used to describe the damping in a structure is the damping ratio $\xi$.

$$\xi = \frac{v}{2\sqrt{km}} \qquad [-] \tag{2.28}$$

Due to the several different causes of damping and the difficulty of analytically deriving viscous damping coefficients in a structure $\xi$ is often estimated as a global system parameter. The estimation can be done by using measured data from buildings with similar structural systems. The damping ratio is often in the range of 2.0% to 5.0% in civil engineering systems (Williams, 2016), and can be as low as 1% in timber buildings (Edskär, 2018). Because of this range, the class of damping in buildings is called subcritical damping, which means that the structure oscillates while the amplitude decays ($0 < \xi < 1$). When subcritical damping is added to a previously undamped SDOF system, the first natural frequency $\omega_1$ is shifted to the systems damped first natural frequency $\omega_d$,

$$\omega_d = \omega_1 \sqrt{1 - \xi^2} \tag{2.29}$$

The equation shows that for a building, the shift in the first natural frequency due to damping is very small and can therefore be neglected.

For an MDOF-system, the structural dynamic equation (2.13) is modified by adding the damping matrix and velocity vector according to Abrahamsson (2019).

$$\mathbf{M}\ddot{\mathbf{u}}(t) + \mathbf{V}\dot{\mathbf{u}}(t) + \mathbf{K}\mathbf{u}(t) = \mathbf{f}(t) \tag{2.30}$$

The mass and stiffness matrices $\mathbf{M}$ and $\mathbf{K}$ are assembled using contributions from the defined elements in a system. The damping matrix $\mathbf{V}$ however, is because of the reasons explained generally assembled on a global system level. One method of producing a generalized damping matrix for a system is by using Rayleigh damping (proportional damping). Rayleigh damping describes a damping matrix defined as a proportion of the mass and stiffness of the system:

$$\mathbf{V} = \alpha\mathbf{K} + \beta\mathbf{M} \tag{2.31}$$

where $\alpha$ and $\beta$ are defined so that for an eigenfrequency $\omega_k$:

$$\xi_k = \frac{1}{2}\Big(\frac{\alpha}{\omega_k} + \beta\omega_k\Big) \tag{2.32}$$

where $\xi_k$ = damping ratio at eigenfrequency $k$

By using (2.31), (2.32) and structural dynamics decoupling theory, a *transformed* damping matrix for a specific eigenfrequency, can be defined:

$$\bar{\mathbf{V}} = \mathrm{diag}(2\xi_k\ \omega_k\ \mu_k) \tag{2.33}$$

where $\mu_k = \boldsymbol{\phi}_k^T\mathbf{M}\boldsymbol{\phi}_k$ = modal mass

A physical viscous damping matrix $\mathbf{V}$ can be calculated from $\bar{\mathbf{V}}$ using equations discussed by Abrahamsson (2019):

$$\begin{cases} \mathbf{V} = \mathbf{\Phi}^{-T}\bar{\mathbf{V}}\mathbf{\Phi}^{-1} \\ \text{or,} \\ \mathbf{V} = \mathbf{M}\mathbf{\Phi}\ \mathrm{diag}(2\xi_k\ \omega_k\ \mu_k)\mathbf{\Phi}^T\mathbf{M} \end{cases} \tag{2.34}$$

where
$\mathbf{\Phi} = [\boldsymbol{\phi}_1\ \boldsymbol{\phi}_2 \cdots \boldsymbol{\phi}_n]$ = modal matrix

### 2.7.4 Frequency response

Frequency response is discussed by Abrahamsson (2019) and is here summarized: Assume a harmonic load and displacement response on the system so that:

$$\begin{cases} \mathbf{u}(t) = \hat{\mathbf{u}}\ e^{i\omega t} \\ \mathbf{f}(t) = \hat{\mathbf{f}}\ e^{i\omega t} \end{cases} \tag{2.35}$$

where
$\hat{\mathbf{u}}$ = displacement amplitude vector
$\hat{\mathbf{f}}$ = force amplitude vector

By differentating the harmonic function of displacement, the harmonic velocity and acceleration can be defnied using the displacement amplitude

$$\dot{\mathbf{u}} = i\omega\hat{\mathbf{u}}\ e^{i\omega t} \tag{2.36}$$

$$\ddot{\mathbf{u}} = -\omega^2\hat{\mathbf{u}}\ e^{i\omega t} \tag{2.37}$$

(2.35), (2.36) and (2.37) into (2.30) gives:

$$[-\omega^2\mathbf{M} + i\omega\mathbf{V} + \mathbf{K}]\hat{\mathbf{u}}\ e^{i\omega t} = \hat{\mathbf{f}}\ e^{i\omega t} \tag{2.38}$$

From this equation, the inverse matrix

$$[-\omega^2\mathbf{M} + i\omega\mathbf{V} + \mathbf{K}]^{-1} = \mathbf{H}_d \tag{2.39}$$

can be used to calculate the displacement amplitude from the force amplitude. The inverse matrix is called the system's displacement transfer function. From the system transfer function, the mobility transfer function can be defined $\mathbf{H}_v = i\omega\mathbf{H}_d$. And the accelerance transfer function $\mathbf{H}_a = -\omega^2\mathbf{H}_d$. The transfer functions are used to calculate the amplitude of harmonic displacement, velocity and acceleration

$$\begin{cases} \hat{\mathbf{u}} = \mathbf{H}_d\ \hat{\mathbf{f}} \\ \hat{\dot{\mathbf{u}}} = \mathbf{H}_v\ \hat{\mathbf{f}} \\ \hat{\ddot{\mathbf{u}}} = \mathbf{H}_a\ \hat{\mathbf{f}} \end{cases} \tag{2.40}$$

### 2.7.5 Tuned mass dampers

A tuned mass damper (TMD) is a commonly used tool to reduce excessive movement of the building (Elias & Matsagar, 2017). TMD:s are passive systems that absorb energy caused by external loads on the building. A TMD is tuned to have a resonance frequency that is similar to the problematic frequency of the building (Connor & Laflamme, 2014). By vibrating out of phase the damper reduces motions in the building. The TMD is tuned to respond to a specific frequency and will only be effective in a small range near that frequency. Two common types of tuned mass dampers are translational tuned mass damper (TTMD) and pendulum tuned mass damper (PTMD).

A TTMD is illustrated in Figure 2.14 and consists of a mass , springs and viscous dampers. The mass is rested on a support that allows it to move horizontally. The viscous dampers and springs are connected to the structural system which allows the mass to move out of phase with the building. A drawback of this system is that the required space since the mass of the often need to be large to counteract the movement of the building (Gutierrez Soto & Adeli, 2013).

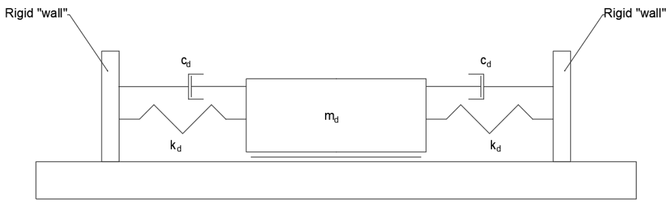



**Figure 2.14:** Illustration of a TTMD.

A PTMD is a system with a mass anchored to the building by cables. When motion of the building occurs, the PTMD produces a horizontal force which acts in the opposite direction of the floor movement (Gutierrez Soto & Adeli, 2013). Figure 2.15 illustrates how the inertia of the pendulum produces a force in the opposite direction to movement in the building. By changing length of the cables the PTMD resonance frequency can be tuned. A drawback of this system is the cable length needed for the pendulum which may occupy several floors.

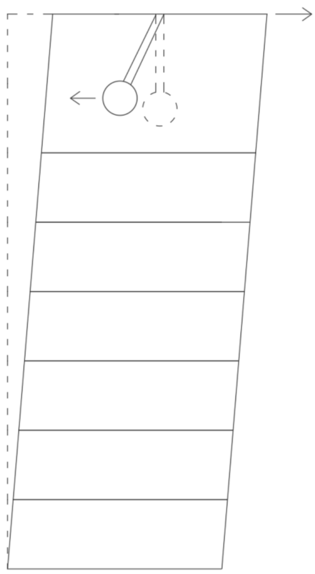


**Figure 2.15:** Illustration of a PTMD.

A tuned mass damper can be designed using a simplified method where the building and damper are each modelled as single dofs, connected with a spring and a viscous damper. The 2-dof model illustrated in Figure 2.16 can be used to calculate the mass, spring stiffness and viscous damping needed in the TMD to achieve an equivalent damping ratio $\xi_{Eq}$ in a building with known dynamic properties. The equivalent damping $\xi_{Eq}$, can be assumed to act globally on the building in the same manner as the structural damping.

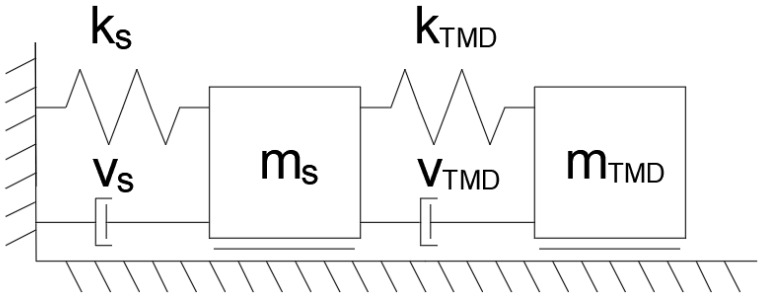



**Figure 2.16:** Simplified 2-dof system of a building with added damper.

where

| | | |
|---|---|---|
| $m_s$ | = | Modal mass of the original building |
| $k_s$ | = | Horizontal stiffness of the building |
| $v_s$ | = | Structural damping in the original building |
| $m_{TMD}$ | = | Mass of the TMD |
| $k_{TMD}$ | = | Stiffness of spring connecting the building to the TMD |
| $v_{TMD}$ | = | Viscous damper connecting the building to the TMD |

For a building with the first known natural frequency $f_1$ and modal mass $m_s$, the horizontal stiffness $k_s$ is calculated with (2.25), which for an SDOF system results in $k_s = m_s\omega_1^2$, where $\omega_1 = 2\pi f_1$.

The structural damping using (2.28) results in $v_s = \xi_s \; 2\; \sqrt{k_s m_s}$, where $\xi_s$ = damping ratio of structural damping.

Then, the buildings frequency response with only structural damping can be calculated using (2.39) and (2.40).

When a TMD is added to the system, $\omega_{TMD}$, $m_{TMD}$ and $\xi_{TMD}$ can be chosen in design to optimize the damper. The spring stiffness and viscous damping can be calculated using (2.25) and (2.28). $\omega_{TMD}$ is the natural frequency of the isolated TMD system. $\xi_{TMD}$ is the equivalent damping in the viscous damper connecting the TMD to the building.

The frequency response of the undamped and the damped building is shown in Figure 2.17. In the figure, the D-axis shows the relative displacement from a harmonic force $f(t)$ compared to the static displacement of an equally large static force on the system. The TMD introduces an additional dof which causes the simplified system to contain 2 shifted eigenfrequencies. The new eigenmodes have significantly smaller responses than the original SDOF system. In Figure 2.17, the red curve illustrates response in the original building with structural damping of 1.5%. The system with a tuned mass damper which causes an equivalent damping of $\xi_{eq} = 4\%$ is also included as the black curve, with the blue curve illustrating an SDOF system with damping ratio of 4% for reference.

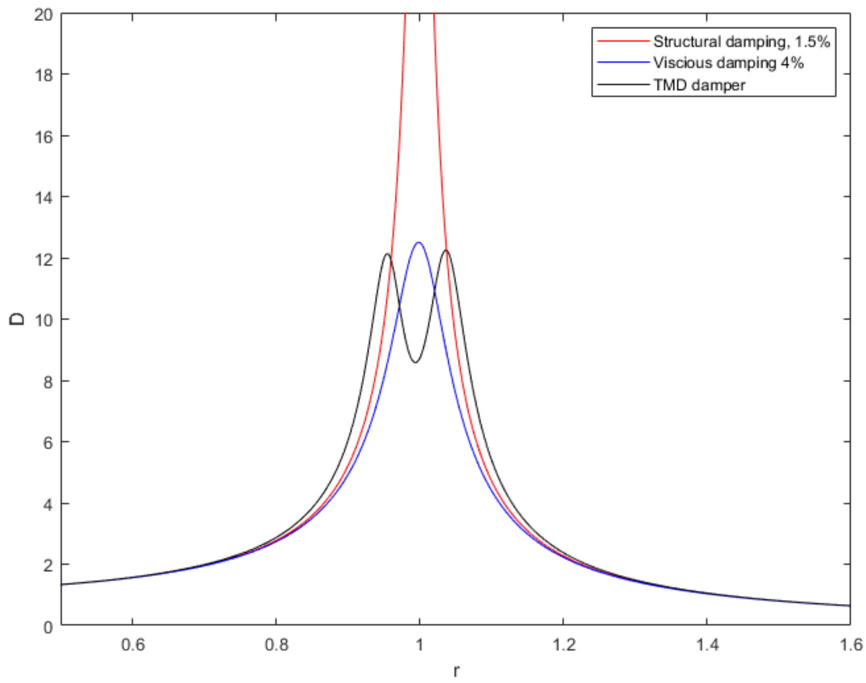



**Figure 2.17:** Normalized displacement transfer functions of different damped systems.

The relation between the TMD mass and the building's modal mass (mass ratio) can be used to determine if the TMD design is reasonable for the building.

$$\overline{m} = \frac{m_{TMD}}{m_s} \tag{2.41}$$

A typical range for $\overline{m}$ is between 1% to 10% (Connor & Laflamme, 2014), however it is important to note that a heavier damper might cause high stresses in the vertical load bearing system of a structure. Figure 2.18 displays the mass ratio of the damper compared to the equivalent damping of a system. It can be seen that a mass ratio of 2% results in around 5% equivalent damping in the system.

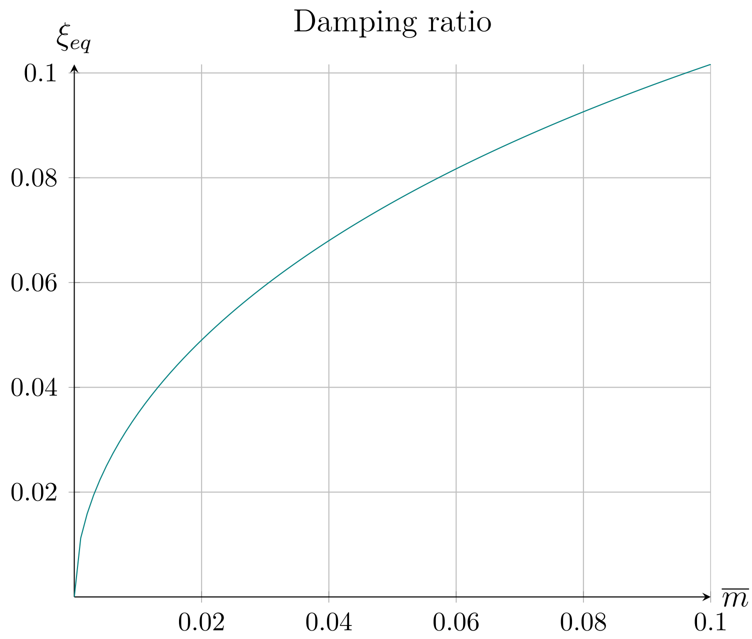



**Figure 2.18:** Equivalent damping ratio for an optimally designed TMD (Connor & Laflamme, 2014).

In order to optimize the function of a tuned mass damper, the resonance frequency of the isolated TMD can be chosen. When the TMD is designed with the optimal frequency, both peaks in the new system have equal displacement amplitudes. In general, the optimal TMD eigenfrequency is close to the building's eigenfrequency to allow the TMD to oscillate out of phase with the remaining structure.

$$\omega_{TMD} \approx \omega_1 \tag{2.42}$$

The optimal natural frequency of the tuned mass damper can however be more accurately approximated using

$$\omega_{TMD} = f_{opt}\, \omega_1 \tag{2.43}$$

where

$$f_{opt} = \frac{\sqrt{1 - 0.5\overline{m}}}{1 + \overline{m}} \tag{2.44}$$

The location and amplitude of the two peaks depend on the damping in the viscous damper connecting the TMD to the building. When $\xi_{TMD} = 0\%$ the two peaks are separated on each side of $r = 1$. When $\xi_{TMD}$ is increased the peaks move towards $r = 1$ and gain a lower amplitude. If $\xi_{TMD}$ is too small, the peaks merge into one peak with increased amplitude. Therefore, there is an equivalent damping of the viscous damper which optimizes the TMD performance (Connor & Laflamme, 2014). The equation for approximating the optimal $\xi_{TMD}$ is

$$\xi_{TMD} = \sqrt{\frac{\overline{m}(3 - \sqrt{0.5\overline{m}})}{8(1 + \overline{m})(1 - 0.5\overline{m})}} \tag{2.45}$$

## 2.8 Wind loads

A common cause of dynamic responses in a building is the effect of wind acting on it. Wind is defined as movement of air relative to the earth's surface, driven by atmospheric air pressure gradients. The movement of air is made up of eddies with varying velocities and rotational characteristics. Wind has fluctuations at a macro-meteorological scale with periods in orders of hours or days, and micro-meteorological scale with periods in orders of seconds or minutes. Wind fluctuations at the micro scale are called gusts. The wind velocity can then be described as a macro wind flow superimposed with micro wind gusts (Isyumov, 2012).

At earth's surface, the wind has almost no velocity as a result of air particles being trapped in the uneven surface. A second layer of particles are free to move above the trapped layer with some velocity. The friction of air rapidly decreases with each layer, causing wind velocities to increase with height over the earth's surface as friction gets lower (Roberts, 2012). At a certain height, the wind speed reaches a level called the gradient wind velocity (Mendis et al., 2007). At the height of the gradient wind velocity, the wind speed no longer by surface friction and can be assumed to be constant at higher heights. The height at which the wind reaches the gradient wind velocity depends on the levels of friction between surfaces and the air near the ground level. Figure 2.19 illustrates the effect of terrain up to the gradient wind velocity. EuroCode 1-4 considers the friction levels using terrain categories, which depend on terrain roughness. There are five terrain categories, from 0 to IV, where 0 is "Sea or coastal area exposed to the open sea" with the lowest gradient wind velocity height. Terrain category IV is "Area in which at least 15% of the surface is covered with buildings and their average height exceeds 15 m". This category has the highest gradient wind velocity height.

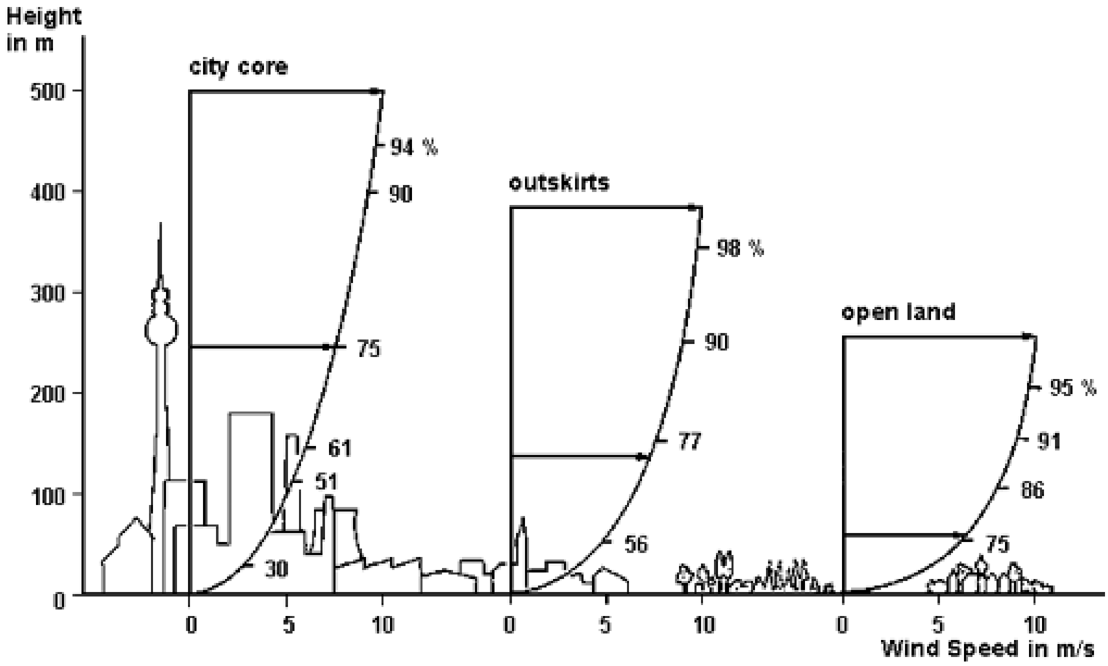



**Figure 2.19:** Wind speed depending on surface roughness with percentages of gradient wind velocity.

Wind flowing around a building causes large fluctuations in pressure on the facade (Mendis et al., 2007). The average pressure of the wind depends on the surrounding terrain, geographic location and the building's geometry. The surrounding terrain is accounted for in EC1 and EKS12 with terrain categories that govern the roughness length $z_0$ [m]. The geographic location of the building determines wind velocities used in calculations by being associated with a basic wind velocity, $v_b$ [m/s]. The basic wind velocity is based on measured weather station data. It is an approximate value of the highest wind gust speed which has a duration of 10 minutes at 10 m above ground level, with a return period of 50 years. The building's geometry is accounted for with the external pressure coefficient $c_{pe}$. The pressure coefficient values are presented in tables in EC1. It is depending on the building's geometry and the wind direction. The height of the building is also considered in wind pressure calculations.

For high-rise buildings, wind loads cause significant dynamic responses in the structure. Dynamic oscillations can occur in an along-wind or cross-wind direction to the wind, and in torsion around the buildings rotational center. The amplitude of the oscillations depends on the magnitude of the wind pressures acting on the building and also depends on the dynamic properties of the buildings structural system (Mendis et al., 2007). Oscillations in the buildings natural frequency can be caused by a wind load with the same frequency. There are however other phenomena which can cause variations in loading on the building at its natural frequency. Examples of such phenomena are buffeting, vortex shedding, galloping, and flutter. Buffeting is the turbulence response of the building parallel to the wind direction. Slender, lighter or more flexible buildings are known to be sensitive to this phenomenon (Landel, 2022). Vortex shedding is the formation of vortexes in the wake of a building subjected to a steady flow of wind (Roberts, 2012). When a wind vortex is formed on one side of the building, the building is subjected to a force from that side and a vortex is formed on the other side. The building is then loaded with a varying load perpendicularly to the wind direction (Boverket, 1997). Vortex shedding and galloping are more sensitive to winds transversal to the building. Flutter is a coupled motion which often is a combination of bending and torsion in the building (Mendis et al., 2007).

**Background and resonance response**

The response of a building due to wind gusts is divided into a background response and a resonance response. The background response in the building is caused by the wind acting as a stationary load (Boverket, 1997). It is the part of the kinetic energy in the wind which is not in resonance with the structure. The background response is accounted for in EKS12 with the background factor $B^2$, eq. (2.55). The resonance response is the response in the building due to a combination of dynamic movements in the structure and the surrounding wind. It is taken into account in EKS12 as the resonance response $R^2$, eq. (2.56).

The peak factor $k_p$, eq. (2.49) is the multiple of the r.m.s wind pressure which results in the 10 minute peak wind pressure. The peak factor is constant until the up-crossing frequency, $\nu$ (eq. 2.50) reaches 0.0822 Hz. At higher up-crossing frequencies the peak factor increases as the frequency increases. When the background factor $B^2$ is large in relation to the resonance response $R^2$ the up-crossing frequency gets closer to the building's natural frequency. When $B^2$ is smaller, the up-crossing frequency is reduced (Boverket, 1997).
The von Kármán wind energy spectrum, F eq. (2.59) is a factor that scales with the energy of the wind acting on the building depending on the building's natural frequency and the mean wind velocity $v_m(h)$, eq. (2.63). The factor is shown on a logarithmic scale in Figure 2.20. If the mean wind velocity remains constant, a lower natural frequency means that the wind acts with higher energy on the structure since frequencies below the peak are rarely seen in buildings. A building's resonance response, $R^2$ is proportional to the Kármán wind energy.

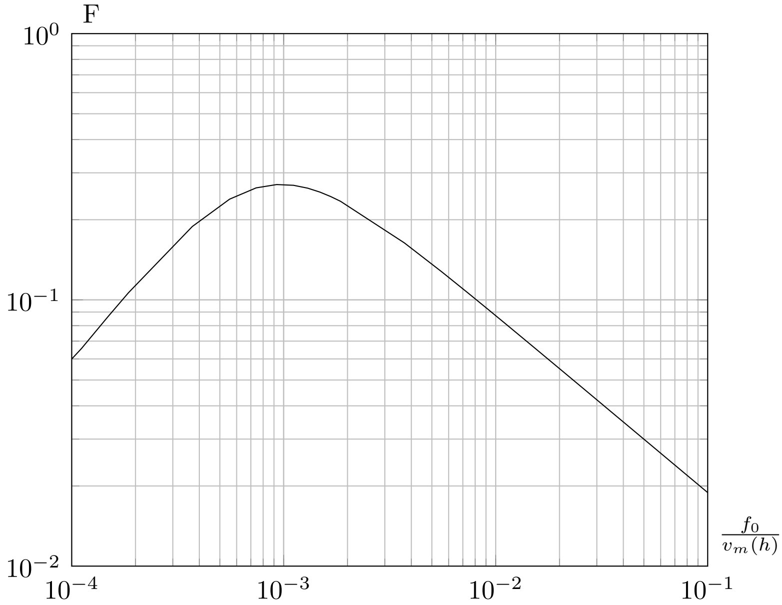



**Figure 2.20:** von Kármán wind energy on a logarithmic scale, equation (2.59).

### 2.8.1 Human response to wind load

The acceleration in buildings caused by oscillations lead to negative consequences on the inhabitants. Consequences can be categorized into motion syndrome, sopite syndrome and fear. Motion syndrome arises when there is higher motion for a shorter time period, up to a few hours. Sopite syndrome, which is also called regular sleepiness and reduced work performance happens at lower levels of motion for an extended time, up to a few days. Occupants can feel fear when there is high levels of motion in the building (Landel, 2022). Mendis et al., (2007) present effects on humans depending on acceleration in the building in Table 2.1

**Table 2.1:** General human perception levels (Mendis et al., 2007).

| LEVEL | ACCELERATION ($m / sec^2$) | EFFECT |
|---|---|---|
| 1 | < 0.05 | Humans cannot perceive motion |
| 2 | 0.05 - 0.1 | a) Sensitive people can perceive motion;<br>b) hanging objects may move slightly |
| 3 | 0.1 - 0.25 | a) Majority of people will perceive motion;<br>b) level of motion may affect desk work:<br>c) long - term exposure may produce motion sickness |
| 4 | 0.25 - 0.4 | a) Desk work becomes difficult or almost impossible;<br>b) ambulation still possible |
| 5 | 0.4 - 0.5 | a) People strongly perceive motion;<br>b) difficult to walk naturally;<br>c) standing people may lose balance. |
| 6 | 0.5 - 0.6 | Most people cannot tolerate motion and are unable to walk naturally |
| 7 | 0.6 - 0.7 | People cannot walk or tolerate motion. |
| 8 | > 0.85 | Objects begin to fall and people may be injured |

## 2.9 Standards

### 2.9.1 ISO 10137 - Serviceability of buildings and walkways against vibration

ISO 10137 (SS, 2008) is a Swedish standard, in the standard limits to wind-induced motions in buildings due to human response are defined. The limiting motions in the buildings are defined as peak horizontal accelerations at the first natural frequency caused by a wind with a *one-year return period*. The limits of acceleration are specified for the main principal directions of the building and in torsion, where the main principal directions are generally along-wind and cross-wind.

The limits of peak acceleration are defined by two curves that are functions of the first eigenfrequency $f_0$ [Hz] in the building. The frequencies which the standard can be applied to are between 0.06 to 5.00 Hz. Figure 2.21 shows the two curves in a logarithmic scale. In the figure, curve 1 defines the limits for buildings that are used as office buildings. Curve 2 shows the limits for buildings used for residence. Residential buildings have lower maximum accelerations allowed than for offices. The allowed acceleration in residential buildings at the same frequency as offices are 2/3 in amplitude, this results in a curve which closely describes the 90 % level of the perception probability. Acceleration in torsion ISO10137 is defined as equivalent translational acceleration

$$A = r \cdot A_\theta(t) \qquad [m/s^2] \tag{2.46}$$

where
$r$ = distance from rotational centre to the objective point
$A_\theta(t)$ = angular acceleration of torsional vibration

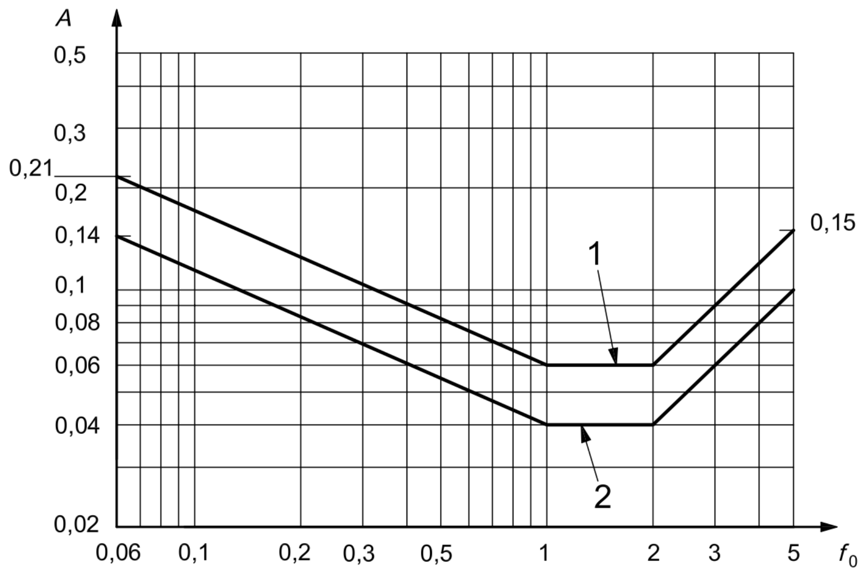



**Figure 2.21:** ISO 10137 peak acceleration limits as functions of natural frequencies for residential and office buildings.

### 2.9.2 ISO 6897 - Response of occupants to low frequency motion

ISO 6897 (ISO, 1984) recommends a highest value of the root-mean-square (rms) of acceleration. The acceleration rms is calculated for the response during the worst 10 minutes of a wind storm with a *return period of five years*. The standard can be applied to frequencies from 0.063 to 1.00 Hz and is therefore not always possible to combine with ISO 10137 because of its smaller range of natural frequencies. Figure 2.22 shows the two curves defined in ISO6897. Curve 1 limits rms acceleration in general purpose buildings. Curve 2 limits rms acceleration in offshore fixed structures.

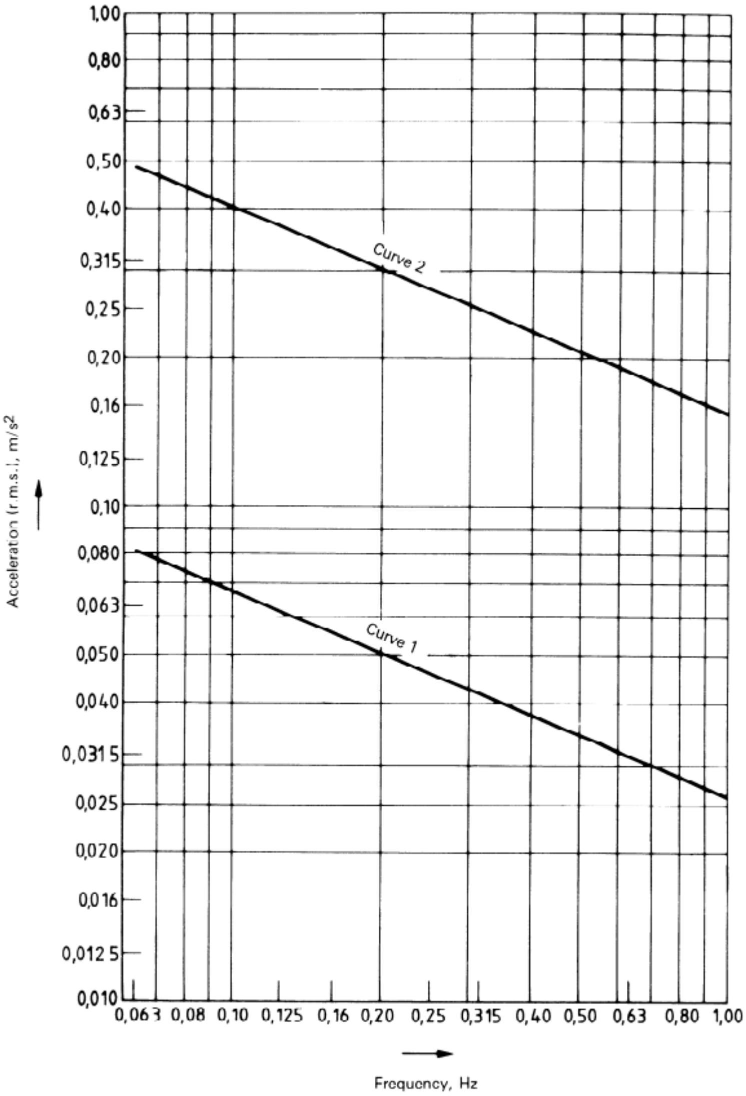



**Figure 2.22:** ISO 6897 r.m.s acceleration limits as functions of natural frequencies for general purpose and offshore buildings.

### 2.9.3 EKS12 and Eurocode 1 - Wind pressure and wind force

Swedish standard EKS12 (Boverket, 2022) and European standard EN 1991-1-4 (EC1)(Eurocode 1, 2005) describe a method of wind load calculations. In order to calculate wind actions on structures, multiple parameters need to be decided. The equation for peak wind pressure $q_p(z)$ is given in EKS12.

$$q_p(z) = [1 + 2 \cdot k_p \cdot I_v(z)] \cdot \left[k_r \cdot \ln\left(\frac{z}{z_0}\right) \cdot c_0(z)\right]^2 \cdot q_b \tag{2.47}$$

where
$k_p$ = peak factor
$I_v(z)$ = turbulence intensity factor at height z
$k_r$ = terrain factor (2.65)
$z_0$ = roughness length (Table 4.1 EC1)
$c_0(z)$ = topography factor according to Annex A.3

$$I_v(z) = \frac{1}{c_0(z) \cdot \ln\left(\frac{z}{z_0}\right)} \tag{2.48}$$

In case the building is static and the eigenfrequencies do not need to be considered, peak factor can be set to 3.0. In cases when the buildings eigenfrequency needs to be considered, such as in tall slender buildings, $k_p$ is calculated using EKS12

$$k_p = \sqrt{2\ln(\nu T)} + \frac{0.6}{\sqrt{2\ln(\nu T)}} \tag{2.49}$$

where
$T = 600\ s$ = averaging time for mean wind velocity
$\nu$ is the up-crossing frequency

$$\nu = f_1 \sqrt{\frac{R^2}{B^2 + R^2}} \tag{2.50}$$

Where the empirical formula in EuroCode for the first eigenfrequency is (2.51). When using the empirical formula for a timber building, results might not be reliable due to the formula being developed for concrete and steel buildings. An alternative is to perform FE-Analyses on the structure to determine natural frequencies (Edskär, 2018).

$$f_1 = \frac{46}{h} \qquad [Hz] \tag{2.51}$$

Wind pressure on external surfaces is often used for facade details and structural parts, given by

$$\mathrm{w}_e = q_p(z_e) \cdot c_{pe} \tag{2.52}$$

$z_e$ = reference height equal to the structure height above ground or building height
$c_{pe}$ = pressure coefficient for external pressure (tables in Section 7, EC1)

Wind forces on structures can be used for wind effects on the global structure

$$F_w = c_s c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref} \tag{2.53}$$

$c_s c_d$ = structural factor (size factor, dynamic factor) defined in EKS12
$c_f$ = force coefficient for the structure or element (Section 7)
$A_{ref}$ = reference area of the structure (Section 7)

The equation for $c_s c_d$ is given in EKS12

$$c_s c_d = \frac{1 + 2k_p I_v(z_s)\sqrt{B^2 + R^2}}{1 + 6I_v(z_s)} \tag{2.54}$$

where
$B^2$ is the background factor

$$B^2 = \exp\left[ -0.05\Big(\frac{h}{h_{ref}}\Big) + \Big(1 - \frac{b}{h}\Big) + \Big(0.04 + 0.01\frac{h}{h_{ref}}\Big)\right] \tag{2.55}$$

$R^2$ is the resonance response

$$R^2 = \frac{2\pi \ F \ \phi_b \phi_h}{\delta_s + \delta_a} \tag{2.56}$$

$\delta_a$ is the aerodynamic logarithmic damping. $\delta_s$ is the logarithmic structural damping

$$\delta_a = \frac{c_f \ \rho \ v_{\mathrm{m}}(z_s) \ b}{2 \ f_1 \ m_e} \tag{2.57}$$

$$\delta_s \approx 2\pi \cdot \xi_1 \tag{2.58}$$

$F$ is Kármán's wind energy spectrum

$$F = \frac{4 \ y_C}{\Big(1 + 70.8 \ y_C^2\Big)^{5/6}} \tag{2.59}$$

$y_C$ is the non-dimensional frequency

$$y_C = \frac{150 \ f_{1,\mathrm{x}}}{v_{\mathrm{m}}(h)} \tag{2.60}$$

$\phi_h$ is a size factor with respect to the height of the structure

$$\phi_h = \frac{1}{1 + \frac{2 \ f_{1,\mathrm{x}} h}{v_{\mathrm{m}}(h)}} \tag{2.61}$$

$\phi_b$ is a size factor with respec to the width of the structure

$$\phi_b = \frac{1}{1 + \frac{3.2 \ f_{1,\mathrm{x}} b}{v_{\mathrm{m}}(h)}} \tag{2.62}$$

$z_s$ = reference height for structural factor (Figure 6.1 EC1)
$h_{ref} = 10\ m$ (Boverket, 1997)
$\xi_1$ = damping ratio at first natural frequency
$f_{1,\mathrm{x}}$ = first eigenfrequency of the structure
$m_e$ = equivalent mass of the structure, calculated using (2.72)
$v_\mathrm{m}(z)$ = mean wind velocity at height $z$. Section 4.3 in EN 1991-1-4 defines $v_\mathrm{m}$ at height $z$

$$v_\mathrm{m}(z) = c_r(z) \cdot c_O(z) \cdot v_b \tag{2.63}$$

where
$c_O = 1.0$ = orography factor
$v_b = 25\ m/s$ = basic wind velocity (Göteborg).

$$\begin{cases} c_r(z) = k_r \cdot \ln\left(\frac{z}{z_0}\right) \quad \text{for} \quad z_{min} \leq z \leq z_{max} \\ c_r(z) = c_r(z_{min}) \quad \text{for} \quad z \leq z_{min} \end{cases} \tag{2.64}$$

$$k_r = 0.19\left(\frac{z_0}{z_{0,II}}\right)^{0.07} \tag{2.65}$$

where
$z_0$ = roughness length (Table 4.1 EC1)
$z_{min}$ = minimum height (Table 4.1 EC1)
$z_{max} = 200\ m$
$z_{0,II} = 0.05\ m$

### 2.9.4 EKS12 and Eurocode 1 - Acceleration in dynamic eigenfrequencies

For oscillations in the first eigenfrequency, the amplitude of acceleration at height $z$ of a constant mass cantilever is calculated using EKS12. Analysis of dynamic effects in the structure using the methods in EC1 and EKS12 is only applicable when the building is vibrating in its first eigenmode. For the effects of different eigenmodes, other methods should be used. The peak acceleration at height $z$ is calculated using

$$\ddot{X}_{max}(z) = k_p \sigma_{\ddot{X}}(z) \tag{2.66}$$

Where $\sigma_{\ddot{X}}(z)$ is the acceleration standard deviation (rms), calculated using

$$\sigma_{\ddot{X}}(z) = \frac{3\ I_v(h)\ R\ q_m(h)\ b\ c_f\ \phi_{1,\mathrm{x}}(z)}{m_e} \tag{2.67}$$

$q_m(h)$ = mean wind velocity pressure at height $h$,

$$q_m(h) = \frac{1}{2}\ \rho\ v_m(h) \tag{2.68}$$

where
$\rho$ = density of air

Mean wind velocity is based on the basic wind velocity with a return period of 50 years $v_b = v_{50}$. To determine the five year acceleration for ISO 6897, the five year velocity is instead used. In EKS12 this velocity is calculated using

$$v_{T_a} = 0.75\ v_{50}\sqrt{1 - 0.2\ \ln\left(-\ln\left(1 - \frac{1}{T_a}\right)\right)} \tag{2.69}$$

Where $T_a$ is the number of years $T_a = 5$ years for five-year wind. Alternatively, EKS12 specifies that for five years:

$$v_5 = 0.855\ v_{50} \tag{2.70}$$

The equation for $v_{T_a}$ does not work for $T_a = 1$ year. To calculate the acceleration return period of 1 year for ISO 10137, $T_a = 2$ years. As an alternative, ISO 6897 suggests directly assuming the 1 year acceleration rms from the result of 5 years acceleration

$$\sigma_{\ddot{X},1year} = 0.72\sigma_{\ddot{X},5years} \tag{2.71}$$

$m_e$ is the equivalent mass per unit height of the fundamental mode.

$$m_e = \frac{\int_0^l m(z)\ \phi_1^2(z)\ dz}{\int_0^l \phi_1^2(z)\ dz} \tag{2.72}$$

where
$m(z)$ = mass per unit height at height $z$
$\phi_1(z)$ = mode shape of the first eigenmode

The equivalent mass can be described as the modal mass divided by an integral of $\phi_1^2$ over the height of the building. The modal mass of the first eigenmode $\mu_1$ is a value which describes the structural system.

$$\mu_1 = \int_0^l m(z)\ \phi_1^2(z)\ dz \tag{2.73}$$

Equations (2.67) and (2.72) show that increasing modal mass results in a reduction of acceleration in the structure. It can be seen in (2.72) that due to the shape of the first eigenmode (2.74), increasing mass at height $z$ will result in a larger increase to modal mass at a higher height than at a lower height. The first mode shape, which has a higher value closer to $z = h$, will result in mass being multiplied with a higher value, thus having a higher influence on modal mass.

$\phi_1(z)$ is defined in EC1 as

$$\phi_1(z) = \left(\frac{z}{h}\right)^n \tag{2.74}$$

where $n$ [−], depends on the type of modal deflection. A lower value relates to a higher proportion of shear deflection while a higher $n$ value is associated with more deflection in bending. For a mode based on higher shear, relative acceleration in the lower floors will be higher compared to for a bending mode. EC1 suggests values between 0.6 to 2.5 depending on type of structural system. Figure 2.23 graphically shows different shapes depending the value of $n$. EKS12 recommends using $n = 1.5$ which according EC1 corresponds to slender cantilever buildings using reinforced concrete cores as stabilizing elements. For a high-rise timber building, a shear mode can be assumed. Therefore, recommended values of $n$ are between 0.6-1.0 (Edskär, 2018). It is also possible to do a modal analysis of the structure to determine a mode shape which is not defined in Eurocode.

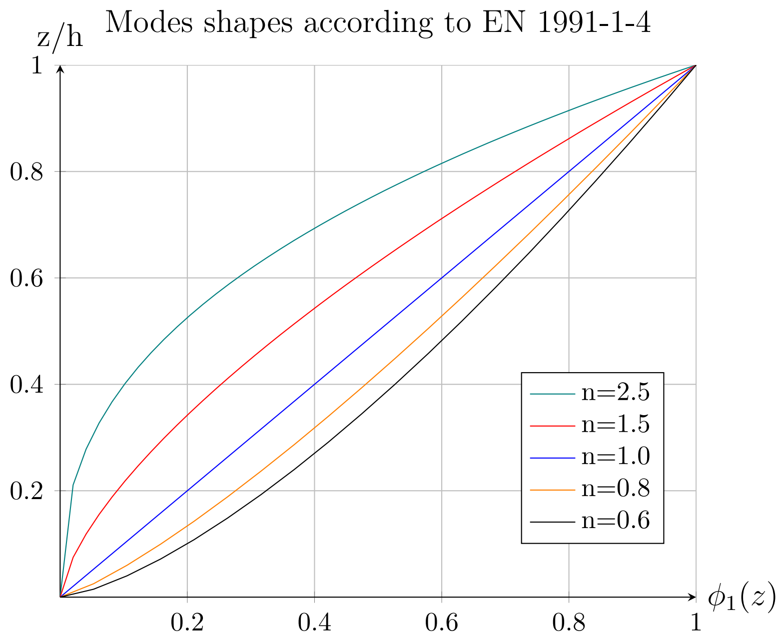



**Figure 2.23:** Five examples of mode shapes given by (2.74) for different values of $n$.

# 3
# Modelling

## 3.1 The building

The building's design is based on a typical Swedish residential point block building, with a footprint of 21.8x21.8m. The building is designed as a timber structure or a timber-hybrid structure that is built with a combination of timber and concrete structural elements. Each storey is 3 m high, which is a comparable storey height to Brock Commons Tallwood house. Layout of floors and walls are designed to be identical on every storey. The building's function is intended to be residential. Six different apartments are planned on each floor. Beside the apartment rooms, there is one shaft for elevators, stairways and installations. The floor plan with its chosen global coordinate system is shown in Figure 3.1.

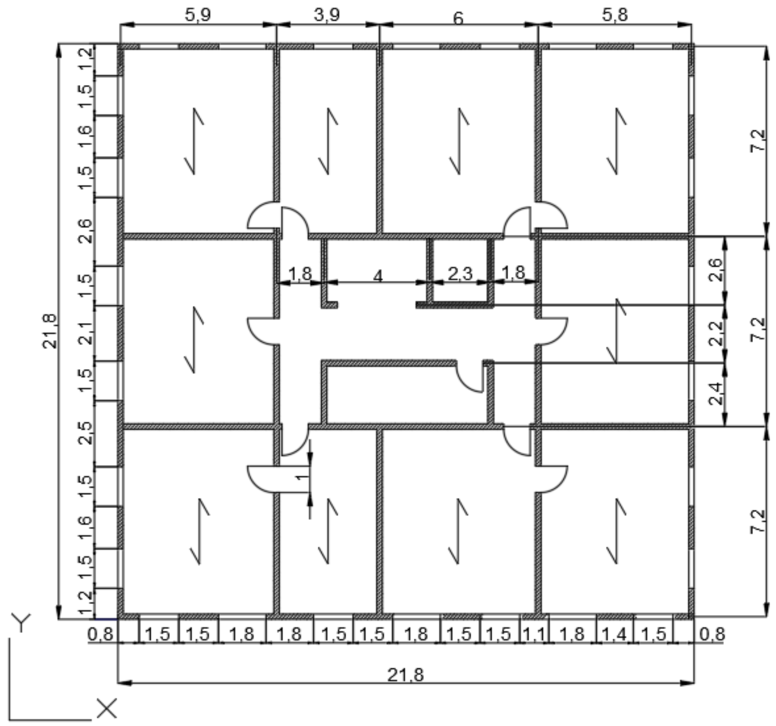



**Figure 3.1:** Floor plan of the building with dimensions [m] and x,y coordinate system.

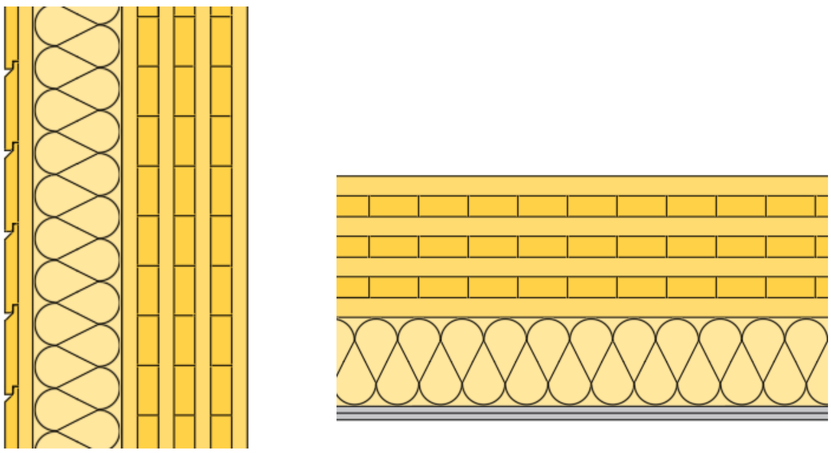
The building's location in Gothenburg corresponds to terrain category III and a basic wind speed of 25m/s acting on the building.

The structural system of the building is a panel system which utilizes shear walls connected to a core through diaphragm floors. The walls around shafts are located in the center of the building and work as a structural core, see Figure 3.3 (a). A concrete slab acts as foundation for the building and is assumed to be stiff enough to act as rigid supports.

Shear walls in the structural system are grouped into three categories: Outer walls, Inner walls and core walls. Where the inner walls are defined as all the walls inside the building that is not a part of the core walls. The outer and inner walls are CLT panels carrying both horizontal and vertical loads. The core walls surround the elevator and staircase shafts, they are initially designed using CLT panels but are also intended to be replaced with concrete walls in the analysis. The floors cover the whole footprint of the building, except for the staircase and elevator shafts. They can be either designed as CLT panels or reinforced concrete slabs in the analysis.

### 3.1.1 Preliminary sizing

The timber materials used in the design are based on products produced by the manufacturer Martinsons with mean stiffness values. The load bearing walls are designed to have a thickness of 240mm and consist of 7 layers see Appendix A.3 for stiffness. The internal walls are not covered with any material. The external walls have an external cover of mineral wool insulation and facade cladding which is illustrated in Figure 3.2 (a). The floor structure is assumed to have a thickness of approximately 500 mm. The floor consists of a CLT-slab of 7 layers with a thickness of 270mm. The slab is decided on standard products from Martinsons, see AppendixA.3 and is based on the maximum span length of 7.2 m in the floor plan, shown in Figure 3.1. The slab is also supplemented with an insulated ceiling to manage the sound demands of apartment dividing floors, figure 3.2 (b) illustrates an example of the floor.



**(a)** **(b)**

**Figure 3.2:** Illustrated designs of (a) External wall (b) Ceiling.

**Table 3.1:** Included materials in the external CLT wall and ceiling. Weights of the CLT panels are excluded as they are later included directly in the modelling process.

| External wall | | |
|---|---|---|
| Material | Thickness [mm] | Weight [kg/m2] |
| Facade panel | 25 | 11.0 |
| 28x95 battens | 28 | 3.9 |
| 45x170 studs + insulation | 170 | 12.4 |
| 7 layer CLT Panel | 240 | - |
| SUM: | 463 mm | 27.3 kg |
| Ceiling | | |
| 2x gypsum board | 26 | 18.7 |
| 45x170 studs + insulation | 170 | 12.4 |
| 7 layer CLT Panel | 270 | - |
| SUM: | 466 mm | 31.1 kg |

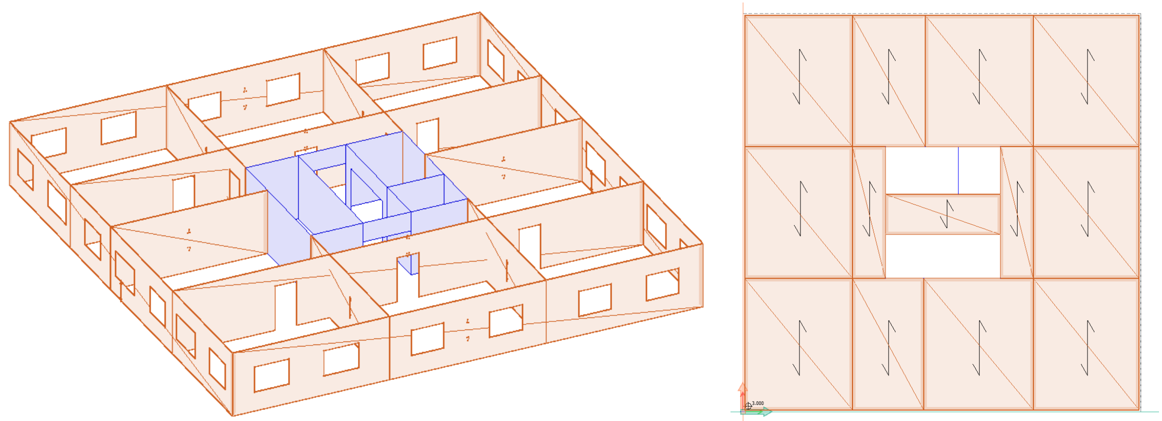
A strength class of C30/37 is chosen for the structural concrete in the building. The stiffness of concrete members is based on the strength class in the material. Influence of reinforcement in concrete members is neglected since cracking is assumed to be negligible in the material. The wall thickness in the concrete core is 200 mm as an initial assumption based on existing structures. Preliminary sizing of a two-way simply supported slab with $l_{span} = 7.2$ m, thickness can be designed to be $l_{span}/30 = 0.24\ m$ for characteristic imposed loading of 5 $kN/m^2$ (ISE, 1985). Therefore concrete slab thickness is initially designed to be 240 mm.

### 3.1.2 Actions on structure

The vertical loads that act on the building is the self weight of the structural material, in addition to that surface loads from the facade cladding , ceiling and live load are included to get a more realistic loading situation in the service state of the building. For the facade cladding and ceiling an assumed weight of 30 kg/m$^2$ is estimated, when excluding the load bearing CLT panels. The acting live load of 0.6kN/m$^2$ is assumed this is 30% of load for residential buildings according to EC1, this is equal to the load for the quasi permanent load.

## 3.2 FE-model

The building is modelled as an FE model using FEM Design. The FE model is designed to be a structurally accurate approximation of the designed building to get more realistic results. CLT panels are modelled using a continuous anisotropic material model. Holes for doors and windows are cut out of the model, which can be seen in Figure 4.15 (a). The main direction of wall CLT panels is in the vertical direction due to the walls functioning as vertical load bearing. For concrete walls, there is no main direction due to the slab being assumed to be uncracked. The CLT panel floors are oriented with the main direction in the global y-direction. This is shown in Figure 3.3 (b). Similarly to concrete walls, floors in concrete have no main direction.



**(a)** **(b)**

**Figure 3.3:** (a) One storey of the FE model. The blue walls are core walls. The orange walls are the surrounding CLT shear walls. (b) The main direction of CLT floors

The storeys in the FEM model are geometrically identical copies. Figure 3.4 shows a 12 storey high model of the building in FEM-Design.

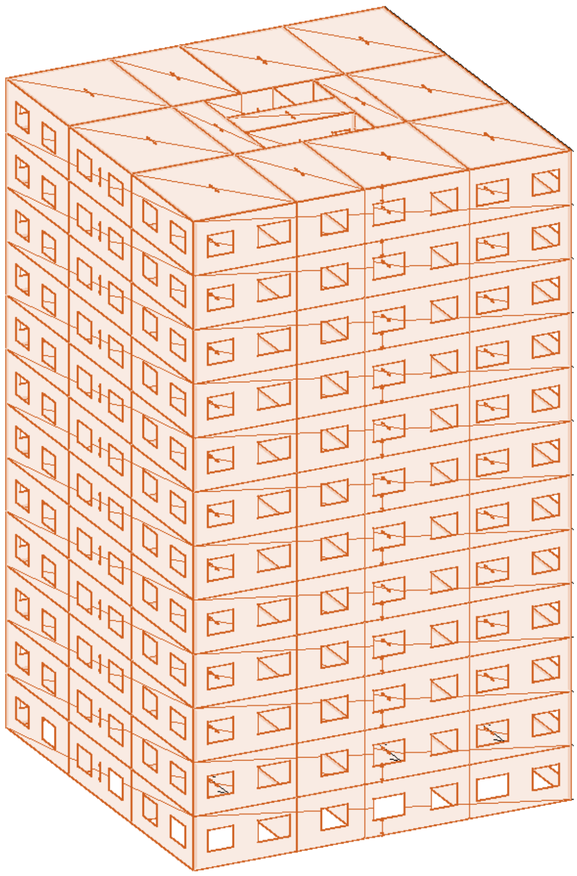


**Figure 3.4:** FE model seen from the side. 12 storeys tall.

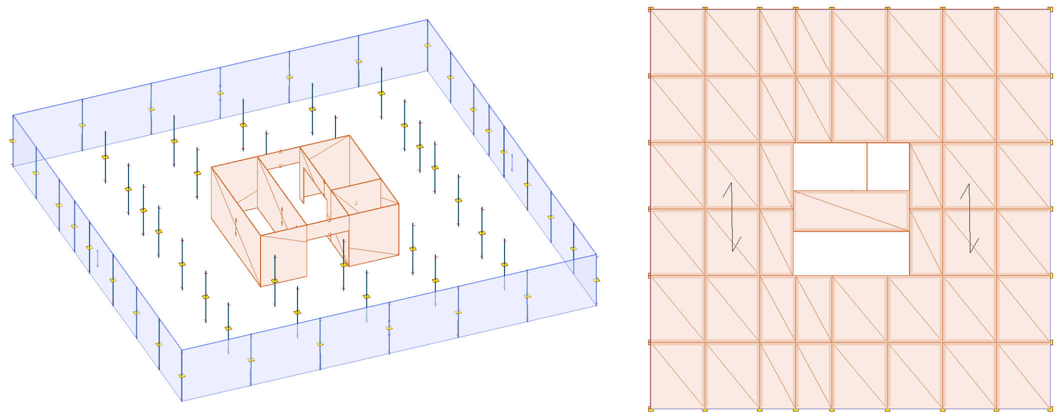
Connections between structural walls elements are modelled as fixed connections, not allowing for any displacement or rotation between the members in the connection points. The walls can be assumed to be fixed by the pressure of the buildings self-weight acting on the walls. Connections between floors and walls are modelled as hinged connections, assuming the floor panels are simply supported on four sides. Timber walls and floors are modelled as continuous over their full width and any connections between each panel along its width is disregarded. The bottom storey walls are supported by continuous line supports acting as fixed connections. The FE model is constructed using a combination of 2D triangular elements with 6 nodes, and four sided elements with 9 nodes. The vertical loads listed in Chapter 3.1.2 are added as permanent and long term loads in FEM Design. The loads are converted to mass for dynamic modelling using the FEM Design mass conversion tool.

**Table 3.2:** Data used for initial FE-modelling in FEM-Design.

| Assumptions in FEM analysis | |
|---|---|
| Element type | 9/6-node surface elements |
| $\rho_{concrete}$ | 2550 kg/m$^3$ |
| $\rho_{CLT}$ | 400 kg/m$^3$ |
| $t_{CLT,floor}$ | 270 mm (7 layers) |
| $t_{CLT,wall}$ | 240 mm (7 layers) |
| $t_{concrete,floor}$ | 240 mm |
| $t_{concrete,wall}$ | 200 mm |
| $q_{ceiling}$ | 0.30 kN/m$^2$ |
| $q_{facade}$ | 0.30 kN/m$^2$ |
| $q_{live,load}$ | 0.60 kN/m$^2$ |
| $h_{storey}$ | 3 m |

## 3.3 Alternative structural system

An alternative model is created with the purpose of comparing with the reference building. The alternative model is made using a column-panel structural system in timber. The modelling of the column building is based on the structural system of Brocks Commons tallwood house but modified to match the footprint and core of the reference building to allow for comparison between the systems. The column-panel building also has the alternative of replacing timber elements in the core and floors with concrete, and has the possibility of adding diagonal bracing to the structural system.



**(a)** **(b)**

**Figure 3.5:** (a) One storey of the column FE model. Surrounding blue panels are cover elements used to distribute facade loads (b) The main direction of CLT floors

Glulam columns are placed in a grid that allows columns to be placed where walls are built in the original floor plan. The grid is based on the brocks commons grid, with slightly smaller spans and slightly larger floor panel widths. The glulam columns are modelled as 260x260 mm GL30c columns, similar to Brocks commons 265x265 columns (Naturally:wood, 2016). The columns are modelled as 1D structural elements with the length on one storey height (3 m). The edge connections of the columns are hinged connections. The column supports to the foundation are hinged supports.

The column grid creates a main direction span width of 3.6 m in the building. Martinsons (Span Appendix) allows using 5 layer 160 mm CLT panels as floor elements. This is a similar dimension to brocks commons which uses 169 mm elements for 4.0 m spans. The maximum width of a panel is designed to be 3.0 m to allow for panels to be produced using standard production methods. In cases where diagonal bracing is used, the bracing elements are modelled as 1D truss elements with glulam material GL30c. The bracing cross section depends on the analysis. Distributed loads acting on the model are the same loads as for the reference building. An additional wall load has been added to account for the weight of non load-bearing walls, which is based on the wall lengths of the reference floor plan. The weight is calculated to 0.27 kN/m$^2$. FEM-Design cover elements are used to distribute facade loads to the closest floors.

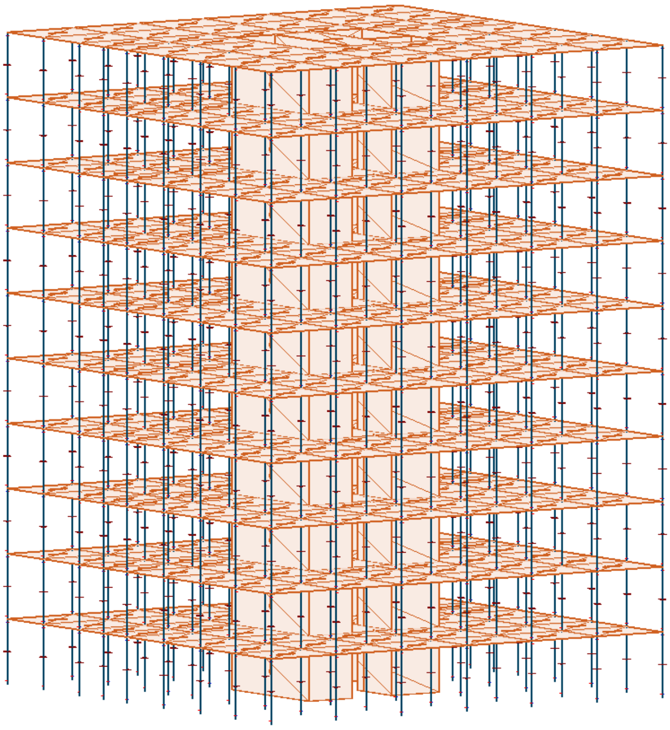


**Figure 3.6:** FE model of the column building seen from the side, without showing cover elements. 10 storeys tall.

A benefit of the column system over the CLT panel system is its suitability for an office building. Another benefit is that it requires less timber in the structural system than the CLT panel system.

## 3.4 FE analysis and calculation of accelerations

The process of studying a buildings dynamic properties for comparison with ISO 10137 and ISO 6897 begins with a FEM analysis. This process is described in Figure 3.7. The FEM analysis begins with a 2D Drawing of the building's floor plan, and the preliminary sizing of structural elements in the building. These elements are then added as walls, floors, columns and truss elements in the FEM-Design model. Material choices are done for the structural parts, where it is decided on where concrete should be included in the building. Loads and supports are added to the model and result points are included on each floor which are used to find deflections for the mode shape. When the model is finished it is divided into suitably sized 2D shell elements. When the element mesh is finished the building is analyzed. The analysis in FEM-Design is treated as a structural dynamics eigenvalue analysis for the relevant eigenmodes. The results which are gathered from a FEM analysis are masses lumped into each floor, the lowest relevant eigenfrequencies, and the relative displacements in the global coordinate system. During the study of a building, the materials, dimensions and building height can be changed in between analyses.

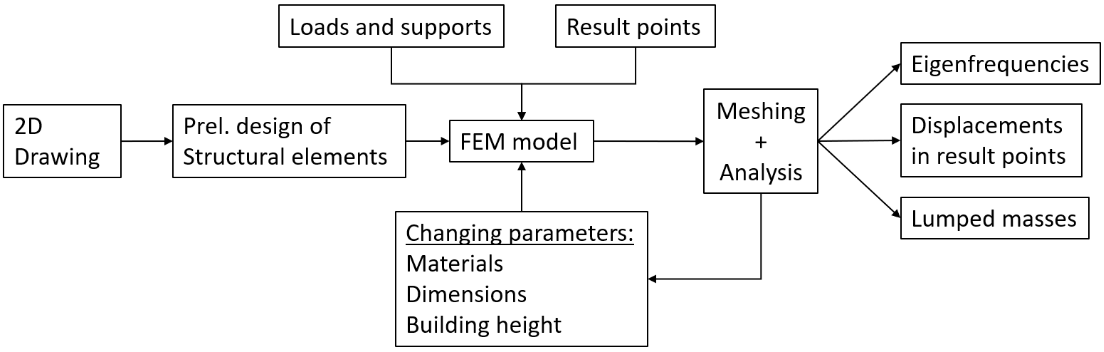



**Figure 3.7:** Flowchart of iterative FEM analysis process.

Hand calculations are done to determine the acceleration response in the building based on the FEM results. The method of deciding accelerations in the building is based on methods in EKS12 and EuroCode 1-4 which is explained in Chapter 2.9.4.

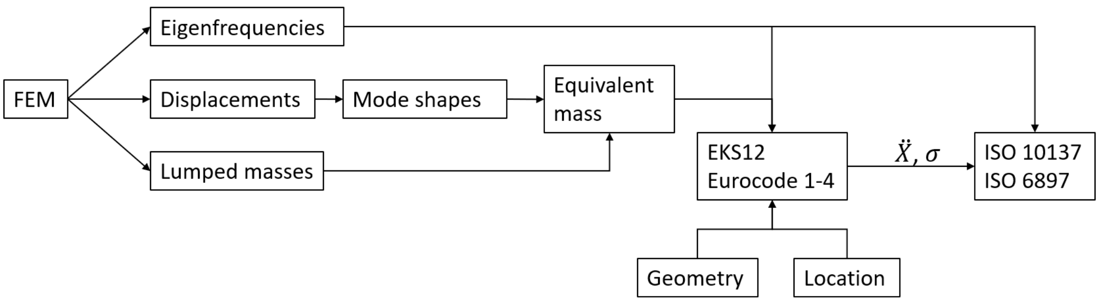



**Figure 3.8:** Flowchart of acceleration calculation based on FEM results.

Displacements in secondary directions are neglected to in order to use the calculation methods described in EKS and Eurocode. The damping coefficient in the structure is assumed to be $\xi_1 = 1.5\%$, which is a recommendation (Boverket, 1997) for a timber structure with mechanical connections. The height, $z$ at which the acceleration is calculated is at $z = h - h_{storey}$ which is the highest height where a residential floor will be built

The equivalent mass per unit height of the building is calculated using lumped mass and displacement results from the FEM analysis. The integration in (2.72) is done using numeric integration where $\phi_1$ is the curve of normalized displacements on every floor and $m(z)$ is the lumped mass at each floor. An approximation of $m_e[kg/storey]$ is given by the numerical integration. Because the values are summed at each floor which are separated by a storey height the value is divided by $h_{storey}$ to get the equivalent mass per meter $m_e[kg/m]$

**Table 3.3:** Assumptions made for hand calculations of acceleration response in buildings.

| EKS Calculation assumptions | |
|---|---|
| Terrain category | III |
| $\xi_1$ | 1.5% |
| $z$ | $h - 3$ m |
| $v_{50}$ | 25 m/s |
| $\sigma_{x,1year}$ | 0.72 $\times \sigma_{x,5years}$ |

### 3.4.1 Convergence study

A convergence study is done on a 10 storey model of the CLT building intended to be analyzed. The convergence study is made to find an FE mesh that gives accurate results with a short calculation time. Using the same model, different element sizes are used for meshing in FEM-Design. For each mesh, an analysis is run where the building's first twelve modes are analyzed. The calculation time for the analysis in FEM-Design is measured. The first eigenfrequency is also taken from the FEM-Design results as it is an important dynamic property which is easy to compare. The result are used to calculate acceleration in the building with different element sizes by using hand calculations.

**Table 3.4:** Convergence study with different element sizes.

| Convergence study | | | | | | |
|---|---|---|---|---|---|---|
| Element size [m] | 2 | 1 | 0.8 | 0.6 | 0.5 | 0.25 |
| Calculation time [min:s] | 00:23 | 00:27 | 00:37 | 01.26 | 01:46 | 08:44 |
| $m_e$ [kg/m] | 47719 | 47658 | 47653 | 47627 | 47624 | 47599 |
| $f_1$ [Hz] | 3.105 | 3.055 | 3.043 | 3.031 | 3.020 | 3.004 |
| $\ddot{X}_{max}$ $[m/s^2]$ | 0.0237 | 0.0242 | 0.0243 | 0.0244 | 0.0245 | 0.0247 |

Table 3.4 shows results from the convergence study where calculation time, equivalent mass, natural frequency and max acceleration. The results show an increase in calculation time with decreasing element size. Equivalent mass and frequency are slightly reduced with smaller element sizes. Acceleration is slightly increased with smaller element sizes. An element size of 0.8 m is sufficiently accurate for the analysis. The difference in results are not large enough to make up for the higher calculation time.

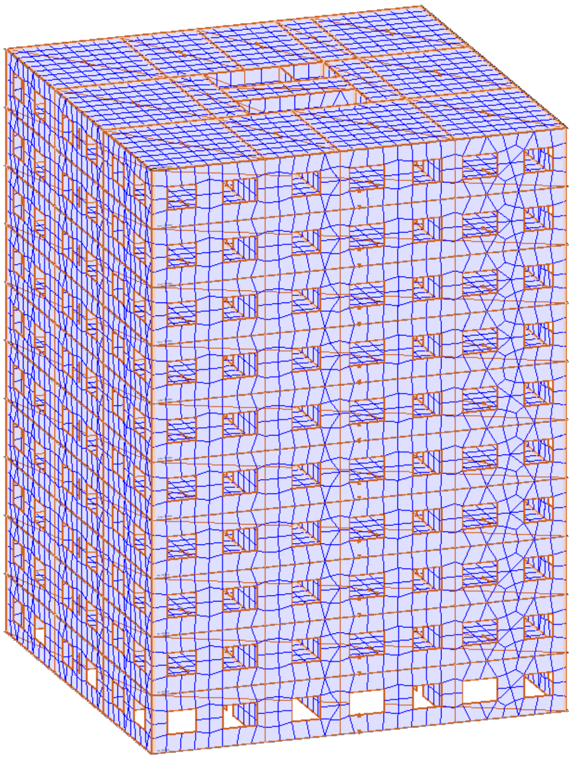


**Figure 3.9:** Meshed 10-storey FE model with an average element size of 0.8 m used for convergence study.

### 3.4.2 Verification of FE-model

A verification of the FE-model's dynamic and static response is done by comparing results of FEM-analysis with simplified hand calculations. The verification is done on the 10 storey CLT building model. The method of verification is done by using a simplified approximation of the building as a one-dimensional undamped mass-spring system, as explained in Chapter 2.7.1. The system is divided into 10 horizontal movement dofs, one on each floor above the ground floor and one on the ceiling. The mass matrix is produced by lumping the weight of one floor to each dof. The mass from facade weight, ceiling weight, and live load are lumped to the closest dof.

The stiffness between each floor is approximated using the model in Chapter 2.4. Equations (2.5) (2.4) and (2.2) are used for the horizontal bracing stiffness of the CLT walls acting in the x-direction. Walls orthogonal to the x-direction are neglected in the stiffness calculation since they are assumed to add no stiffness in their weak direction. Wherever there is a hole for a window or door, the wall is assumed to provide no stiffness and is neglected. Figure 3.10 shows the walls that are included in the horizontal stiffness calculation of one storey.

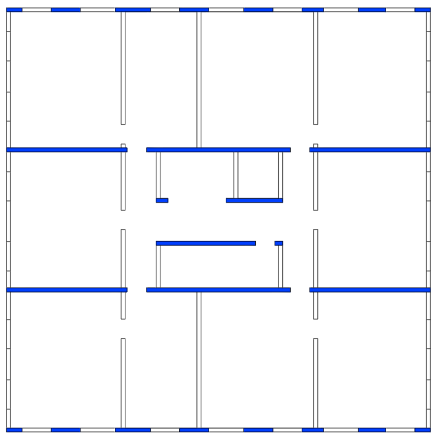


**Figure 3.10:** Floor plan showing structural walls with stiffness in x-direction used for a simplified hand calculation.

In CLT panels, displacements due to movements in joints are neglected since joint stiffness is not modelled in the FE-model. Stiffnesses of each wall is summed into a single simplified stiffness value of one floor: $\Sigma S_i = k\ [\frac{N}{m}]$. Material data used for stiffness calculations are mean stiffness values given in Martinsons tables (Reference to Appendix). The stiffness matrix is assembled and combined with the system's mass matrix to solve eigenvalue problem (2.20).

Table 3.5 shows the resulting two lowest eigenfrequencies in the FE-model and mass-spring model. Figure 3.11 shows the resulting mode shapes of the two different models. The mode shape of the FEM model is similar in shape to the mass-spring model and both results are relatively similar to mode shapes in Eurocode 1-4 (2.74). The results suggest that $n$ should be somewhere between 0.6 and 0.8 to best fit the

10 storey model.

**Table 3.5:** Comparison of two lowest eigenfrequencies for verification of the FE model using hand calculations.

| | mass-spring model | FE-model | Difference |
|---|---|---|---|
| $f_1$ [Hz] | 3.43 | 3.12 | -9.9% |
| $f_2$ [Hz] | 10.24 | 9.13 | -12.2% |

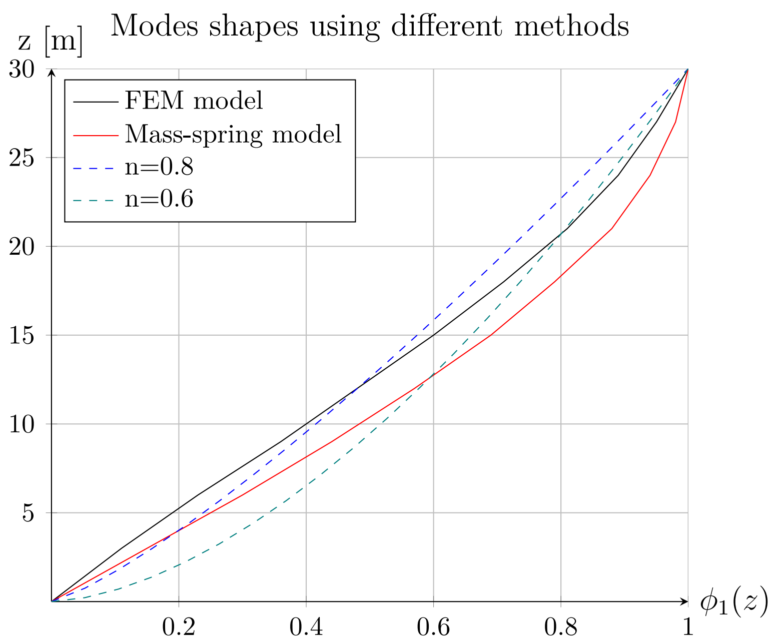



**Figure 3.11:** Mode shapes for an FE model compared to a mass-spring model and Eurocode.

A verification of static vertical reaction forces in the bottom supports is done by comparing hand calculation and FE-model results. The 10 storey reference building is used for the verification. Loads applied on the building are the loads listed in Chapter 3.1.2 The verification shows an acceptable difference of 0.6% between the hand calculations and FEM model which can be seen in table 3.6.

**Table 3.6:** Comparison of the reaction forces at the line supports to verify the FEM model using hand calculations.

| | Hand calc. | FEM | Difference |
|---|---|---|---|
| Reaction force [kN] | 14740.3 | 14653.0 | -0.6% |

# 4

# Analysis

In most analyses, the building is being designed as an apartment building, where limits of dynamic movements given in ISO 10137 and ISO 6897 are applied for residential buildings. When the building is being designed for commercial purposes, the limits for office buildings are applied instead. In cases where the natural frequency of the building is above 1 Hz, ISO 6897 provides no threshold for r.m.s. acceleration and is therefore not used to evaluate the results.

## 4.1 Concrete volume

One of the main criteria when optimising the high rise timber-hybrid building is to reduce the volume of concrete and allow for more timber in the structural system. Reduction of concrete volume in the building is important due to marketing, added value and environmental considerations.

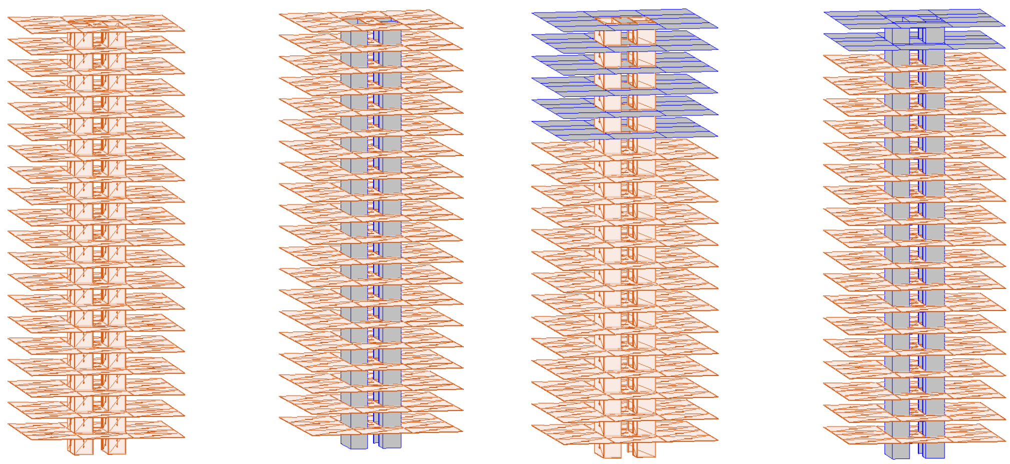
The environmental impact gained by utilizing timber instead of concrete is an aspect that should be considered for every building project. Building with concrete results in higher emissions than timber, and minimizing the use of concrete therefore leads to more sustainable production. Another reason to use as little concrete as possible is for marketing of the building as a high rise timber building. At the moment, a tall timber building attracts attention in the architectural and building sectors. If an unnecessary amount of materials such as concrete and steel are used, it could negatively affect the industry interest in the building. The tenant of an apartment or office in a high rise timber building may have an interest in making an environmentally conscious purchase. Therefore, a potential added value in the building can be expected when the building can be promoted as a timber-hybrid building. In this report the total concrete volume per square metres of rentable area in the building [$m^3/m^2$] is used as a unit to compare material efficiency buildings with different heights.

## 4.2 CLT building types

The CLT timber hybrid building is categorized into four types: T,C,F,CF. The categorisation is done so that it is clear which types of building is being discussed, as there are many different parameters involved in the analysis. The types of CLT building are:

- CLT-T: The initial building, modelled using only CLT timber elements.
- CLT-C: Initial model where a number of CLT core walls are replaced with reinforced concrete core walls from the bottom up.
- CLT-F: Initial model where a number of CLT floors are replaced with reinforced concrete floors from the top down.
- CLT-CF: Initial model which includes both concrete core walls and concrete floor elements.

Some parameters are not included in building types such as building height, element thicknesses and number of concrete floors. Numbers are added behind the building type in cases where it is relevant in the format (ss,cc/tc,ff/tf), where ss is the total number of storeys in the building, cc/tc is the number of concrete cores and the core thickness and ff/tf is the number of concrete floors and the floor thickness. For example, CLT-CF(14,14/200,4/280) is a 14 storey tall building with 14 core walls of 200 mm concrete and the top 4 floors are replaced with 280 mm concrete floors.

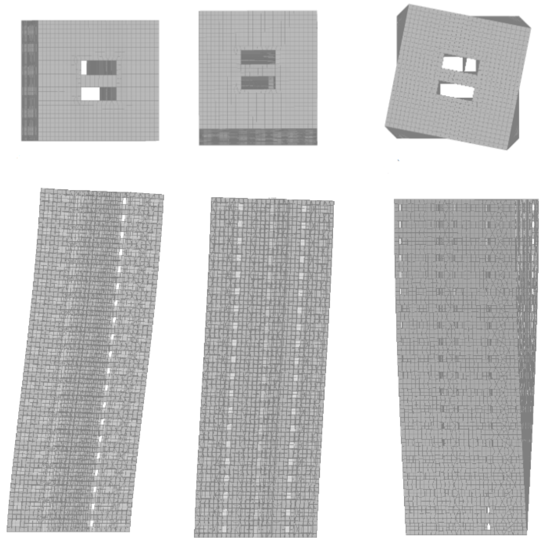


**(a)** **(b)** **(c)** **(d)**

**Figure 4.1:** Examples of the 4 different CLT building types: (a) CLT-T, (b) CLT-C, (c) CLT-F, (d) CLT-CF.

## 4.3 First 3 mode shapes

Figure 4.2 shows the first three dynamic displacement mode shapes in a 20 storey CLT-T building. The first mode is mainly translational displacement in the x-direction while the second mode is displacements in the y-directions. The third mode shows a torsional mode where the displacement occurs as a rotation around the central core. In the first three eigenmodes, displacements are zero at the ground level and continuously increase to the max value at the building's roof. It can be concluded from the first mode being in the x-direction that the stiffness of the building is the lowest in that direction.



(a) (b) (c)

**Figure 4.2:** (a) First mode shape (b) Second mode shape (c) Third mode shape. Top image shows view from the top. Bottom image shows view from the side.

## 4.4 Dynamic response in CLT-T building

The dynamic response is studied on a CLT-T building. The study is carried out on the building with varying numbers of storeys from 10 to 20 in steps of 2. Figure 4.3 shows results from the analysis, where the response of the building is plotted on a logarithmic scale together with the curve of peak acceleration threshold for residential buildings in ISO 10137. The results show that increasing the number of storeys leads to a lower natural frequency and an increase in acceleration. The figure also shows that the CLT-T building manages the acceleration limits up to a height of 14 storeys, showing that the tallest CLT-T building that can be built is 14 storeys high (42 m).

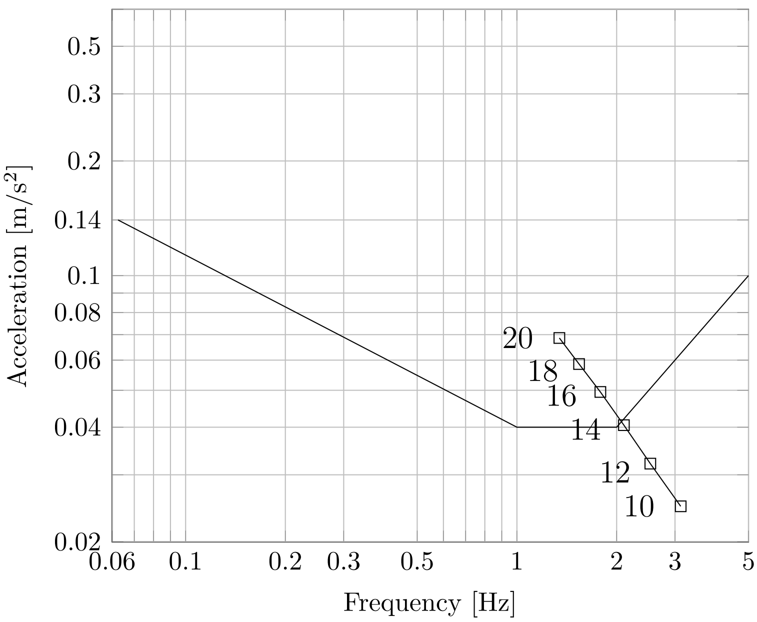



**Figure 4.3:** Dynamic response in the CLT-T building. Next to the marks, the number of storeys are shown.

## 4.5 Effect of modelling choices on results

Many parameters that depend on modelling assumption influence the results in the dynamic FEM-analysis. Parameters that are examined in this report are:

- Connections and support types
- Local effects of reduced stiffness over openings
- Added stiffness of structure by replacing CLT core walls with reinforced concrete core walls.
- Added mass of the structure by replacing CLT floors with reinforced concrete.
- Added damping in the structure by including a tuned mass damper.

## 4.6 Connection and support types

Different modelling parameters are used to analyze the influence of changing conditions at the ends of panels. Connections between panels and supports at the foundation are changed from fixed to hinged. This allows for free rotation between structural elements in the building as a result of connections being less rotationally stiff than expected. The continuous CLT panels in the building's walls are separated into multiple smaller panels with hinged connections, illustrated in Figure 4.4. The intention is to model the division of walls into panel elements similar to how CLT walls are assembled in buildings. The panels are also given gaps of 3 mm by using a built in material model in FEM-Design, with the intention of modelling possible imperfections in the structural system. Results from the analyses are presented in Table 4.1. The comparison shows a negligible difference in results between the different parameters in the model. This indicates that the connection types of the panels has a low influence in the dynamic response of the building. Therefore, the original connection assumptions can be used without significant impact on results.

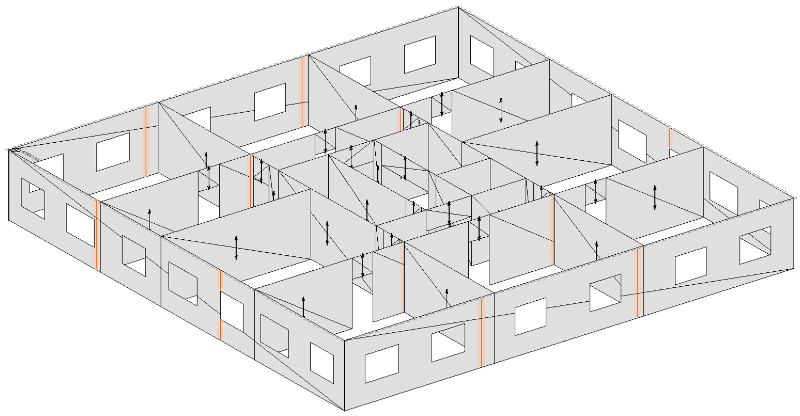


**Figure 4.4:** Division of CLT-panels are shown with vertical orange lines.

**Table 4.1:** Acceleration and natural frequency depending of element edge parameters.

| | Reference | Hinged connections | Divided panels | Div. panels + 3mm gap |
|---|---|---|---|---|
| $f_1[Hz]$ | 2.113 | 2.110 | 2.110 | 2.105 |
| $\ddot{X}_{max}$ | 0.0411 | 0.0412 | 0.0412 | 0.0413 |
| Diff. ($\ddot{X}_{max}$) | | 0.2% | 0.2% | 0.3% |

## 4.6.1 Local geometric effects above openings

It is possible that local effects in CLT panels lead to a reduction in stiffness over openings for doors. Reduced influence of material in FEM model due to sections being cut out or local stiffness lost due to connection are modelled in FEM-Design to understand their effect on results. A dynamic finite element analysis of accelerations on the 14 storey CLT-T is carried out by changing the modulus of elasticity in the panels over the doors in the inner walls which is illustrated in Figure 4.5. The analysis is carried out by reducing the stiffness in steps which can be seen in table 4.2. It can be seen in the table that the eigenfrequency is reduced due to lower stiffness of the panels and also that the acceleration is increased. By reducing the stiffness from 100% to 0% the acceleration of the building is increased by 19%. Because it cannot be known beforehand if the stiffness is lower in a real building, the reduction is chosen not to be accounted for in the analyzed model, meaning that stiffness is set to 100% of the material stiffness. It should still be considered that some local effects may produce less favourable dynamic results than is shown in a FEM analysis and depends on the conditions in a real world application.

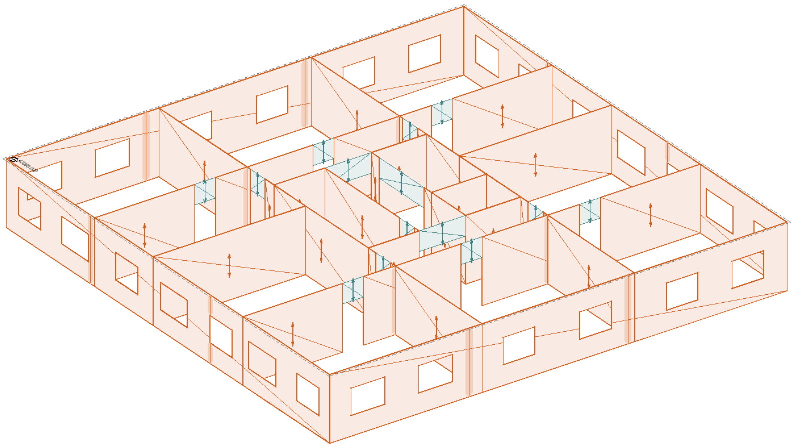


**Figure 4.5:** Panels over openings where the stiffness is changed are marked in green.

**Table 4.2:** Acceleration response depended on the stiffness of the panels over doors.

| CLT panel stiffness | | | | | | | |
|---|---|---|---|---|---|---|---|
| CLT stiffness [%] | 0 | 10 | 30 | 50 | 70 | 90 | 100 |
| $f_1$ | 1.83 | 1.91 | 2.00 | 2.05 | 2.08 | 2.10 | 2.12 |
| $m_e$ | 46101 | 46101 | 46101 | 46101 | 46101 | 46101 | 46101 |
| $\Phi$ | 0.9623 | 0.9623 | 0.9623 | 0.9623 | 0.9623 | 0.9623 | 0.9623 |
| $\ddot{X}_{max}$ $[m/s^2]$ | 0.0487 | 0.0463 | 0.0439 | 0.0426 | 0.0418 | 0.0413 | 0.0409 |
| Difference | 19.0% | 13.2% | 7.2% | 4.2% | 2.3% | 1.0% | |

## 4.7 Effect of adding concrete core and concrete floors

The effect of replacing CLT panel elements with reinforced concrete to improve dynamic response is studied. To understand the effect of replacing CLT-panels with a heavier and stiffer material, FEM studies are done where the reference model is edited such that timber elements in the core walls and floors are replaced with reinforced concrete. Addition of a concrete core (CCore) is primarily intended to increase the stiffness in the system. In theory, increasing stiffness has the effect of reducing accelerations in the building. Another potential benefit is shifting the building's natural frequency to a higher value. If the natural frequency is increased to a value higher than 2.0 Hz, ISO 10137 allows for higher accelerations in the structure. Another expected effect of replacing CLT core walls with concrete core walls is an increased modal mass. Addition of concrete floors (CFloor) is intended to have the main effect of added modal mass in the system. In theory, increasing modal mass in a system has the effect of lowering accelerations. It is also expected to lower eigenfrequencies, which has a positive effect when the natural frequency is reduced to below 1.0 Hz where the standards allow for higher accelerations in the structure.

### 4.7.1 200 mm concrete core

The initial analysis is done on a CLT-C building where the CLT core is replaced with a 200 mm concrete core of class C30/37 through the full height of the building. The analysis is carried out for the height of 10 to 20 storeys in step of 2. Results of the study show that it is possible to build a CLT-C building up to a height of 18 storeys without exceeding the maximum allowed acceleration according to ISO 10137. The acceleration in the building at that height is just at the limit of the ISO 10137 with an acceleration of 0.04 m/s$^2$ which can be seen in figure 4.6. Compared to the CLT-T building, results have been shifted to higher natural frequencies, which is expected when adding stiffness to the system.

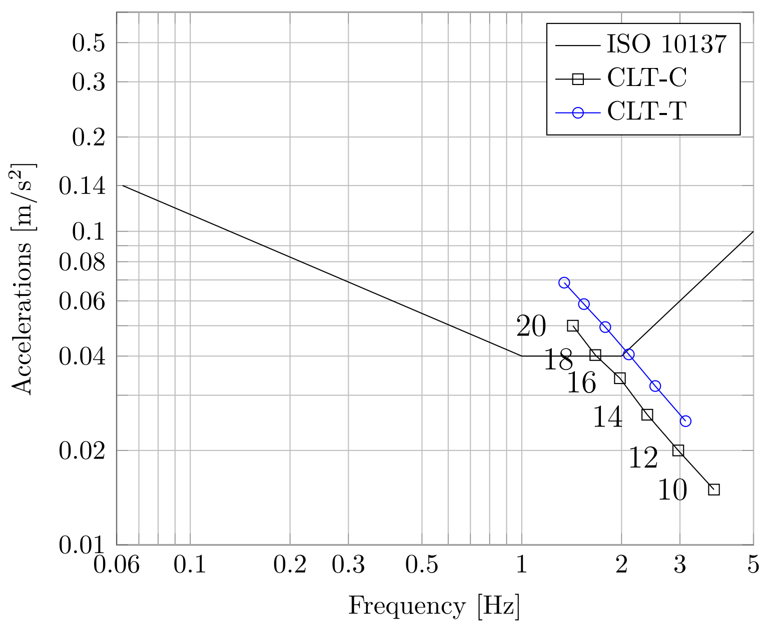



**Figure 4.6:** Dynamic response of a CLT-C building with 200 mm concrete core walls throughout the full height of the building.

### 4.7.2 240mm Concrete floors

According to Eurocode (Eurocode 1, 2005), the most effective areas to add mass in a building for dynamic response is at the top 1/3 of the building's height. In an analysis of a CLT-F building, 240 mm concrete slabs replace CLT panels in the top 1/3 floors. Figure 4.7 shows that the concrete floors in the CLT-F building improves the dynamic response of the building so that 16 floors can be built before the threshold of ISO 10137 is reached. Results compared to a CLT-T building have been shifted towards lower natural frequencies, which is expected when adding mass to the system.

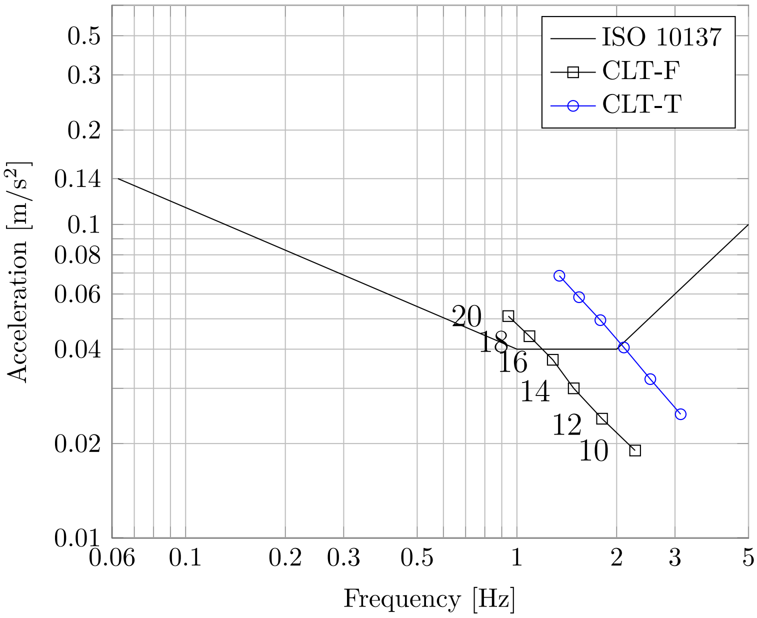



**Figure 4.7:** Dynamic response of a CLT-F with 240 mm concrete floors in the top 1/3 of the building.

### 4.7.3 Concrete floor thickness optimization

A study is done on the CLT-F building where different concrete floor thicknesses are tried in order to find how the floor thickness affects concrete volume in the building. The results of the study for building heights of 16, 18 and 20 storeys are presented in Figure 4.8. For each concrete floor thickness, the lowest number of concrete floors required to pass the requirements of ISO 10137 are found. The number of concrete floors needed are shown next to the points in the figure. The resulting concrete volume in the building is plotted against the floor thickness. Results show that it is possible to achieve more concrete efficient versions of CLT-F at building heights of 18 and 20 storeys. For a 16 storey CLT-F building however, there is very little difference in concrete volume when changing floor thickness. For the taller buildings, the concrete volume in the building reduces as the floor thickness increases, as a result of less floors being needed in the building to pass the peak acceleration requirements. At a certain floor thickness, the concrete volume in the building no longer reduces significantly as the concrete floor thickness is increased.

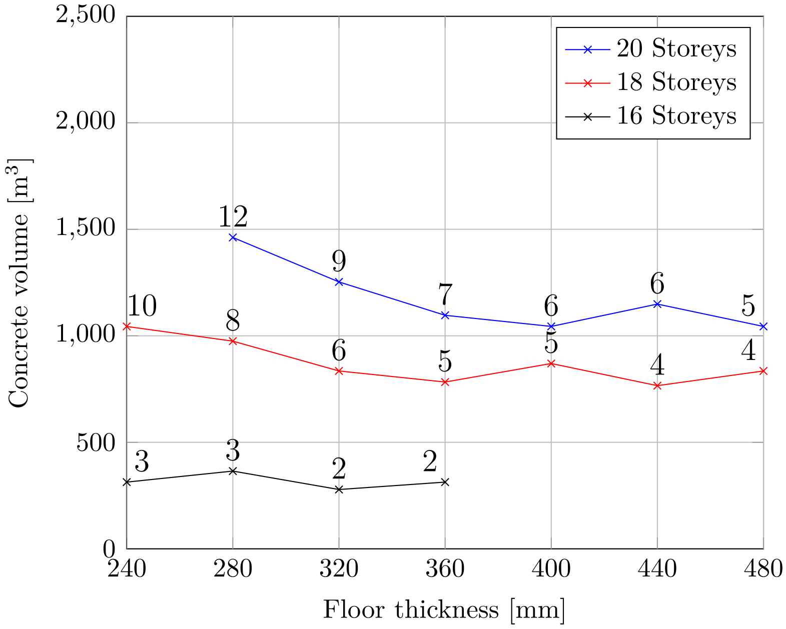



**Figure 4.8:** Influence of floor thickness and concrete volume. Numbers show the number of concrete floors in the building to keep it below the thresholds of ISO 10137 and ISO 6897.

## 4.8 Optimization of a 20 storey building

A 60 meter tall 20 storey building is studied to determine different material efficient solutions to pass the ISO 10137 and ISO 6897 requirements for dynamic responses. The 20 storey building is chosen since it passes the requirements when the CFloor thickness is increased, therefore an optimization when adding CCore is carried out to reduce the concrete volume. For the studied CLT building, changes to the structure are needed to manage the required acceleration for the 20 storey tall building. Such as using concrete in the building which is what this study is focused on. An optimisation is done which focuses on lowering concrete volume in timber-hybrid structures. The placement and thickness of the concrete core and floors are studied to optimise the dynamic response.

### 4.8.1 Adding concrete floors and concrete core

Figure 4.9 shows the response of a 20 storey CLT-T building when concrete elements are added to the building. The blue curve shows the dynamic response of the building as the roof and floors are continuously replaced with 240 mm concrete slab floors from the top, until all 20 CLT floors are replaced with concrete floors. The other curve shows when CLT walls in the core is continuously replaced with 200 mm concrete walls from the bottom up until all CLT core walls are replaced with concrete core walls. Both studies show that the 20 storey building never meets the requirements of ISO 10137 with the considered dimensions. It can be seen in the graph that the natural frequency starts decreasing when cores are added to the upper storeys. This indicates that the added modal mass of the concrete in the core has a bigger effect on the structure than the added stiffness gained from the core. It can also be seen in the figure that when concrete floors are added to the lower storeys, the frequency increases. This suggests that the added modal mass in the lower storeys is less impactful in the structure than the added stiffness gained in the building by replacing timber panels with concrete.

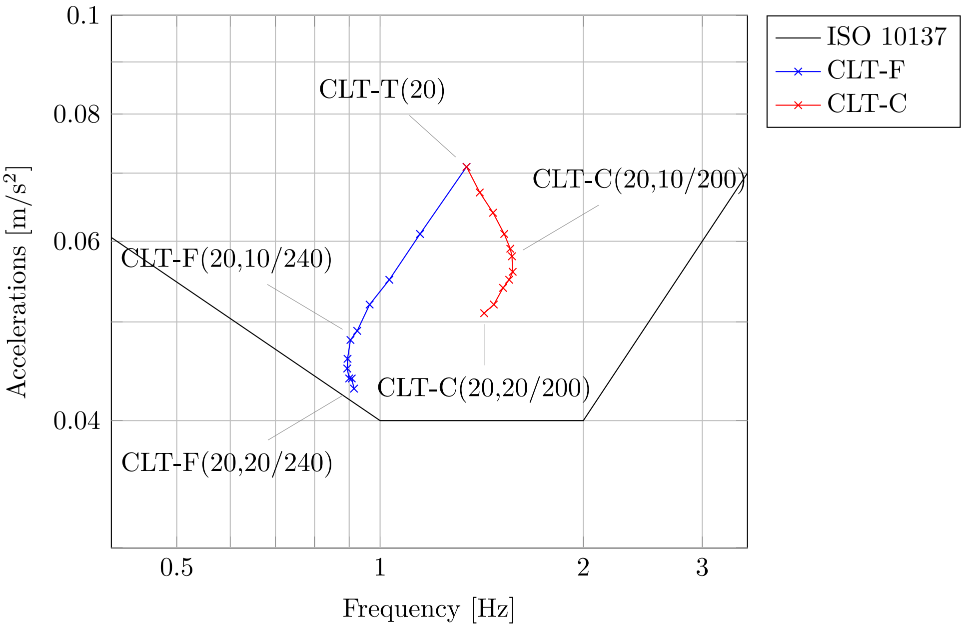



**Figure 4.9:** Dynamic response of a 20 Storey tall building with increasing number of concrete floors or concrete cores.

To find the most efficient placement of concrete elements in the 20 storey building, using 200mm thick concrete core walls and 240mm thick concrete floors. The improvement of adding concrete is shown in Figure 4.10. Effect [%] is how much the addition of concrete elements move the acceleration towards the ISO 10137 threshold. The accelerations for these buildings is compared to the initial CLT-T building without any concrete elements.
It can be seen that there is benefit to gain from including concrete in both core and floors, and that the main benefit of the concrete core is seen in the bottom storeys while the benefit of concrete floors is mainly in the top storeys. While concrete floors significantly less efficient after the first top storeys, the concrete core keeps a larger portion of its efficiency throughout the building's height. This can be explained by the different effects of replacing core walls compared to floors. The concrete core increases both the stiffness and modal mass in the building. Concrete floors contribute very little to the buildings horizontal stiffness while having significant effect on the modal mass in the top storeys.

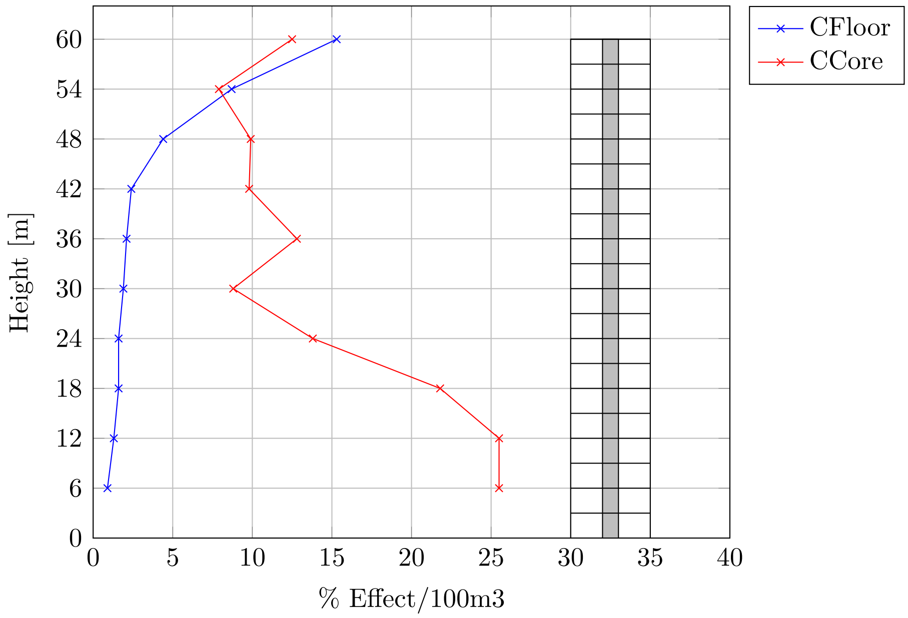



**Figure 4.10:** Effect of adding 2 storeys of CCore or CFloor at different heights of the 20 storey building.

When the frequency of the CLT-F building goes below 1.0 Hz, the response must be compared to ISO 6897 in addition to ISO 10137. Figure 4.11 shows the r.m.s acceleration response of the 20 storey building compared to the ISO 6897 limit for a residential building. For buildings with a natural frequency of 1 Hz or less, the r.m.s. acceleration is below the max limit. It is found that for every building in the study, the ISO 6897 requirements are less strict for the building than ISO 10137. ISO 6897 is therefore disregarded in further studies. As a result, the standard used to evaluate the building is ISO 10137 in all cases.

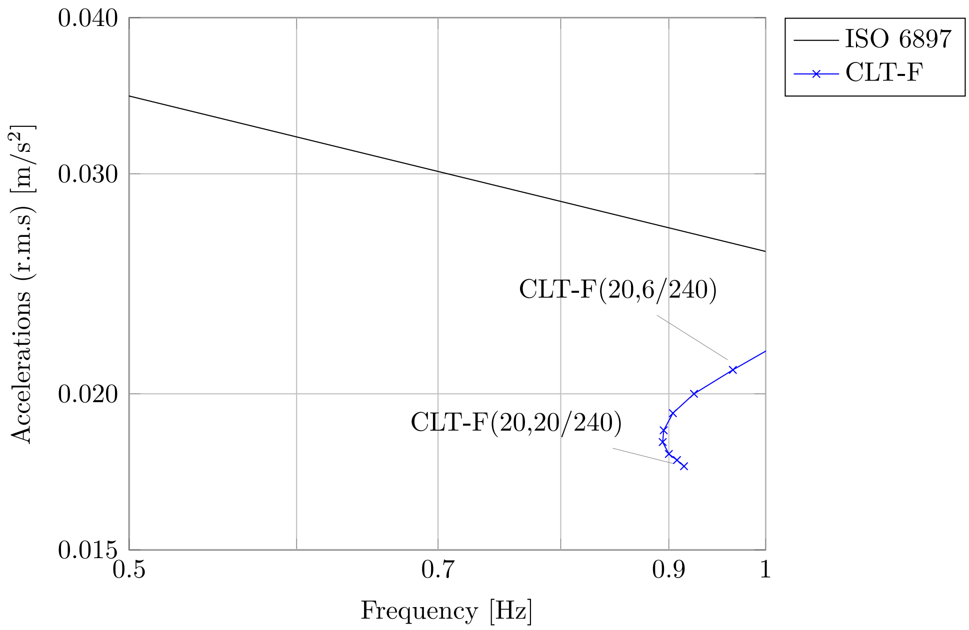



**Figure 4.11:** Dynamic response 20 Storeys high building with increasing number of concrete floors compared to ISO 6897.

### 4.8.2 Concrete elements' effect on mode shape.

A FEM analysis is done where the first mode shape is studied when adding concrete core and concrete floors to the CLT-T(20) building. Figure 4.12 shows the mode shapes of a CLT-T(20), a CLT-C(20,20/200) and a CLT-F(20,20/240). The figure also shows two mode shapes based on equation (2.74) which is stated in EC1. One with the value of n=1.5 which corresponds to a building stabilized with concrete cores according to EC1. One with n=0.8 that corresponds to a building stabilized by timber members according to Edskär (2018).

The resulting mode shape curves show that when adding concrete floors no major difference in mode shape can be seen compared to the CLT-T(20). However when adding a concrete core, a greater difference can be seen. The mode shape shifts towards the mode shape recommended by EC1 for buildings stabilized by concrete cores. The differences in the different mode shapes from FE analysis are relatively small, showing that replacing timber with concrete has a small influence on mode shape. Given that there is a clear difference between the FEM mode shapes and the EC1 mode shapes, the FEM mode shapes should be used for each specific building to get the most accurate results.

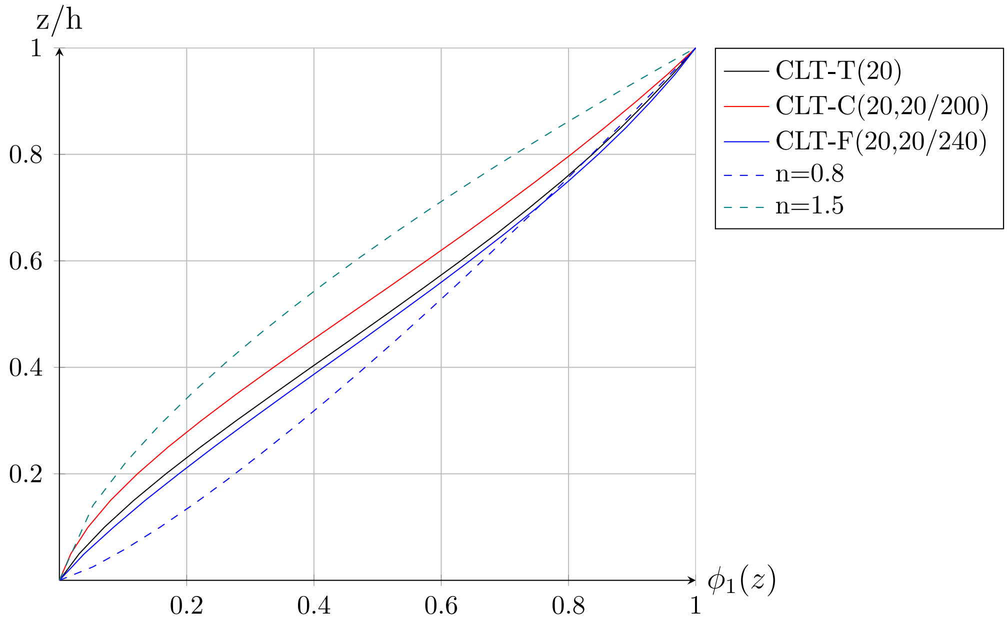



**Figure 4.12:** FEM results of mode shapes in different 20 storey buildings, compared to mode shapes suggested by Eurocode.

### 4.8.3 20 storey concrete core thickness

To produce a 20 storey high CLT-C building, a 200 mm wall thickness is too small to get an allowed dynamic response. A study is done using FEM analysis to find the required concrete core wall thickness for the 20 storey building that fulfils accelerations below the threshold of ISO 10137. Results of the study are presented in Table 4.3. To obtain an acceleration response which is allowed in ISO 10137, the concrete core wall thickness needs to be at least 400 mm CLT-C(20,20/400).

**Table 4.3:** Dynamic response of the CLT-C building with varying concrete core wall widths. Acceleration is compared to the ISO 10137 threshold.

| Thickness [mm] | 200 | 240 | 280 | 320 | 360 | 400 |
|---|---|---|---|---|---|---|
| $f_1$ [Hz] | 1.426 | 1.419 | 1.410 | 1.401 | 1.392 | 1.382 |
| $\ddot{X}_{max}$ | 0.050 | 0.048 | 0.045 | 0.042 | 0.042 | 0.039 |
| % of ISO10137 | 125% | 120% | 113% | 105% | 105% | 98% |

### 4.8.4 Combining concrete core and floor

Figure 4.10 supports the idea that a combination of concrete floors and concrete core may be more optimal than utilizing only one of the alternatives. A 20 storey CLT-CF building is analyzed, which has a concrete core reaching through the full height of the building, and concrete floors added to the top storeys of the building. Changes in core and floor thicknesses are done in increments of 40 mm. The smallest floor thickness used is 240 mm, and the smallest core thickness used is 200 mm. An optimization is done so that for specific floor and core thicknesses, the lowest number of concrete floors possible are used to pass below the threshold of ISO 10137. This means that for the combination of core and floor thickness the building is optimized in terms of concrete volume. Concrete volume is measured in the FE-model and plotted for the studied buildings in Figure 4.13.

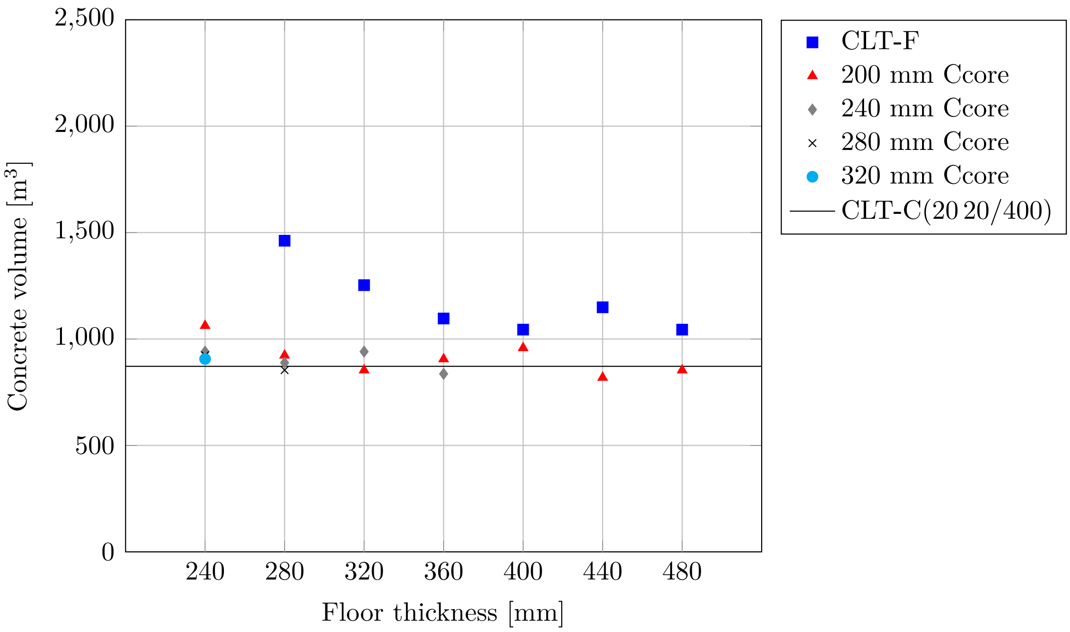



**Figure 4.13:** Total concrete volume used in building depending on CCore and CFloor thickness, for a 20 storey building.

The results suggest that combining a concrete core with concrete floors is generally better for reducing concrete volume than using a CLT core with concrete floors. Additionally, it is shown that using a combination of concrete core and concrete floors produces alternatives which require less concrete than only using a 400 mm concrete core, however the benefit is less substantial. In table **??** the 4 different CLT-types is compared where the most concrete optimized solutions of each type is compared.

## 4.9 Column building

A study is done on the dynamic behaviour of the alternative structural system, made up of glulam columns and CLT panels for floors and core walls. In analysis of the column building using a CLT core as stabilisation results in the first mode of oscillation being a rotational mode. With a rotational mode, accelerations can not be calculated using the EKS12 method. Diagonal bracing is added to the structure to force translational eigenmodes in the building. The bracing also improves the dynamic response by providing stiffness to the system. Figure 4.14 shows the modified column building with bracing is added with a cross pattern on all four sides of the building, the bracing is assumed to have the same cross sections and material as the glulam columns.

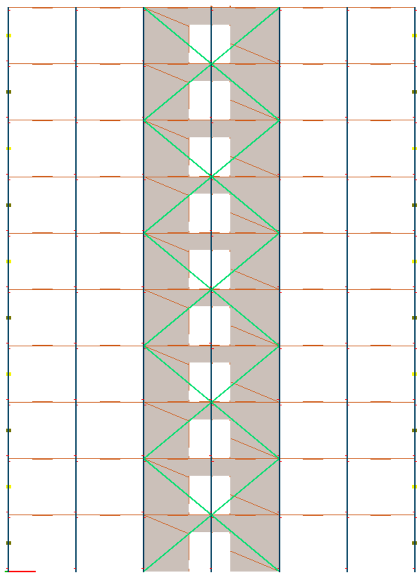


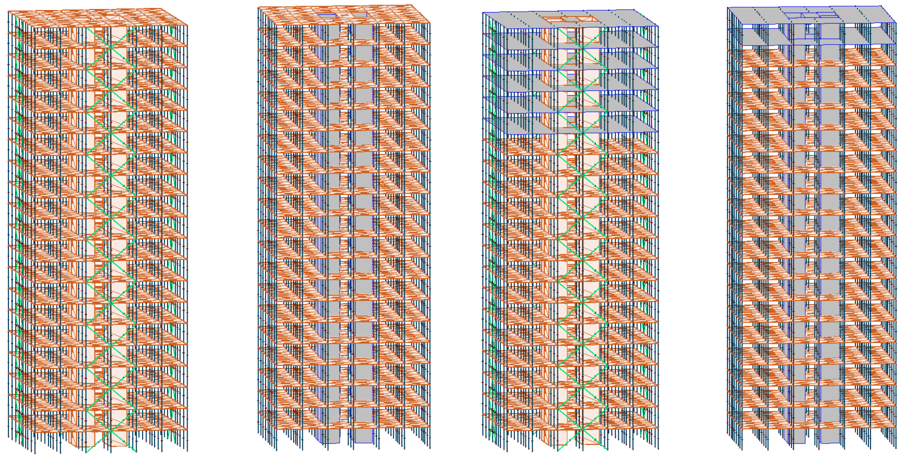
**Figure 4.14:** FE model of the 10 storey column building with added cross bracing.

### 4.9.1 Column building types

Column buildings are categorized similarly to CLT buildings.

- Col-T: The cross-braced pure timber building modelled using glulam columns with CLT elements in the core walls and floors.
- Col-C: Non cross-braced building where the CLT core walls are replaced with reinforced concrete core walls from the bottom up.
- Col-F: Cross-braced building where a number of CLT floors are replaced with reinforced concrete floors from the top down.
- Col-CF: Non cross-braced which includes both concrete core walls and concrete floors.

As for the CLT building, variables such as building height, element thicknesses and number of concrete floors are independent of the building type.



**(a)** **(b)** **(c)** **(d)**

**Figure 4.15:** Examples of the 4 different column building types: (a) Col-T, (b) Col-C, (c) Col-F, (d) Col-CF

### 4.9.2 Analysis of Col-T, Col-C and Col-F buildings.

The dynamic response of the Col-T building is analyzed and then compared to Col-C and Col-F buildings. Results are compared to the ISO curves for both residential and office requirements due to the structural system's suitability to function for both types of building.

Concrete elements are included using the same initial dimensions of concrete elements as for the CLT buildings. Concrete floors having a thickness of 240 mm and concrete core walls a width of 200 mm. Analysis is made for 10-20 storeys of the Col-F building where all the floors consists of 240 mm concrete. The analysis shows that none of the buildings 10-20 storeys manages the residental requirements of ISO 10137 but that both acceleration and frequency is lowered compared to the timber floor building as a result of increased modal mass.

An analysis is also done for the Col-C building where the core consists of 200 mm thick concrete in the full height if the buildings. The results show that the core gives sufficient horizontal bracing to prevent a torsional mode by itself, therefore the diagonal bracing is not included in the concrete core analysis. The frequency is increased and the accelerations is decreased for the CLT-C building compared to the Col-T building. The increased frequency is due to a higher stiffness of the concrete core compared to the timber bracing while the decreased acceleration. The analysis of Col-C and Col-F buildings are shown in Figure 4.16 together with the Col-T building.

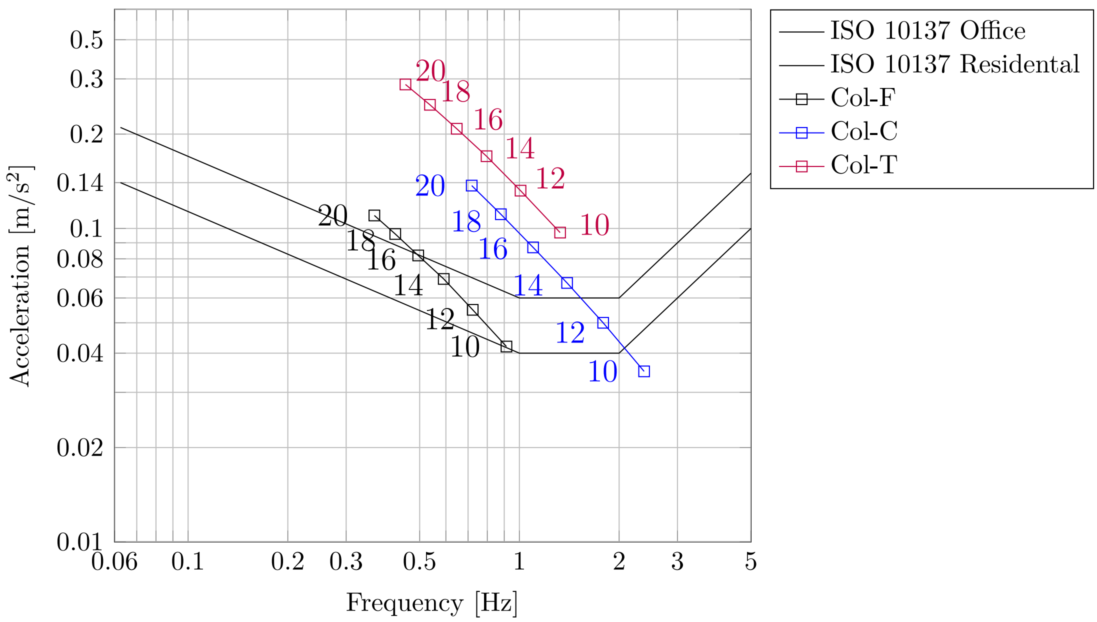



**Figure 4.16:** Dynamic response of the Col-T, Col-C and Col-F buildings.

A 20 storey Col-CF building is analyzed with varying core thicknesses and floor thicknesses. The models are designed with a concrete core at all the storeys of the building and the fewest possible concrete floors are used in the building in order to pass ISO 10137. This means that for the combination of core and floor thickness the building is optimized in terms of concrete volume. Figure 4.17 shows the results. The results suggest that larger concrete core widths and concrete floor thicknesses generally lead to lower concrete volumes in the building. This is unlike the CLT-CF building where such a pattern can not be observed.

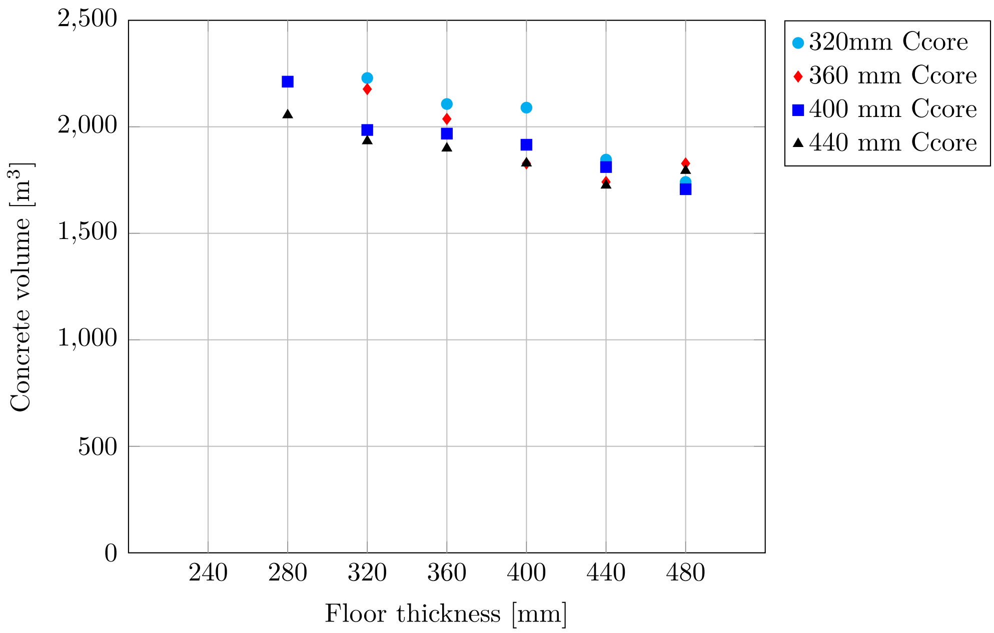



**Figure 4.17:** Concrete volume in 20 storey Col-CF buildings with different CCore and CFloor sizes.

### 4.9.3 Comparison between CLT and Col systems

Comparing the Col-T building with the reference CLT-T building shows that the column building requires larger concrete dimensions than for the the CLT building to pass the requirements of ISO 10137. Another observed difference is higher acceleration and lower natural frequency in all cases, showing that the effect of lost stiffness in the building is higher than the loss of mass as a result of replacing the CLT shear walls with timber columns.

Compared to the CLT system, the column system requires less timber in the structure. Table 4.4 compares properties Col-T building with those of a CLT-T building. Both buildings are pure timber buildings. The timber volume in the structural systems which excludes the non load-bearing walls and facade elements is smaller for the Col-T than for the CLT-T as expected. The mass of one storey in the model is also smaller as a result of this - leading to a smaller equivalent mass in the column building. The stiffness of each system is compared by calculating the horizontal deflection from a horizontal load on one storey, showing that the CLT-T system has a many times higher stiffness per storey.

**Table 4.4:** Comparison between a CLT-T and a Col-T building.

| | CLT-T | Col-T |
|---|---|---|
| Timber volume (10 storey building) | 2465 $m^3$ | 1083 $m^3$ |
| Mass of 1 storey | 147 t | 104 t |
| Equivalent mass (10 storey building) | 45056 kg/m | 30049 kg/m |
| Deflection of 1 storey (100 kN/m) | 0.302 mm | 2.019 mm |

As a result of the Col-T having lower equivalent mass and less horizontal stiffness, more concrete is required in the column building than in a CLT building to achieve an allowed acceleration.

Table 4.5 compares the two systems' concrete volume requirement to pass below the threshold of ISO 10137 at different building heights. The comparison is done between CLT-CF and Col-CF for the most concrete volume optimized buildings found during analysis of both systems. The results leads to the conclusion that the column building always requires more concrete than the CLT system at the same building height.

**Table 4.5:** Volume of concrete required to pass the requirements of ISO 10137 at different building heights, using the most concrete volume optimized alternatives found during FEM analysis.

| Building | CLT-CF [$m^3$ concrete] | Col-CF [$m^3$ concrete] | Increase [$m^3$ concrete] |
|---|---|---|---|
| 16 storeys | 131 | 976 | 845 (645%) |
| 18 storeys | 497 | 1437 | 940 (189%) |
| 20 storeys | 814 | 1707 | 893 (110%) |
| 25 storeys | 1960 | 3085 | 1125 (57%) |
| 30 storeys | 3162 | 4676 | 1514 (48%) |

### 4.9.4 Comparison of mode shapes

It is found during the study that for the column system, a concrete core provides better dynamic response improvements than multiple concrete floors do. It is generally most efficient to always utilize a concrete core in the column systems. Figure 4.18 shows the mode shape of a 20 storey Col-T building compared to the mode shape of a 20 storey CLT-T building, based of FE-analysis results. The curves show that a Col-T has smaller mode shape values along its height, since the modal mass is a function of the squared mode shape, it follows that added mass in the column building has less influence on the modal mass of the structure than in the CLT building. This is one reason why the stiffness gain of a concrete core has a larger relative effect on the buildings with column systems than added mass from concrete floors.

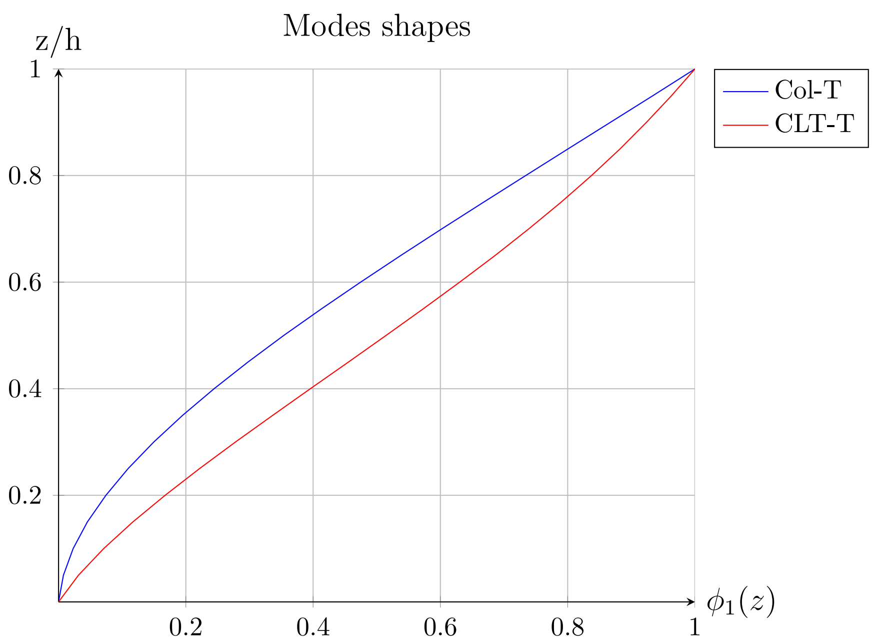



**Figure 4.18:** Mode shape of a Col-T building compared to a CLT-T building.

## 4.10 Study of buildings taller than 20 storeys

The analysis of the CLT and column buildings shows that it is possible to manage the dynamic requirements in the 20 storey buildings. This is done by replacing CLT-panels with reinforced concrete in the structural systems. A further study is carried out by extending the storey height up to 40 storeys of the previously analyzed systems. In addition to that a new system called mixed is added in the trying of lowering the use of concrete in the structural system.

## 4.11 Mixed building

A new model of the building is made by combining the CLT- and column-system and is categorized as a mixed building. The mixed building consists of a CLT-system at the lower storeys and the 5 top storeys of a timber column system with concrete floors. The storeys in the building that consist of the CLT-system is chosen to function as residential or hotel. The 5 highest storeys are designed to function as office storeys. At the bottom storey of the column system part, a concrete floor of at least 240 mm is added for the the vertical load distribution of the timber columns to the CLT walls.

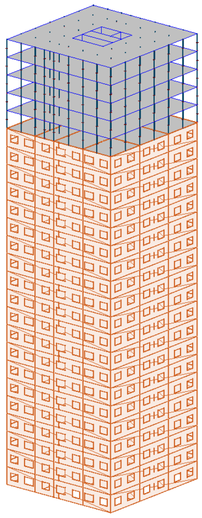


**Figure 4.19:** FE model of the 25 storey mixed building. The bottom 20 storeys are designed with a CLT system. The 5 top storeys are designed with a column system.

In the CLT part of the mixed building which is intended to function as living or hotel space, the residential requirements of ISO 10137 are applied. For the office part however, ISO 10137 allows for 50% higher accelerations at the same frequency. Given that the highest accelerations occur at the top of the building, it is beneficial to design the building is such a way that the requirements allow for higher accelerations in the building's upper parts. The building height can then be extended to a higher level than what would be allowed using only the stricter residential requirements.

Another favorable effect of adding the column system is the increased mass at the top of the building from the concrete floors. The increased mass of the concrete floors have the influence of reducing both the frequency and acceleration which is analyzed in chapter 4.8.1. The beneficial effect comes from placing the mass in a position where it is allowed more movement than it normally would, which allows it to be placed in an optimal spot for modal mass while reducing the acceleration in the residential part where the requirements are more strict.

### 4.11.1 Analysis of mixed buildings

FE analysis is carried out on the mixed building with the heights of 16, 18, 20, 25, 30, 35 and 40 storeys. Figure 4.20 illustrates the dynamic response for the most optimized mixed buildings found in the analysis. Optimized meaning that the least amount of concrete is used to produce the building. The acceleration is then compared with the residential and commercial requirements of ISO 10137. It can be seen in figure 4.20 that for buildings higher than 20 storeys, the residential storeys govern acceleration limits. In contrast, for the lower buildings the top commercial storey is governing in terms of building height.

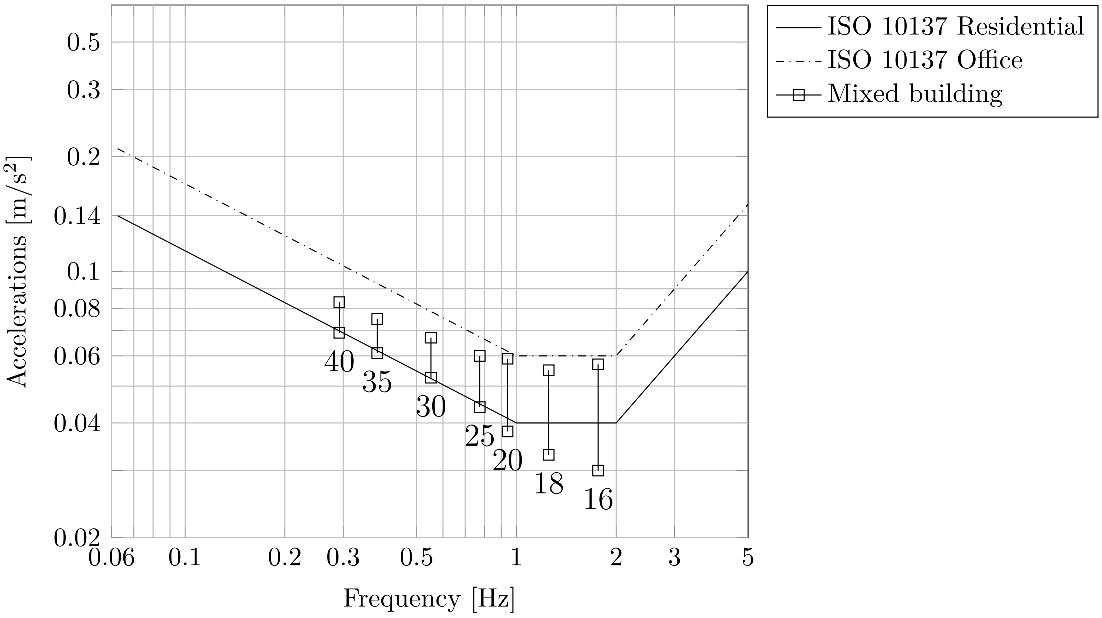



**Figure 4.20:** Dynamic response of mixed building. The the dynamic response in the top office floor is connected with a line to the response in the top residential floor. Numbers next to marks show the total number of storeys in the building.

### 4.11.2 Comparison between different structural systems

In Table 4.6 a comparison between the most concrete volume optimized CLT-CF and mixed system is presented. By using a mixed system instead of the CLT-CF the analysis shows that the concrete volume can be reduced significantly.

**Table 4.6:** Comparison of concrete volume between CLT-CF and Mixed systems.

| Building height | CLT-CF [m$^3$ concrete] | Mixed [m$^3$ concrete] | Difference [m$^3$ concrete] |
|---|---|---|---|
| 16 storeys | 131 | 104 | -27 (-21%) |
| 18 storeys | 497 | 365 | -132 (-27%) |
| 20 storeys | 814 | 452 | -362 (-44%) |
| 25 storeys | 1960 | 1084 | -876 (-45%) |
| 30 storeys | 3162 | 2025 | -1137 (-37%) |
| 35 storeys | 4968 | 3439 | -1529 (-31%) |
| 40 storeys | 7245 | 4964 | -2281 (-31%) |

The mode shape of a mixed building is different from that of a CLT or column building. Figure 4.21 shows the mode shape of a mixed building based on FEM analysis results. The mode shape has a noticeable change in angle at the bottom of the column part. In the lower residential CLT part of the building, the mode shape is almost linear with some bending. In the weaker column structure on top, the mode shape clearly shows more shear deflections.

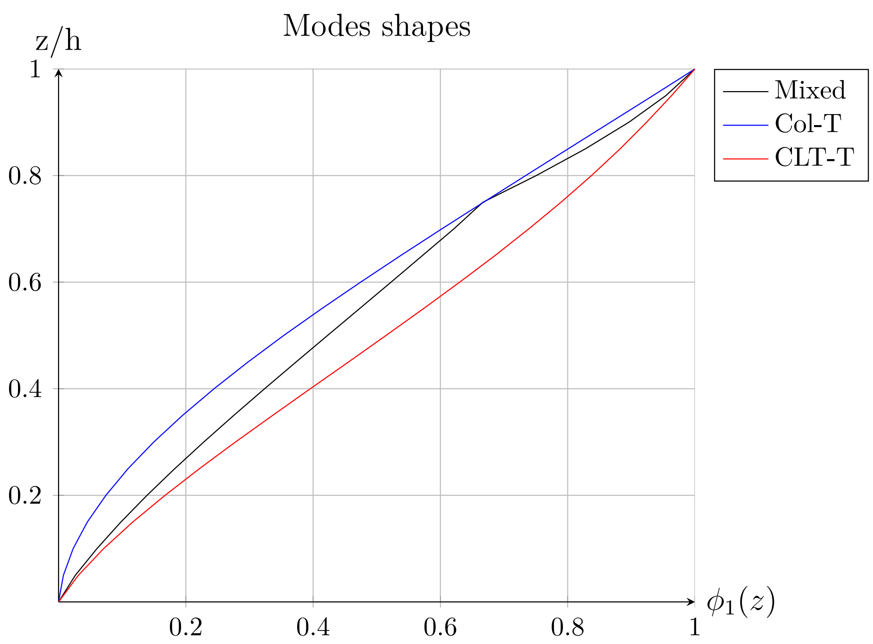



**Figure 4.21:** Mode shape of a 20 storey mixed building, compared to 20 storey CLT-T and Col-T.

## 4.12 Material comparisons between different systems.

The total concrete volume needed for the optimal solution is compared between structural systems. The total volume of concrete per rentable square meter [$m^3/m^2$] is used as the unit of comparison so that the material efficiency can be compared between different building heights. Figure 4.22 shows concrete volumes depending on building height in the three different structural systems. A column building designed with office standards in ISO 10137 is also included in the comparison to illustrate the difference between requirements. It can be concluded from the figure that concrete volume per $m^2$ can be approximated as linearly increasing values with building height, showing that a taller building is less material efficient than a lower building in terms of concrete volume for all systems. In the figure, the estimated concrete volume of existing timber buildings is included. The concrete volumes are assumed based on drawings and reports available online. From the estimated values, two of the existing buildings are within the range of concrete volume that are found for the studied building. The Ascent is more concrete efficient for its height than any building found during the study.

### Reinforced concrete

In a study made by Kim et al. (2009) the material quantities for the structural system of a reinforced concrete building is analyzed. In the study buildings of 20, 30, 40 storeys with a storey height of 3m is analyzed. The analyzed system in the study is a combination of concrete shear walls and concrete columns. Results from the study is shown in figure 4.22 (RC) where the concrete volume of the foundation is neglected. From the figure in can be seen that up to approximately 120m the timber-hybrid structure requires less concrete than the reinforced concrete building. The purpose of including the concrete building is to use as reference against concrete volumes in timber-hybrid buildings. The concrete building is however designed only for static cases and the comparison is therefore not completely accurate.

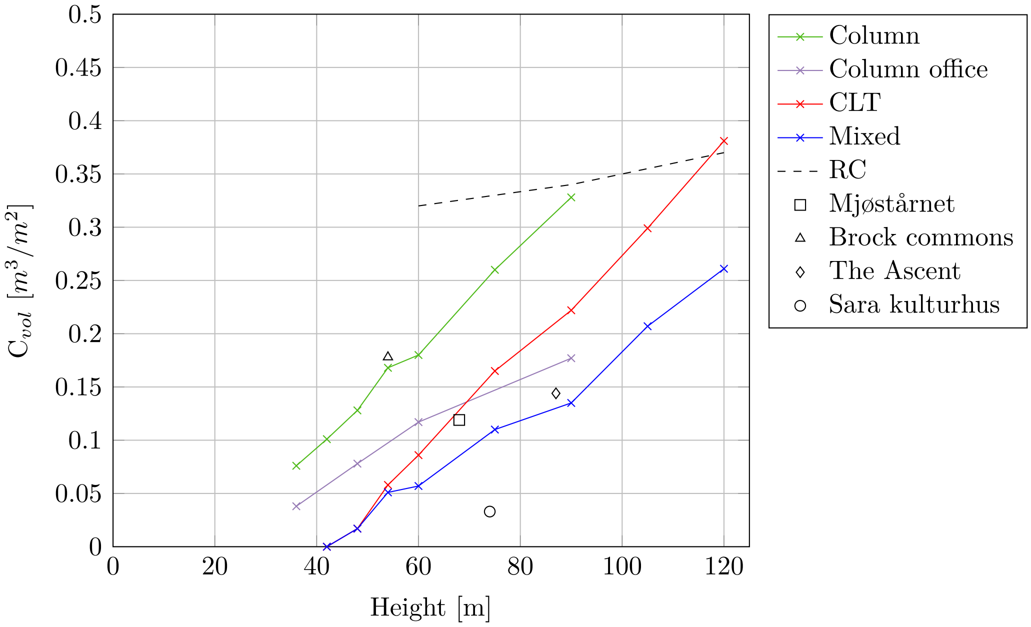



**Figure 4.22:** Concrete volume per floor area $[m^3/m^2]$.

To get a full picture of material usage for the different system, a similar comparison for the timber volume is done for the same buildings. Figure 4.23 shows the timber volumes for the different structural systems. The figure clearly displays the decreasing volume of timber volume used per $m^2$ rentable area as the building height increases.

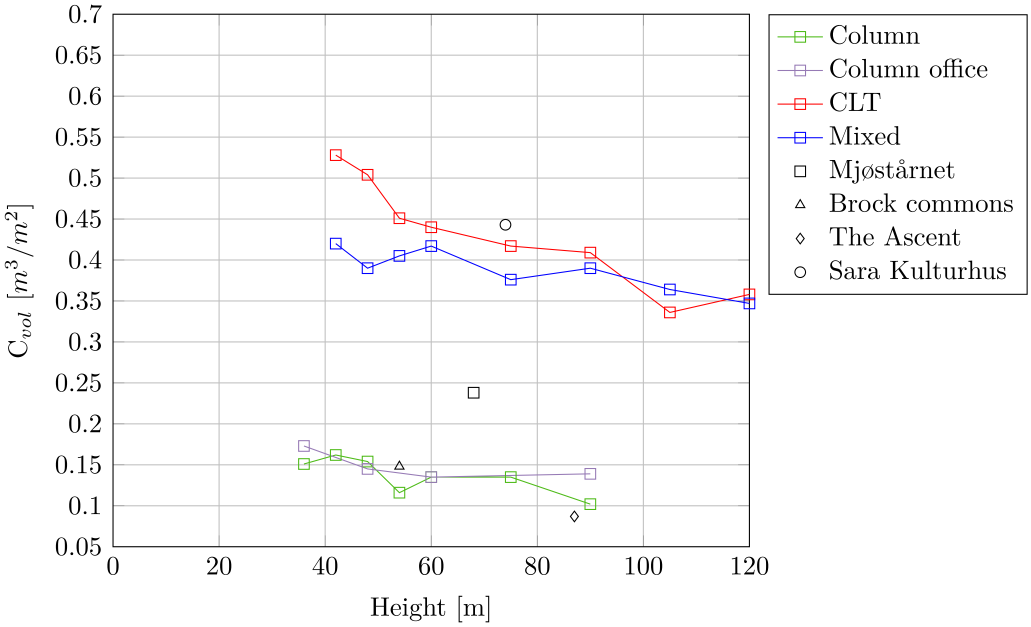



**Figure 4.23:** Timber volume per floor area $[m^3/m^2]$.

The change of material with increasing volume of concrete and decreasing volume of timber as the building height increases relates dynamic behavior of the building. As the building gets higher the extra stiffness and mass of the concrete gives a beneficial behavior.

## 4.13 Effect of tuned mass damper

The effect of adding a TMD is studied for different buildings from the analysis. The goal of using a TMD is to reduce acceleration in the building to be able to build taller and with less concrete.

The intention of introducing a TMD is to add equivalent damping to the system to move the building's peak acceleration $\ddot{X}$ below the threshold of ISO 10137. For different buildings, different properties in the TMD are used to get the correct equivalent damping $\xi_{eq}$ while using as small mass as possible in the TMD. Table 4.7 shows the required equivalent damping for a chosen selection of different buildings and the optimal TMD specifications to achieve the damping.

Another application of a TMD is to increase the structural height of timber-hybrid buildings without using unreasonable amounts of reinforced concrete in the structure. Table 4.8 displays resulting concrete volume reduction by introducing tuned mass dampers. The tuned mass damper allows for building heights of at least 40 storeys (120m). Compared to The Ascent, which is the tallest tower today at 86m, the maximum building height of a timber-hybrid building is higher than what is built today when considering dynamics.

**Table 4.7:** Different buildings with required equivalent damping $\xi_{Eq}$ and the mass and damping needed in the TMD to achieve the equivalent damping in the system.

| Building | $m_s$ [ton] | $f_1$ [Hz] | $f_{opt}$ [-] | $\xi_{Eq}$ [%] | $\xi_{TMD}$ [%] | $m_{TMD}$ [ton] | $\overline{m}$ |
|---|---|---|---|---|---|---|---|
| CLT-T 16S | 920 | 1.786 | 0.998 | 2.4 | 1.6 | 1.56 | 0.17% |
| CLT-T 18S | 1031 | 1.540 | 0.994 | 3.4 | 3.0 | 4.12 | 0.40% |
| CLT-T 20S | 1111 | 1.343 | 0.989 | 4.5 | 5.0 | 9.67 | 0.87% |
| Col-C 14S | 621 | 1.393 | 0.991 | 4.1 | 4.0 | 4.47 | 0.72% |
| Col-C 16S | 621 | 1.100 | 0.9915 | 6.7 | 7.2 | 18.19 | 2.93% |
| Col-C 18S | 694 | 0.897 | 0.988 | 4.6 | 4.5 | 7.29 | 1.05% |
| Col-C 20S* | 766 | 0.718 | 0.978 | 5.8 | 5.9 | 16.01 | 2.09% |
| 35S Mixed | 3334 | 0.467 | 0.993 | 3.8 | 3.8 | 17.67 | 0.53% |
| 40S Mixed | 3505 | 0.386 | 0.99 | 4.4 | 5.8 | 26.99 | 0.77% |
| 40S CLT-CF | 2479 | 0.352 | 0.981 | 5.7 | 6.8 | 40.90 | 1.65% |

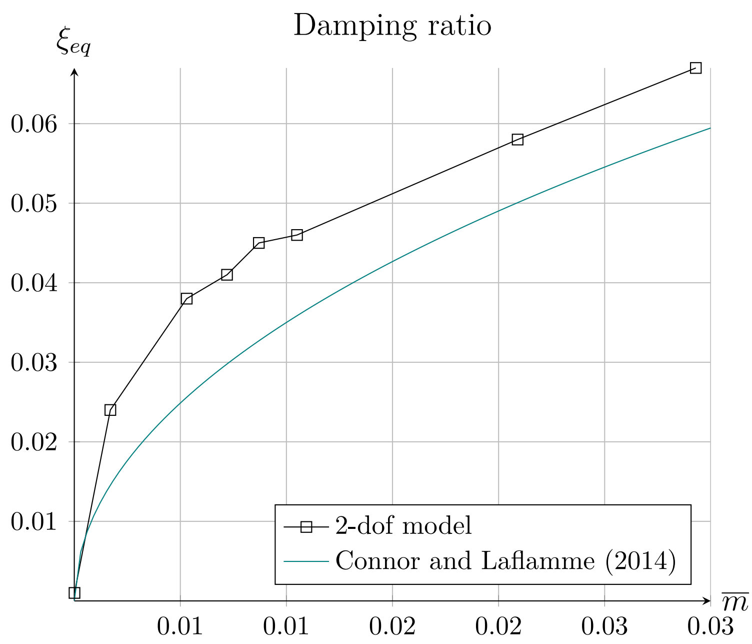



**Figure 4.24:** Comparison of calculated damping vs Connor and Laflamme's estimation of optimal damping depending on the mass ratio of the TMD

### Comparison of concrete volume with or without TMD

A FEM study is done to analyse how much concrete can be saved in timber hybrid buildings by utilizing the damping from TMD:s instead. Table 4.8 shows concrete volume requirements with no TMD and after introducing a TMD into a chosen selection of different buildings. The results show that it is possible to reduce the concrete volume by more than half in every building where a TMD is introduced.

**Table 4.8:** Different types of buildings with their concrete requirements before and after introducing a TMD to the system.

| Concrete comparison | | | |
|---|---|---|---|
| Building | Cvol [$m^3$] No TMD | Cvol [$m^3$] With TMD | diff |
| CLT-CF 20S | 814 | 0 | -100% |
| CLT-CF 40S | 7245 | 2788 | -61.5% |
| Col-C 14S | 671 | 305 | -54.5% |
| Col-C 16S | 976 | 349 | -64.3% |
| Col-C 20S* | 1115 | 436 | -60.9% |
| Mixed 35S | 3439 | 1285 | -62.6% |
| Mixed 40S | 4964 | 1394 | -71.9% |

*Office requirements for accelerations

# 5

# Discussion

Dynamically optimizing a high-rise timber hybrid building is a task where there are a great number of possible combinations of parameters. It has therefore been important to decide what analyses are most important to perform in order to get results that most effectively support the aim of the project. Many decisions have been made as a result of discussions during the project and from studying analysis results. As an example, the mixed structural system was not intended to be analyzed as part of the project, but was instead found by studying the ISO 10137 curve together with earlier results from CLT and column buildings.

During the study, different concrete dimensions have been tried in FEM-Design in order to find efficient solutions. An issue during analysis has been uncertainties about the practicality of some designs. The maximum floor thickness has, for example been limited to 480 mm in the study. This limit might be within a reasonable range. However it is possible that it is too large to use in real buildings or that it is possible to find more effective solutions by increasing the concrete floor thicknesses even more. In those cases, the study is incomplete since it does not include every alternative. The same reasoning has been used for limiting core wall thicknesses, and some solutions have been disregarded due to the unreasonable sizes of core elements. In theory there is no limit to how thick core walls can be designed as long as there is free space in the area around the core. It has however been the goal to find a solution which is realistic and is based somewhat on existing solutions. In those cases, the limits chosen for analysis make the study possible to perform with a higher probability to find practically reasonable solutions.

A lot of focus was put on 20 storey tall buildings. This is done because after initial analyses, it was clear that 20 storeys were an achievable height, which required both CLT and column buildings to include concrete. The 20 storey buildings also allowed for many different solutions to be tried without concrete element sizes becoming too large.

## 5.1 Analysis of results

### 5.1.1 Addition of concrete core and concrete floor

The analysis suggests that the addition of concrete core is useful throughout the full height of both CLT and column buildings. The frequency values in Figure 4.9

shows that the increase of stiffness is most effective in the lower half of the building for the concrete core while the mass has more effect in the upper half of building. It therefore appears reasonable from a production method perspective, that if a concrete core is planned in a timber-hybrid building, it should reach to the top. This is also what can be seen in existing tall timber-hybrid buildings.

The concrete core have the benefit of both being able to work as lateral stabilization of the building and to carry vertical loads of the surrounding structure. Disadvantages of increasing core thickness when needed is that the total available floor area is decreased while by increasing the floor thickness when needed the total building height increases, both of this aspects is needed to account for in the design.

The addition of concrete floor have the most effect in the top third of the building which is the same that EC1 states. There seems to be no reason from a dynamics perspective to add concrete floors on the bottom of the building instead of at the top. From the FEM results, it can be seen that the benefit of concrete floors drops off more quickly when moving away from the top. It seems like replacing every floor with concrete is not a very efficient method of solving dynamic problems in the building. Instead, it is much more beneficial to concentrate large portions of mass at the highest possible point to increase the modal mass of the building.

A concrete floor can be used to increase the building's modal mass while at the same time functioning as an insulating floor for installations or as a load bearing element for a TMD, since a concrete floor often has greater capacity than a CLT floor and often have larger thicknesses than timber floors. It is however a disadvantage with concrete floors that they add more vertical loads in the structure from their self weight.

During the analysis it is found that a combination of replacing both the CLT-core and CLT-floors often is a more efficient solution than just changing the core or just the floors in terms of reducing total concrete volume. However depending on the building, the benefit of including both is sometimes small, and therefore other factors should be considered first. The other factors include production method, static load bearing, and architectural design.

### 5.1.2 Structural systems

Three types of systems are analyzed in the report CLT- , Column- and mixed-systems.

Because of the vertical load bearing function of the CLT walls, it is difficult to change the floor plan of such building after construction. Since CLT-panels are a solid timber product, a consequence of this that buildings produced using CLT panels use relatively high amounts of timber material. This is confirmed in results in Figure 4.23.

Column buildings has the advantage creating a more open floor area but also more easily variable floor plan layout as the lateral loads is transferred through the columns and not through the walls as the CLT-building. Disadvantages is the reduced stiffness and mass of the column building which has a negative effect on the dynamic behavior. In order to achieve accelerations below the threshold, analyses shows that higher amount of concrete is needed than for the CLT-based system. Another disadvantage is that the horizontal load bearing capacity of the column system is smaller than for a shear wall system, and therefore there is a significant possibility that horizontal bracing is required. Horizontal bracing generally causes issues by taking up space and limiting the architectural possibilities in the building.

The main positive aspect of using the mixed building as structural system is the allowance of higher acceleration as the top storey can follow the ISO 10137 standard for offices. By utilizing a column system with concrete floors at the top 5 storeys of the building the added mass is placed at the most efficient location which has a positive effect in the building dynamic response. A disadvantage of the system is that it is uncommon to build offices on top of residential storeys, and if the CLT storeys are instead used as hotel, the building is limited by the hotel area needed in the building.

If a tall timber hybrid building with the most optimal solution in terms of concrete volume is desired, then the mixed building type is recommended as is shown in Figure 4.22. It might however not be practical to include the column system on top, which means that a CLT building is instead the best solution. The column building with residential requirements is the least concrete volume optimized system, however it does use significantly less timber than the alternatives. If the goal is building as high as possible, then a mixed system is the best alternative. The CLT system is also possible to use, however the dimensions of concrete elements might become too large for practical applications.

It has been shown in the study that mode shapes from FEM analyses are not similar to the recommended mode shapes given by Eurocode. The results suggest that the CLT building has a mode of mostly shear which is verified by the mass-spring model calculation, while the column building has a mostly bending mode. It is therefore an unexpected result that the mixed building has more bending in the lower CLT part with more shear in the top column part. The mode shapes of timber-concrete hybrid buildings are more a combination of bending and shear rather than one clear type, as is the case with the Eurocode equation. It can still be argued however that the Eurocode mode shapes are good enough approximations for all timber and timber-hybrid buildings when no FEM analysis results are available.

### 5.1.3 Building height

No theoretical limit to building height due to accelerations were found during the study. The height is instead limited for practical reasons. Many of the concrete elements become too large to practically fit in a building. Another issue when building higher is the building floor plan. The concrete core needs to become larger as the number of storeys increase. This is to make room for more elevators, staircases and installations as the building becomes utilized by more people. The initial concrete core with one elevator shaft is not realistic for a 40 storey tall building. Therefore it is not practical to study any taller buildings without redesigning the building's floor plan and foot print.

### 5.1.4 Comparing with existing buildings

All of the existing buildings discussed in this report utilize concrete in their structural system. Hoho tower, Brooks commons and The Ascent utilize a combinations of concrete cores and timber-concrete composite floors. Sara kulturhus, Treet and Mjøstårnet use concrete floors to increase modal mass. Some of the buildings have bottom storeys made of only reinforced concrete. None of the tallest timber-hybrid buildings currently use a combination of a concrete core with reinforced concrete floors. It can be argued that concrete floors are better than timber-concrete composite floors from a pure dynamics point of view, since it allows for more mass to be concentrated at the top where it contributes more to modal mass. The study also shows that it is more efficient to use a combination of concrete core and floors than just using concrete floors. There may be many reasons to not use the combination of these elements, for example for practical production method reasons. Another reason might be to minimize the number of concrete "elements" in the building, in order to allow more timber to be exposed for architectural reasons.

In the comparison between the studied FEM models and the existing buildings it can be seen that the volumes of both concrete and timber is in the same ranges when volume material is compared to the gross area of the buildings. For the FEM models the exact material volume is calculated in the model. Meanwhile in the existing buildings the concrete and timber volume is roughly estimated according drawings and reference literature. The estimations of material volume in existing buildings generally resulted in existing buildings being within the expected material efficiency calculated in FEM models.

### 5.1.5 Results of the TMD study

The reason for doing the study of tuned mass dampers in the system is to learn more about if TMD:s are an option for high-rise timber hybrid buildings. The first part of the study of TMD:s showed that many buildings over the ISO 10137 limit could be improved to allowed levels of acceleration using reasonable mass ratios in TMD:s of $\bar{m}$ = 0%-3%. It is also clear that it is possible to build 40 storey (120 m) tall timber-hybrid buildings with reasonable concrete volumes and dimensions using realistic masses in TMD:s. Without any added damper in the system this may be a challenge. The study shows that around 60% concrete volume can be reduced in a building by using a TMD. When comparing the options of using a TMD to using higher concrete volumes in the building, some different factors need to be considered:

- How large of an improvement in acceleration can be gained from a tuned mass damper?
- What is the cost of installing a TMD in a building, and how much space does it require?
- What is the environmental impact of producing and installing a TMD compared to the reinforced concrete it replaces?
- At what building height is a TMD worth considering instead of more concrete in a timber-hybrid building?

It is difficult to weigh advantages and disadvantages in a general case and in each specific building case it might be more obvious if a TMD is reasonable. It does however seem from the results in the study that a TMD has the potential to be an effective option in cases where the dynamic effects in a building are problematic.

## 5.2 Potential sources of error

Multiple sources of error are present in the project as a result of simplifications, approximations and assumptions done during the project.

Damping is a relatively large uncertainty that affects every dynamic oscillation analysis. It is a factor with significant influence on the results that is difficult to determine in a building using calculations or FEM analysis. Furthermore, structural damping in a building changes depending on materials used and the building's geometry. This means that in reality each building analyzed has a unique damping percentage. A value of 1.5% has been estimated in every building which has been determined by looking at tests from other tall timber hybrid buildings, Eurocode and recommendations from other authors. However, it is unlikely that most buildings in the study have a structural damping of 1.5% in reality. Therefore resulting accelerations in the study should be seen as approximations.

The tuned mass damper is not included in the FEM model of the building. It is instead introduced as a change in structural damping in the post processing calculations. This means that the change in dynamic behaviour from adding a TMD to the building is not analyzed, and only the damping ratio changes. Furthermore, modelling of the equivalent damping of a TMD is done using a simplified 2-dof system and should therefore be seen as an approximation of the actual equivalent damping gain from the TMD.

It was shown in Chapter 4.6.1 that the assumed influence of panels over doors have a significant effect on the building's acceleration. It was decided that all panels over doors should utilize their full stiffness. The large changes in acceleration were found when the stiffness utilization of the CLT panels were reduced to unreasonably small levels (0% - 30%). When looking at stiffness utilization that appear more realistic (50% - 100%) it is found that the accelerations vary by a few percent and therefore a small uncertainty comes from this effect.

In the optimization of reducing the concrete volume the same strength of concrete is used in the models, where the core and floor thickness is changed to increase the stiffness and mass. Further optimisation can be carried out mainly by increasing the stiffness of the core to which may reduce the need of extra mass and could have high influence on the required concrete volumes.

The hand calculated accelerations in EC1 and EKS12 are done using an approximate method that requires the building to be assumed as a free cantilever oscillating in one plane. The building is in reality however an asymmetrical 3D structure with spread out connections to a foundation. The wind is also modelled in hand calculations as an approximation based on empirical results. Because of these reasons, using EC1 and EKS12 to calculate the dynamic response in a building requires that many details are disregarded, and therefore the results cannot be fully accurate. It is however an accepted method of determining accelerations which is practical to use in many cases.

## 5.3 Thoughts about high rise timber-hybrid buildings

During the study, different combinations of material choices have been utilized, with some buildings being pure timber buildings to others having a large portion of reinforced concrete included in the structure. It is therefore interesting to decide what a reasonable concrete volume in a timber hybrid building should be in reality. One definition that can be used for timber buildings is that the lateral load bearing system has to be made entirely out of timber for the building to be called a timber building. While for timber-hybrid structures no clear definition exists. In the analyses made in this project, models where 0-74% of the material volume is concrete are analyzed. It is debatable when a building can be called a timber-hybrid structure and when it becomes a concrete structure. There is no clear border but at the point where the concrete volume exceeds the volume of timber used in the building it is obviously difficult to name it a timber-hybrid building.

## 5.4 Future studies

From the work presented in this report the following need for further research can be identified:

- Including TMD in FEM model

  It was found in the project that tuned mass dampers could possibly be successfully used to improve the dynamic response in high-rise timber hybrid buildings. There were however uncertainties in the method used to calculate the equivalent damping and it is unclear how much mass can be used in a TMD until the timber elements can no longer support the weight. A FEM study where a TMD is included in the model could be used to more accurately find the influence of the damper and to understand how the surrounding structure is affected by the concentrated mass.

- Controlling for deflections and ULS from wind loads

  The study showed results in terms of the comfort limits given by ISO 10137 and ISO 6897. These standards are developed with human perception of motion as a limiting factor. A future study could focus more on deflections and structural capacity in a timber hybrid building as a result of dynamic wind loads.

- Modelling connections between timber elements

  Connections between elements were modelled in a simplified way. Finding a method of modelling connections and seeing their influence on the dynamic response is possibly a future study that can be done on a timber or timber hybrid building.

- Elements around openings

  Elements around the openings was modelled as fully stiff neglecting the possible local effects when the geometry of the panels is changed. A future study could focus on studying how the local effects around the opening is influencing the stiffness of the CLT when openings is cut out of a solid panel.

# 6
# Conclusion

By comparing allowed building heights between timber buildings and timber-hybrid buildings it is concluded that building heights can be significantly increased when concrete is included. The results show that there are different methods to optimize a high-rise timber building's dynamic response. It is shown that when increasing the height of every structural system, the necessary concrete volume per $m^2$ area increases linearly in the building, meaning that the total concrete volume increases exponentially with building height.

The analyses show a big difference in the dynamic response between the different structural systems. The CLT system has higher horizontal stiffness and modal mass than the column building. It is shown that the CLT building requires less concrete than the Column building, however the CLT building uses more timber. The column system can also be more applicable as a commercial building such as office. In that case, ISO 10137 allows for higher accelerations and the concrete volume can be significantly reduced which is shown in figure 4.22. The structural system that requires the least concrete is the mixed building.

It can be concluded from the study that it is possible to build considerably higher timber-hybrid structures than what exists today. There was no limit to the possible building height found during the study, but at higher heights more concrete needs to be added for the extra stiffness in the structure. At some height the required concrete volume compared to timber volume may be too much, which prevent the building being called a timber-hybrid structure.

Tuned mass dampers can improve the response in timber and timber-hybrid buildings such that the concrete volume can be reduced by around 60%. The study also concludes that a tuned mass damper can be used to allow a CLT-T building to be built to at least 20 storeys (60 m) with only timber as structural material. No examples of high rise timber or timber-hybrid buildings with dampers were found in the literature study, however it does appear to be a viable solution in theory.

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# A

# Appendix 1

## A.1 Dead loads

Calculation of dead loads in the building, based on insulated wall and floor designs, and material densities.

*External wall*

| | Facade panel | | 28x95 battens | | 45x170 studs | | 170 mineral wool | | Total |
|---|---|---|---|---|---|---|---|---|---|
| ρ [kg/m$^3$] | 440 | ρ [kg/m$^3$] | 440 | ρ [kg/m$^3$] | 440 | ρ [kg/m$^3$] | 40 | ρ [kg/m$^2$] | 27.3 |
| t [mm] | 25 | h [mm] | 28 | h [mm] | 170 | t [mm] | 170 | | |
| ρ [kg/m$^2$] | 11 | b [mm] | 95 | b [mm] | 45 | ρ [kg/m$^2$] | 6.8 | | |
| | | cc [mm] | 300 | cc [mm] | 600 | | | | |
| | | ρ [kg/m$^2$] | 3.9 | ρ [kg/m$^2$] | 5.6 | | | | |

*Floor/Ceiling*

| | 45x170 studs | | 170 mineral wool | | 2x13 Gypsum board | | Total |
|---|---|---|---|---|---|---|---|
| ρ [kg/m$^3$] | 440 | ρ [kg/m$^3$] | 40 | ρ [kg/m$^3$] | 720 | ρ [kg/m$^2$] | 31.1 |
| h [mm] | 170 | t [mm] | 170 | t [mm] | 26 | | |
| b [mm] | 45 | ρ [kg/m$^2$] | 6.8 | ρ [kg/m$^2$] | 18.72 | | |
| cc [mm] | 600 | | | | | | |
| ρ [kg/m$^2$] | 5.61 | | | | | | |

*Apartment dividing walls*

| | 45x70 studs | | 70 mineral wool | | 2x13 Gypsum board | | Total | Total wall length | | |
|---|---|---|---|---|---|---|---|---|---|---|
| ρ [kg/m$^3$] | 440 | ρ [kg/m$^3$] | 40 | ρ [kg/m$^3$] | 720 | ρ [kg/m$^2$] | 47.7 | L | 88.2 | |
| h [mm] | 70 | t [mm] | 70 | t [mm] | 26 | | | h | 3 | |
| b [mm] | 45 | ρ [kg/m$^2$] | 2.8 | ρ [kg/m$^2$] | 18.72 | | | Area | 264.6 | |
| cc [mm] | 600 | | | | | | | weigth | 12611 | kg |
| ρ [kg/m$^2$] | 2.31 | | | | | | | load | 123712 | N |
| | | | | | | | | Area load | 0.27 | kN/m2' |

**Figure A.1:** Calculation of dead loads in the building

## A.2 TMD calculations

Calculation of tuned-mass damper effect, using a 2-dof damped mass-spring system.

```
clc
clear fig
clear all
%Indata
f1=0.352;                           %First eigenfrequency
M_s=2478807;                        %Modal mass [kg]
xsi_s=1.5;                          %Structural damping [%]
xsi_TMD=6.8;                        %Viscous damping TMD [%]
xsi_eq=5.7;                         %Eqiualent damping [%]
P=10000;                            %Force [N]
my=0.001;



%Beräkningar
n=2000;
r=linspace(0,2.5,n);
w1=2*pi*f1;                        %Angular frquency [rad/s]
Omega=r*w1;
K_s=M_s*w1^2;                      %Structural stiffness
U_0=P/K_s;                         %Static displacements
v_s=xsi_s/100*2*sqrt(K_s*M_s);
v_vd=xsi_eq/100*2*sqrt(K_s*M_s);   %Viscous dampingcoefficient


for i=1:n
Hd_s(i)=inv(-Omega(i).^2*M_s+1i*Omega(i)*v_s+K_s);
                                    %Transfer function
U_s(i)=Hd_s(i)*P;
D_s(i)=abs(U_s(i))/U_0;
end

%Addition of visciuos damper
for i=1:n
Hd_vd(i)=inv(-Omega(i).^2*M_s+1i*Omega(i)*v_vd+K_s);
U_vd(i)=Hd_vd(i)*P;
D_vd(i)=abs(U_vd(i))/U_0;
end
Dmax_vd=max(abs(D_vd));
Dmax_TMD=abs(max(D_s));
f_opt=0.981;
w_opt=f_opt*w1;

%Added TMD
while Dmax_TMD > Dmax_vd
m_TMD=my*M_s;
K_TMD=m_TMD*w_opt^2;
v_TMD=xsi_TMD/100*2*sqrt(K_TMD*m_TMD);
%v_TMD=2*xsi_TMD/100*w_opt*m_TMD
K_sys=[K_s+K_TMD -K_TMD;
        -K_TMD    K_TMD];
M_sys=[M_s 0;
        0  m_TMD];
V_sys=[v_s+v_TMD -v_TMD;
        -v_TMD    v_TMD];

for i=1:n
Hd_TMD=inv(-Omega(i).^2*M_sys+1i*Omega(i)*V_sys+K_sys);
U_TMD=Hd_TMD(1,1)*P;
D_TMD(i)=abs(U_TMD(1,1))/U_0;
end
f_opt_connor=sqrt(1-0.5*my)/(1+my);
Dmax_TMD=max(abs(D_TMD));
my=my+0.00001;
if my>0.2
brake
```

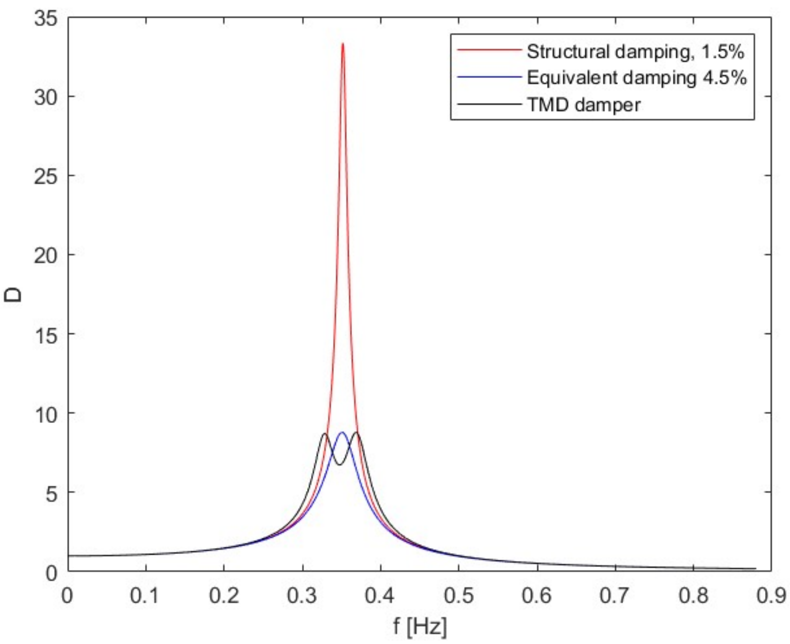
```
end
end

D1_max=max(abs(D_s));
D2_max=max(abs(D_vd));
D3_max=max(abs(D_TMD));

Dratio1=D2_max/D1_max
Dratio2=D3_max/D1_max

figure(1)
clf
plot(Omega/2/pi,D_s,'r',Omega/2/pi,D_vd,'b',Omega/2/pi,D_TMD,'black')
set(gcf,'color','w');
legend('Structural damping, 1.5%','Equivalent damping 4.5%','TMD damper')
xlabel('f [Hz]')
ylabel('D')

List=[f1 ; f_opt; xsi_eq; xsi_TMD ; m_TMD/1000]
```



## A.3 Material data (Martinssons)

Tables of strength properties and span length for CLT-elements according Martinsons, n.d.

### Spännviddstabell bjälklag [1)]

KL-Träskivor Format: 3000 x längd x tjocklek [mm]

| Lasttyp[2)] | Kategori A (Bostäder) 2,0 kN/m² | | | | Kategori B (Kontor) 2,5 kN/m² | |
|---|---|---|---|---|---|---|
| Skiva[3)] | Max spännvidd[4)] | Deformation[6)] | Max spännvidd[5)] | Deformation[6)] | Max spännvidd[4)] | Deformation[6)] |
| L60-3s | 2,3 | L/567 | 2,0 | L/862 | 2,2 | L/582 |
| L70-3s | 2,6 | L/579 | 2,6 | L/579 | 2,5 | L/587 |
| L80-3s | 3,1 | L/529 | 3,0 | L/583 | 2,9 | L/584 |
| L90-3s | 3,4 | L/543 | 3,4 | L/543 | 3,2 | L/591 |
| L100-3s | 3,7 | L/547 | 3,7 | L/547 | 3,5 | L/588 |
| L120-3s | 4,5 | L/513 | 4,3 | L/588 | 4,2 | L/576 |
| L140-3s | 5,1 | L/526 | 4,7 | L/674 | 4,9 | L/546 |
| | | | | | | |
| L100-5s | 3,5 | L/646 | 3,5 | L/646 | 3,4 | L/641 |
| L120-5s | 4,0 | L/730 | 3,9 | L/788 | 3,8 | L/778 |
| L130-5s | 4,6 | L/531 | 4,4 | L/607 | 4,4 | L/555 |
| L140-5s | 4,5 | L/520 | 4,3 | L/596 | 4,3 | L/546 |
| L150-5s | 5,2 | L/498 | 4,6 | L/720 | 4,9 | L/548 |
| L160-5s | 5,7 | L/502 | 5,0 | L/746 | 5,5 | L/515 |
| L180-5s | 5,7 | L/547 | 5,0 | L/812 | 5,6 | L/533 |
| L200-5s | 6,3 | L/592 | 5,6 | L/845 | 6,3 | L/549 |
| L230-5s | 6,8 | L/674 | 6,0 | L/983 | 6,8 | L/627 |
| | | | | | | |
| L170-7s | 5,0 | L/612 | 4,4 | L/898 | 5,0 | L/565 |
| L210-7s | 6,3 | L/605 | 5,6 | L/863 | 6,3 | L/562 |
| L240-7s | 7,1 | L/707 | 6,3 | L/1000 | 7,1 | L/659 |
| L270-7s | 7,4 | L/781 | 6,5 | L/1000 | 7,4 | L/731 |
| L280-7s | 7,4 | L/773 | 6,6 | L/1000 | 7,4 | L/724 |

### **KL-trä** Konstruktionsfakta

STYVHETSVÄRDEN FÖR DEFORMATIONSBERÄKNINGAR (50 %-FRAKTILEN)

| Tjocklek [mm][2)] L / T[3)] | E-modul, böjning kring i-axeln, $E^{mi,50}$ | | | | E-modul, drag // i-axeln $E^{ti,50}$ | | | E-modul, tryck // i-axeln $E^{ci,50}$ | | | Skjuvmodul, $G^{ij,50}$ | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Z Styv | X Vek | Y1 Skivv.styv | Y2 Skivv.vek | X Styv | Z Vek | Y Tvärdrag | X Styv | Z Vek | Y Fibrerna | XY Böj.styv | YZ Böj.vek | XZ Böj.skiv |
| 60-3s | 10606 | 764 | 7457 | 3913 | 7457 | 3913 | 370 | | | | 117 | 96 | 690 |
| 70-3s | 10152 | 892 | 6384 | 3211 | 6384 | 3211 | 310 | | | | 95 | 108 | 582 |
| 80-3s | 10832 | 474 | 8308 | 2028 | 2028 | 2028 | 335 | | | | 148 | 87 | 627 |
| 90-3s | 10601 | 616 | 7410 | 2580 | 7410 | 2580 | 323 | Samma värden som för drag. | | | 117 | 96 | 606 |
| 100-3s | 10311 | 794 | 6692 | 3022 | 6692 | 3022 | 314 | | | | 100 | 104 | 590 |
| 120-3s | 10601 | 616 | 7410 | 2580 | 7410 | 2058 | 323 | | | | 117 | 96 | 606 |
| 140-3s | 10601 | 616 | 7410 | 2580 | 7410 | 2058 | 323 | | | | 117 | 96 | 606 |
| | | | | | | | | | | | | | |
| 100-5s | 8760 | 1749 | 6692 | 3022 | 6692 | 3022 | 314 | | | | 100 | 104 | 590 |
| 120-5s | 7859 | 2304 | 5615 | 3685 | 5615 | 3685 | 300 | | | | 83 | 119 | 565 |
| 130-5s | 9451 | 1324 | 7686 | 2410 | 7686 | 2410 | 327 | | | | 125 | 93 | 613 |
| 140-5s | 7106 | 2767 | 4846 | 4159 | 4846 | 4159 | 290 | | | | 73 | 133 | 547 |
| 150-5s | 8760 | 1749 | 6692 | 3022 | 6692 | 3022 | 314 | Samma värden som för drag. | | | 100 | 104 | 590 |
| 160-5s | 9822 | 1095 | 8308 | 2028 | 8308 | 2028 | 335 | | | | 148 | 87 | 627 |
| 180-5s | 7859 | 2304 | 5615 | 3685 | 5615 | 3685 | 300 | | | | 83 | 119 | 565 |
| 200-5s | 8760 | 1749 | 6692 | 3022 | 6692 | 3022 | 314 | | | | 100 | 104 | 590 |
| 230-5s | 8760 | 1749 | 6692 | 3022 | 6692 | 3022 | 314 | | | | 100 | 104 | 590 |
| | | | | | | | | | | | | | |
| 170-7s | 6 877 | 2 908 | 5 298 | 3 880 | 5 298 | 3 880 | 295 | | | | 79 | 125 | 557 |
| 210-7s | 7 891 | 2 284 | 6 384 | 3 211 | 6 384 | 3 211 | 310 | | | | 95 | 108 | 582 |
| 240-7s | 9 401 | 1 354 | 8 308 | 2 028 | 8 308 | 2 028 | 335 | Samma värden som för drag. | | | 148 | 87 | 627 |
| 270-7s | 8 740 | 1 761 | 7 410 | 2 580 | 7 410 | 2 580 | 323 | | | | 117 | 96 | 606 |
| 280-7s | 7 891 | 2 284 | 6 384 | 3 211 | 6 384 | 3 211 | 310 | | | | 95 | 108 | 582 |

**Karakteristiska styvhets- och hållfasthetsvärden (MPa)**
KL-trä Martinsons standardtjocklekar

Material: 1)

C24 till bitar i styv riktning
C14 till bitar i vek riktning

## A.4 Concrete and Timber volume estimation.

Estimations of concrete and timber volume in a selection of existing high rise timber-hybrid buildings. The estimations are based on existing drawings and published information about the buildings.

Brocks common tall Wood building

$h := 54 \cdot m \qquad A_{floor} := 840 \cdot m^2 \qquad n_{storeys} := 18 \qquad A_{tot} := A_{floor} \cdot n_{storeys} = \left(1.512 \cdot 10^4\right)\ m^2$

$t_{core} := 0.45 \cdot m \qquad l_{core} := 67 \cdot m$

$t_{topping} := 0.04 \cdot m$ Concrete topping on floors

$t_{floor} := 0.6 \cdot m$ Concrete slab at first storey

$n_{columns} := 40 \qquad A_{columns} := 0.4 \cdot m \cdot 0.4 \cdot m = 0.16\ m^2 \qquad h_{columns} := 3 \cdot m$

$V_{timber} := 2233\ m^3$

Concrete volume calculations

$V_{core} := t_{core} \cdot l_{core} \cdot h = \left(1.628 \cdot 10^3\right)\ m^3$

$V_{topping} := t_{topping} \cdot A_{floor} \cdot n_{storeys} = 604.8\ m^3$

$V_{floor} := t_{floor} \cdot A_{floor} = 504\ m^3$

$V_{col} := A_{columns} \cdot h_{columns} \cdot n_{columns} = 19.2\ m^3$

$V_{concrete.tot} := V_{topping} + V_{core} + V_{floor} + V_{col} = \left(2.756 \cdot 10^3\right)\ m^3$

$$\frac{V_{concrete.tot}}{A_{tot}} = 0.182\ \frac{m^3}{m^2}$$

$$\frac{V_{timber}}{A_{tot}} = 0.148\ \frac{m^3}{m^2}$$

**Figure A.2:** Concrete and timber volume estimation for Brooks common tallwood house.

Mjöstårnet

$h := 88.8 \cdot m \qquad A_{floor} := 36.8 \cdot m \cdot 16.3 \cdot m = 599.84\ m^2$

$n_{storeys} := 18 \qquad A_{tot.floor} := A_{floor} \cdot n_{storeys} = \left(1.08 \cdot 10^4\right)\ m^2$

$V_{concrete.slabs} := 1100\ m^3 \qquad V_{timber} := 2685\ m^3$

$$\frac{V_{concrete.slabs}}{A_{tot.floor}} = 0.102\ \frac{m^3}{m^2}$$

$$\frac{V_{timber}}{A_{tot.floor}} = 0.249\ \frac{m^3}{m^2}$$

**Figure A.3:** Concrete and timber volume estimation for Mjöstornet.

The Ascent

$h := 86 \cdot m \quad h_{storey} := 3.4\ m \quad n_{concrete.storeys} := 6 \quad n_{timber.storeys} := 19$

$A_{concrete.storeys} := 2280\ m^2 \quad A_{timber.storeys} := 1504\ m^2$

$t_{concrete.slabs} := 0.19 \cdot m \quad t_{topping.timber.storeys} := 0.05 \cdot m$

$n_{col.concrete.storeys} := 60 \quad A_{columns} := 0.16\ m^2$

$t_{core} := 0.4\ m \quad l_{core} := 48\ m$

$A_{tot} := n_{concrete.storeys} \cdot A_{concrete.storeys} + n_{timber.storeys} \cdot A_{timber.storeys} = \left(4.226 \cdot 10^4\right)\ m^2$

$V_{timber} := 2200 \cdot m^3$

Concrete volume calculations

$V_{slabs} := A_{concrete.storeys} \cdot n_{concrete.storeys} \cdot t_{concrete.slabs} = \left(2.599 \cdot 10^3\right)\ m^3$

$V_{topping} := A_{timber.storeys} \cdot n_{timber.storeys} \cdot t_{topping.timber.storeys} = \left(1.429 \cdot 10^3\right)\ m^3$

$V_{columns} := A_{columns} \cdot n_{col.concrete.storeys} \cdot h_{storey} \cdot n_{concrete.storeys} = 195.84\ m^3$

$V_{core} := t_{core} \cdot l_{core} \cdot h = \left(1.651 \cdot 10^3\right)\ m^3$

$V_{tot.concrete} := V_{slabs} + V_{topping} + V_{columns} + V_{core} = \left(5.875 \cdot 10^3\right)\ m^3$

$$\frac{V_{tot.concrete}}{A_{tot}} = 0.139\ \frac{m^3}{m^2}$$

$$\frac{V_{timber}}{A_{tot}} = 0.052\ \frac{m^3}{m^2}$$

**Figure A.4:** Concrete and timber volume estimation for The Ascent.

# A.5 Verifications.

## A.5.1 Vertical load verification.

A vertical load verification is carried out and compared to the support reactions of the FEM-results.

| Type | rho kg/m3 | thickness [m] | Self Weight [kg/m2 | Load [N/m2] |
|---|---|---|---|---|
| IW 210-s7 | 400 | 0.21 | 84 | 824.04 |
| 240-s7 | 400 | 0.24 | 96 | 941.76 |
| 270-s7 | 400 | 0.27 | 108 | 1059.48 |
| C30/37 | 2400 | 0.3 | 720 | 7063.2 |
| Facade | - | - | | 300 |
| Ceiling | - | - | | 300 |
| Live load | - | - | - | 600 |

**Figure A.5:** Load per $m^2$ element.

| Area [m2/storey] | |
|---|---|
| Facade/OW | 209.76 |
| Cieling | 435.06 |
| Floor(core) | 13.86 |
| Floor | 421.2 |
| Outer wall | 209.76 |
| Inner Walls | 265.2 |
| Core wall | 118.2 |

**Figure A.6:** Total $m^2$ element per storey.

| Type | Area [$m^2$] | Load [N/m2] | Total load [N] | Load [kN] | FEM | |
|---|---|---|---|---|---|---|
| Facade | 209.76 | 300 | 62928 | 62.9 | - | |
| Cieling | 435.06 | 300 | 130518 | 130.5 | - | |
| Floor(core) | 13.86 | 1659.48 | 23000 | 23.0 | - | |
| Floor | 421.2 | 1659.48 | 698973 | 699.0 | - | |
| Outer wall | 209.76 | 941.76 | 197544 | 197.5 | - | |
| Inner Walls | 265.2 | 941.76 | 249755 | 249.8 | - | |
| Core wall | 118.2 | 941.76 | 111316 | 111.3 | - | Diff. |
| | | ∑: | 1474034 | 1474.0 | 1465.3 | 0.6% |

**Figure A.7:** Summation and comparison of loads, hand calculations vs FEM.

## A.5.2 Frequency verification.

A frequency and mode shape verification is carried out and compared to FEM-results.

| Wall length [mm] | Wall length [m] | Wall width [m] | Area [m^2] | h [m] | E_mean [Pa] | G_mean [Pa] | I [m4] | $S_{si}$ | $S_{bi}$ | $1/S_i$ | $S_i$ | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 800 | 0.8 | 0.24 | 0.192 | 3 | 2.03E+09 | 6.27E+08 | 0.01024 | 4.01E+07 | 2.08E+07 | 7.31E-08 | 1.37E+07 | |
| 1500 | 1.5 | 0.24 | 0.36 | 3 | 2.03E+09 | 6.27E+08 | 0.0675 | 7.52E+07 | 1.37E+08 | 2.06E-08 | 4.86E+07 | |
| 1800 | 1.8 | 0.24 | 0.432 | 3 | 2.03E+09 | 6.27E+08 | 0.11664 | 9.03E+07 | 2.37E+08 | 1.53E-08 | 6.53E+07 | |
| 1500 | 1.5 | 0.24 | 0.36 | 3 | 2.03E+09 | 6.27E+08 | 0.0675 | 7.52E+07 | 1.37E+08 | 2.06E-08 | 4.86E+07 | |
| 1500 | 1.5 | 0.24 | 0.36 | 3 | 2.03E+09 | 6.27E+08 | 0.0675 | 7.52E+07 | 1.37E+08 | 2.06E-08 | 4.86E+07 | |
| 1100 | 1.1 | 0.24 | 0.264 | 3 | 2.03E+09 | 6.27E+08 | 0.02662 | 5.52E+07 | 5.40E+07 | 3.66E-08 | 2.73E+07 | |
| 1400 | 1.4 | 0.24 | 0.336 | 3 | 2.03E+09 | 6.27E+08 | 0.05488 | 7.02E+07 | 1.11E+08 | 2.32E-08 | 4.31E+07 | |
| 800 | 0.8 | 0.24 | 0.192 | 3 | 2.03E+09 | 6.27E+08 | 0.01024 | 4.01E+07 | 2.08E+07 | 7.31E-08 | 1.37E+07 | |
| 6200 | 6.2 | 0.24 | 1.488 | 3 | 2.03E+09 | 6.27E+08 | 4.76656 | 3.11E+08 | 9.67E+09 | 3.32E-09 | 3.01E+08 | |
| 7400 | 7.4 | 0.24 | 1.776 | 3 | 2.03E+09 | 6.27E+08 | 8.10448 | 3.71E+08 | 1.64E+10 | 2.75E-09 | 3.63E+08 | |
| 6200 | 6.2 | 0.24 | 1.488 | 3 | 2.03E+09 | 6.27E+08 | 4.76656 | 3.11E+08 | 9.67E+09 | 3.32E-09 | 3.01E+08 | |
| 800 | 0.8 | 0.24 | 0.192 | 3 | 2.03E+09 | 6.27E+08 | 0.01024 | 4.01E+07 | 2.08E+07 | 7.31E-08 | 1.37E+07 | |
| 1500 | 1.5 | 0.24 | 0.36 | 3 | 2.03E+09 | 6.27E+08 | 0.0675 | 7.52E+07 | 1.37E+08 | 2.06E-08 | 4.86E+07 | |
| 1800 | 1.8 | 0.24 | 0.432 | 3 | 2.03E+09 | 6.27E+08 | 0.11664 | 9.03E+07 | 2.37E+08 | 1.53E-08 | 6.53E+07 | |
| 1500 | 1.5 | 0.24 | 0.36 | 3 | 2.03E+09 | 6.27E+08 | 0.0675 | 7.52E+07 | 1.37E+08 | 2.06E-08 | 4.86E+07 | |
| 1500 | 1.5 | 0.24 | 0.36 | 3 | 2.03E+09 | 6.27E+08 | 0.0675 | 7.52E+07 | 1.37E+08 | 2.06E-08 | 4.86E+07 | |
| 1100 | 1.1 | 0.24 | 0.264 | 3 | 2.03E+09 | 6.27E+08 | 0.02662 | 5.52E+07 | 5.40E+07 | 3.66E-08 | 2.73E+07 | |
| 1400 | 1.4 | 0.24 | 0.336 | 3 | 2.03E+09 | 6.27E+08 | 0.05488 | 7.02E+07 | 1.11E+08 | 2.32E-08 | 4.31E+07 | |
| 800 | 0.8 | 0.24 | 0.192 | 3 | 2.03E+09 | 6.27E+08 | 0.01024 | 4.01E+07 | 2.08E+07 | 7.31E-08 | 1.37E+07 | |
| 6200 | 6.2 | 0.24 | 1.488 | 3 | 2.03E+09 | 6.27E+08 | 4.76656 | 3.11E+08 | 9.67E+09 | 3.32E-09 | 3.01E+08 | |
| 7400 | 7.4 | 0.24 | 1.776 | 3 | 2.03E+09 | 6.27E+08 | 8.10448 | 3.71E+08 | 1.64E+10 | 2.75E-09 | 3.63E+08 | |
| 6200 | 6.2 | 0.24 | 1.488 | 3 | 2.03E+09 | 6.27E+08 | 4.76656 | 3.11E+08 | 9.67E+09 | 3.32E-09 | 3.01E+08 | |
| 5100 | 5.1 | 0.24 | 1.224 | 3 | 2.03E+09 | 6.27E+08 | 2.65302 | 2.56E+08 | 5.38E+09 | 4.09E-09 | 2.44E+08 | |
| 2900 | 2.9 | 0.24 | 0.696 | 3 | 2.03E+09 | 6.27E+08 | 0.48778 | 1.45E+08 | 9.89E+08 | 7.89E-09 | 1.27E+08 | |
| 700 | 0.7 | 0.24 | 0.168 | 3 | 2.03E+09 | 6.27E+08 | 0.00686 | 3.51E+07 | 1.39E+07 | 1.00E-07 | 9.96E+06 | |
| | | | | | | | | | | $\Sigma S_i$ | 2.93E+09 | N/m |

**Figure A.8:** Summation of stiffness used for stiffness matrix.

$m := 101 \cdot 10^3\ kg \qquad m_{walls} := 53.98 \cdot 10^3\ kg$

$S_i := 2.93 \cdot 10^9\ \frac{N}{m}$ From Excel

$k := S_i$

$n_{floors} := 10$

$i := 0..n_{floors}-1 \qquad ii := 0..n_{floors}-2 \qquad K_{n_{floors}-1,\, n_{floors}-1} := 1$

$j := 0..n_{floors}-1$

$K_{i,j} := 0 \qquad K_{i,i} := 2 \qquad K_{ii+1,\, ii} := -1 \qquad K_{ii,\, ii+1} := -1 \qquad K_{0,0} := 1$

$M_{i,j} := 0 \qquad M_{i,i} := 1$

$K := k \cdot K \qquad M := m \cdot M$

$M_{0,0} := M_{0,0} - 0.5 \cdot m_{walls} \qquad M_{n_{floors}-1,\, n_{floors}-1} := M_{n_{floors}-1,\, n_{floors}-1} - 0.5 \cdot m_{walls}$

$$K = \begin{bmatrix} 2.93 \cdot 10^9 & -2.93 \cdot 10^9 & 0 & 0 & 0 & \\ -2.93 \cdot 10^9 & 5.86 \cdot 10^9 & -2.93 \cdot 10^9 & 0 & 0 & \\ 0 & -2.93 \cdot 10^9 & 5.86 \cdot 10^9 & -2.93 \cdot 10^9 & 0 & \\ 0 & 0 & -2.93 \cdot 10^9 & 5.86 \cdot 10^9 & -2.93 \cdot 10^9 & \\ 0 & 0 & 0 & -2.93 \cdot 10^9 & 5.86 \cdot 10^9 & \\ 0 & 0 & 0 & 0 & -2.93 \cdot 10^9 & \\ 0 & 0 & 0 & 0 & 0 & \\ 0 & 0 & 0 & 0 & 0 & \\ 0 & 0 & 0 & 0 & 0 & \\ & & & & & \ddots \end{bmatrix} \frac{N}{m}$$

$$M = \begin{bmatrix} 7.401 \cdot 10^4 & 0 & 0 & 0 & 0 & 0 & \\ 0 & 1.01 \cdot 10^5 & 0 & 0 & 0 & 0 & \\ 0 & 0 & 1.01 \cdot 10^5 & 0 & 0 & 0 & \\ 0 & 0 & 0 & 1.01 \cdot 10^5 & 0 & 0 & \\ 0 & 0 & 0 & 0 & 1.01 \cdot 10^5 & 0 & \\ 0 & 0 & 0 & 0 & 0 & 1.01 \cdot 10^5 & \\ 0 & 0 & 0 & 0 & 0 & 0 & \\ 0 & 0 & 0 & 0 & 0 & 0 & \\ & & & & & & \ddots \end{bmatrix} kg$$

$M_iK := M^{-1} \cdot K$

**Figure A.9:** Calculation of mass and stiffness matrices.

$$w := \frac{\sqrt{\text{eigenvals}(M_iK)}}{2 \cdot \pi} = \begin{bmatrix} 54.161 \\ 52.882 \\ 50.321 \\ 46.47 \\ 41.397 \\ 35.243 \\ 28.205 \\ 20.517 \\ 12.431 \\ 4.16 \end{bmatrix} \frac{1}{s} \qquad w2 := \text{eigenvals}(M_iK) = \begin{bmatrix} 1.158 \cdot 10^5 \\ 1.104 \cdot 10^5 \\ 9.997 \cdot 10^4 \\ 8.525 \cdot 10^4 \\ 6.766 \cdot 10^4 \\ 4.904 \cdot 10^4 \\ 3.14 \cdot 10^4 \\ 1.662 \cdot 10^4 \\ 6.101 \cdot 10^3 \\ 683.036 \end{bmatrix} \frac{1}{s^2}$$

$$\text{eigenvec}\left(M_iK, w2_9\right) = \begin{bmatrix} -0.431 \\ -0.423 \\ -0.406 \\ -0.379 \\ -0.343 \\ -0.299 \\ -0.248 \\ -0.191 \\ -0.13 \\ -0.066 \end{bmatrix}$$

**Figure A.10:** calculation of eigenvector and eigenvalues.

## A.6 Modal acceleration calculation.

Calculations of peak and r.m.s accelerations using EKS 12 and Eurocode. Orange cells are varying values depending on results in FEM-Design.

| | | |
|---|---|---|
| $a=k_p*\sigma(z)$ | $(z=h_{building}-1*h_{storey})$ | |
| **Building properties** | | |
| Storey h | 3 | m |
| Storyes | 14 | [n] |
| h= | 42 | m |
| b= | 21.6 | m |
| d= | 21.6 | m |
| ξ= | 0.015 | - |
| n_1= | 2.106 | |
| $m_e$= | 46100.5 | kg |
| | | |
| **$k_p$:** | | |
| v= | 0.31758207 | \geq 0,08 |
| T= | 600 | s |
| $k_p$= | 3.42550837 | \geq 3 |
| | | |
| **$I_v(h)=1/(c0(h)*ln(h/z0))$** | | |
| c0= | 1 | (flat terrain assumed) |
| Iv(h)= | 0.20236187 | |
| | | |
| **qm(h):** | | |
| qm(h)= | 323.506696 | N/m2 |
| | | |
| **Φ(z)** | | |
| Φ(z) | 0.96294152 | |
| | | |
| **σ(z):** | | |
| σ(z)= | | |
| | | |
| **$v_{5year}$ for 5 years acceleration (EKS12):** | | |
| $v_b= v_{50}$= | 25 | m/s |
| $T_a$= | 5 | years |
| $v_5$= | 21.4 | m/s |
| | | |
| **$v_{2year}$ for 1 year acceleration (EKS12):** | | |
| $T_a$= | 2 | years |
| $v_2$= | 19.4 | m/s |
| | | |
| **$\sigma_x=(3*I_v(h)*R*q_m*b*c_f*\Phi(z))/m_e$** | | |
| **$\sigma_{x,5}$** | 0.01672479 | [m/s²] |
| **$\sigma_{x,1}$** | 0.01204 | |
| | | |
| **$X_{max}=k_p*\sigma_x$** | | |
| **$X_{max,1}$** | 0.04124946 | [m/s^2] |

| | | | | | |
|---|---|---|---|---|---|
| **$R^2$=2pi*F*Φb*Φh/(ds+da), $B^2$=...** | | | | | |
| **F:** | | | | | |
| | **yc:** | | | | |
| | | **vm(h):** | | | |
| | | c0(h)= | 1 | | |
| | | | **cr(h):** | | |
| | | | zmin(III)= | 5 m | |
| | | | zmax= | 200 m | |
| | | | z0(III)= | 0.3 m | |
| | | | z0(II)= | 0.05 m | |
| | | | kr= | 0.215 | |
| | | | cr(h)= | 1.06 | |
| | | vm(h)= | 22.75 m/s | | *(5 years)* |
| | yc= | 13.89 | | | |
| F= | 0.0199 | | | | |
| Φh | 0.1140 | | | | |
| Φb | 0.1352 | | | | |
| **da:** | | | | | |
| ρ= | 1.25 kg/m³ | | | | |
| | **cf:** | | | | |
| | $c_{pe10D}$= | 0.80 | | | |
| | $c_{pe10E}$= | 0.55 | | | |
| | $c_{petot}$= | 1.35 | | | |
| | cf= | 1.35 | | | |
| | **vm(zs):** | | | | |
| | $z_s$= | 25.2 | | | |
| | cr(zs)= | 0.95 | | | |
| | vm(zs)= | 20.40 m/s | | *(5 years)* | |
| da= | 0.004 | | | | |
| ds= | 0.09 | | | | |
| $R^2$= | 0.01963 | R= | 0.140101 | | |
| **$B^2$:** | | | | | |
| href= | 10 m | | | | |
| $B^2$= | 0.84 | | | | |

**Figure A.11:** Calculations of peak and r.m.s accelerations using EKS 12 and Eurocode.

## A.7 Optimized results.

A table of the most concrete volume optimized buildings. The buildings are categorized by structural systems and building heights.

| *Type:* | $N_{Storeys}$ | Height [m] | Tot. floor area [$m^2$] | $f_1$ [Hz] | $m_e$ [kg/m] | $X''_{max}$ [$m/s^2$] | $X''_{ISO10137}$ [$m/s^2$] | % (ISO10137) |
|---|---|---|---|---|---|---|---|---|
| *Col-CF (12, 12/200, 1/400)* | 12 | 36 | 5703 | 1.381 | 77772 | 0.038 | 0.040 | 95% |
| *Col-C.office (12, 12/200)* | 12 | 36 | 5703 | 0.859 | 81514 | 0.063 | 0.064 | 98% |
| *Col-C (14, 14/440)* | 14 | 42 | 6653 | 1.667 | 64433 | 0.038 | 0.040 | 95% |
| *CLT-T (14)* | 14 | 42 | 6653 | 2.103 | 45997 | 0.041 | 0.042 | 96% |
| *Mix-T (14)* | 14 | 42 | 6653 | | | | | |
| *CLT-C (16, 6/200)* | 16 | 48 | 7604 | 2.11 | 71790 | 0.041 | 0.042 | 97% |
| *Col-CF (16, 16/440, 1/480)* | 16 | 48 | 7604 | 1.106 | 143580 | 0.039 | 0.040 | 98% |
| *Col-CF.office (16, 16/200, 2/280)* | 16 | 48 | 7604 | 0.859 | 81514 | 0.063 | 0.064 | 98% |
| *Mix-F (16, 5/120)* | 16 | 48 | 7604 | 1.764 | 40621 | 0.057 | 0.060 | 95% |
| *CLT* | 18 | 54 | 8554 | 1.538 | 71686 | 0.039 | 0.04 | 97% |
| *Col-CF (18, 18/440, 3/440)* | 18 | 54 | 8554 | 0.781 | 140833 | 0.042 | 0.045 | 93% |
| *Mix-F (18, 5/120)* | 18 | 54 | 8554 | 1.251 | 64405 | 0.055 | 0.06 | 92% |
| *CLT-CF (20, 9/440, 2/440)* | 20 | 60 | 9505 | 1.22 | 94188 | 0.039 | 0.040 | 98% |
| *Col-CF (20, 20/400, 4/400)* | 20 | 60 | 9505 | 0.604 | 157224 | 0.049 | 0.050 | 99% |
| *Col-CF.office (20, 20/320, 4/240)* | 20 | 60 | 9505 | 0.634 | 101066 | 0.072 | 0.073 | 98% |
| *Mix-F (18, 5/120)* | 20 | 60 | 9505 | 1.043 | 73830 | 0.059 | 0.060 | 99% |
| *Mix-CF (25, 25/200, 5/200)* | 25 | 75 | 11881 | 0.809 | 93348 | 0.063 | 0.066 | 95% |
| *Col-CF (25, 25/500, 10/440)* | 25 | 75 | 11881 | 0.0404 | 197104 | 0.060 | 0.060 | 99.6% |
| *CLT-CF (25 25/400, 5/400)* | 25 | 75 | 11881 | 0.715 | 149321 | 0.045 | 0.046 | 98% |
| *CLT-CF (30, 30/520, 7/480)* | 30 | 90 | 14257 | 0.492 | 188557 | 0.054 | 0.055 | 98% |
| *Mix-CF (30, 30/300, 5/240)* | 30 | 90 | 14257 | 0.552 | 118732 | 0.075 | 0.078 | 96% |
| *Col-CF (30, 30/600, 13/480)* | 30 | 90 | 14257 | 0.292 | 230692 | 0.069 | 0.069 | 100% |
| *Col-CF.office (30, 30/480, 5/440)* | 30 | 90 | 14257 | 0.315 | 147251 | 0.100 | 0.100 | 100% |
| *CLT-CF (35, 35/600, 14/440)* | 35 | 105 | 16633 | 0.359 | 221655 | 0.062 | 0.063 | 99% |
| *Mix-CF (35, 35/400, 10/400)* | 35 | 105 | 16633 | 0.38 | 174421 | 0.061 | 0.062 | 99.5% |
| *CLT-CF (40, 40/800, 18/480)* | 40 | 120 | 19010 | 0.27 | 253943 | 0.070 | 0.071 | 98% |
| *Mix-CF (40, 40/500, 15/400)* | 40 | 120 | 19010 | 0.29 | 198086 | 0.069 | 0.069 | 100% |

**Figure A.12:** Optimized results of each building type.