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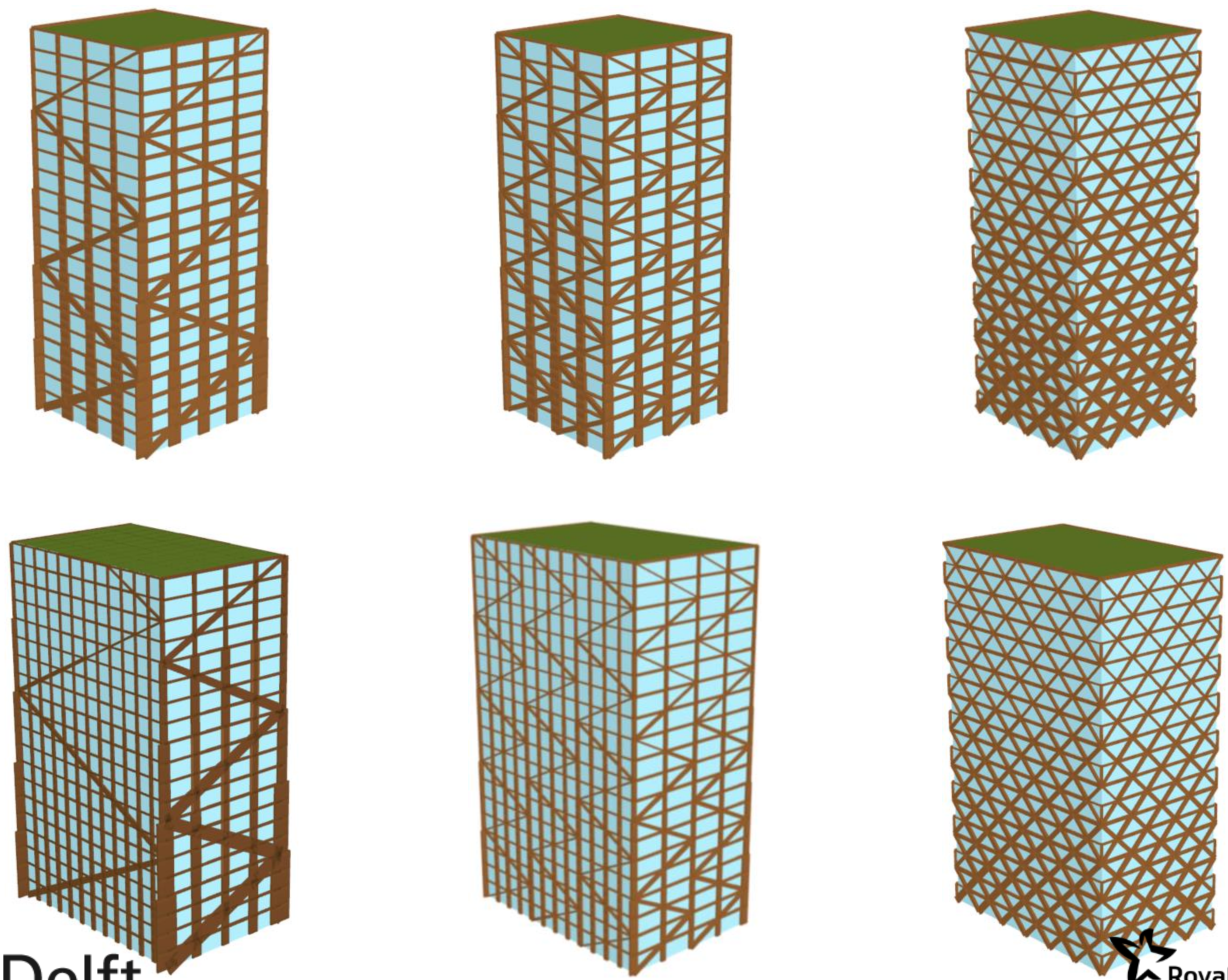
A parametric study of externally braced timber stability systems considering the connection design

Sophie van de Leur



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by

S.F.C. van de Leur

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Preface

This report presents my graduation thesis to finish the masters degree of Building Engineering at Delft University of Technology. During this research I was able to combine my interests in sustainability and structural engineering, which is not always an obvious combination. I want to express my gratitude to RHDHV and Walter van Adrichem for sharing my interests and allowing me to research my preferred topic at their company. I also want to thank the other colleagues at RHDHV who helped me a lot with their expertise in modelling and keeping up my spirits during the project.

The support of the rest of the graduation committee including Geert Ravenshorst, Chris Noteboom, Maria Felicita and Roy Crielaard has also been a vital part in creating this thesis. Your help and feedback was much appreciated and I want to thank you all for sharing your knowledge with me and also for providing me with personal guidance when needed.

I also want to thank my friends. First, I want to thank my fellow students for helping me during my masters. Most of our masters have been during the corona pandemic and having friends available to share our knowledge and still be able to experience our studies together has been a lot of fun. Secondly, I want to thank my roommates. You have always given me a listening ear, cared for me during busy times and provided me with the sometimes much needed relaxation.

I would also like to thank my family. You have always been my biggest supporters during my studies. Although, I have taken a while longer to finish you have always believed in my capabilities as an engineer and pushed me to keep going. Without you I would have never been able to accomplish my goals and for that I'm eternally grateful. Lastly, Harm deserves a special thanks. Thanks for always keeping my life lighthearted and providing me with help and encouragement no matter the question or situation.

Sophie van de Leur
Delft, January 2023

Abstract

Bio-based materials like timber have been gaining in popularity due to sustainability reasons. Not only for low-rise buildings but also for high-rise buildings. Providing enough structural stability with timber can be challenging. To compete with traditional materials the timber stability system must be optimized to reduce the material usage. From literature the externally braced stability systems were concluded to be the most efficient for timber high-rise. The externally braced systems are the external braced frame and diagrid stability system. According to various sources the diagrid systems are the most efficient. Nonetheless, in practice only external braced frame systems are used for timber high-rise buildings. This disconnect could be caused by the steel connections which are often times simplified or overlooked in literature studies but have a large influence on the material usage of a timber stability system. Consequently, the following main research question is studied to optimize the material efficiency of timber high-rise stability systems:

How can different design choices influence the material efficiency of an externally braced timber stability system for high-rise buildings, based on an integral comparison considering both connections and timber elements?

To research this topic a 3D parametric model is made in grasshopper where the different design choices are tested. To determine what is studied in the parametric model first the most significant design choices according to literature are defined. These choices are the connection design, timber element size, the slenderness of the building, the floor weight, the floor span and the angle of the bracing. It is concluded that slotted-in steel plate connections are the most suitable for externally braced timber systems and will be used in this research. The connection design will not be a parameter so an exploratory study is performed where most connection parameters are fixed. The only parameters that are not fixed are the timber element size and number of rows in the connection. The other design choices are also defined. The timber element widths that are studied range from 400 mm to 650 mm. The slendernesses considered are 1.67 and 2.5, with a building height of 68 m and plot sizes of 27.2 x 27.2 m and 27.2 x 40.8 m. Three different floor weights ranging from 3.5 kN/m^2 to 6.7 kN/m^2 and two different floor spans of 3.4 m and 6.8 m are defined. The last design choice of the angle of the bracing is included in the stability system designs. In an exploratory study where 2D models are researched, three stability system designs are determined. One diagrid design with a top angle of 90° and a ring beam every floor and two external braced frame designs. One external braced frame design has a single brace with slope 1:2 and the other has a double brace with slope 1:2.

The parametric model will create preliminary designs for all the combinations of the parameters. In the parametric model first the ULS element checks are performed where every timber element and connection is sized individually. These checks include a regular ULS member check, a fire member check and the connection design component. In the fire member check the reduced cross-section method is used to account for fire safety. The connection design component can increase the number of rows used in the connection and it can increase the timber element heights to increase the connection capacity. The component also calculates the axial stiffnesses of the connections which is included in the parametric model. After the ULS checks the model is sized based on the SLS checks including the global displacement and the along-wind acceleration. When the SLS requirements are not met the global stiffness of the stability system will be increased by increasing the timber element heights in groups. In the exploratory study on the design for SLS it is seen that increasing the connection stiffness can also be used to enhance the global stiffness. This is not included in the parametric model to reduce the scope of the research.

From the results it is seen that all buildings are sized on the connection design or on the along-wind acceleration. The diagrid designs have a higher global stiffness so they are sized more often on the connection design whereas the braced frame designs are sized more often on the along-wind acceleration. The diagrid designs use 3x more steel than the braced designs. The results for the other parameters are:

- Plot size: A smaller plot size requires a higher floor load to meet the acceleration requirement, therefore the large plot size is more material efficient.
- Floor span: The floor span in the external braced frame has a large influence on the designs with a small plot size. That is because the facade with the smaller span becomes normative for the global displacement. When this happens a smaller floor span decreases the material efficiency significantly. In the larger plot size the influence of the floor span on the braced frame designs is insignificant. For the diagrid designs the larger floor span causes higher normal forces in the diagonals. This increases the material usage in the facade. However, considering the internal structure of the building the large floor span is still more efficient.
- Floor weight: A higher floor weight is more timber efficient for a small plot size, and the lowest floor weight is the most material efficient for a large plot size. With a higher floor weight the steel usage always increases since the connection designs are sized on the ULS checks only.
- Element width: The steel efficiency increases when the element width increases. This is a result of the chosen connection design. The timber usage is similar for all the element widths.

Some of the designs with a small plot size and low floor weight are unfeasible since the elements in the facade are too large. This is a result of the choice to only increase the timber element heights and not the connection stiffness to improve the global stiffness. Therefore, it is recommended to study how the global stiffness can be improved more material efficiently. Other recommendations for further research are on the stability system design and the connection design. Since in this thesis many designs are fixed in simplified exploratory studies to decrease the design space of the parametric model.

Contents

1
Introduction

1.1. Introduction to the subject

The Netherlands is currently dealing with a housing shortage. In the coming years a decrease in new construction of houses is expected. This is caused by the issuance of building permits that are temporarily halted due to nitrogen and PFAS problems [45]. In a report by the CBS [27] a prognosis is made about the number of inhabitants in the Netherlands. By the year 2035 the population will have grown by 1.3 million. The biggest housing shortages will take place in the large cities and especially those in the Randstad. To deal with this growth it is very important that enough extra houses are built in the upcoming years. Next to the housing shortage, there is another big problem in the building industry and that is the climate crisis. In a report from the United nations environment program (2020), it is stated that the building industry was responsible for 38% of the CO2 emissions in 2019 [75]. In the Netherlands we want to decrease the emission of greenhouse gasses with 50% in 2030 and 90% in 2050 [43]. Meaning that there is a lot of correction needed in the built environment. For this reason, bio-based materials like timber have been gaining in popularity. Not only for low-rise buildings but also for high-rise buildings. One example of a bio-based high-rise building is Haut in Amsterdam with a height of 73 meters.

Providing enough structural stability with timber can be challenging as it has a lower material stiffness and lower density than concrete or steel [25]. Building with timber is relatively expensive because you need more material to achieve the same global stiffness and the material is expensive. Moreover, the production of these structural timber elements is still quite new which drives up the prices. Consequently, projects that were first fully imagined in timber can get a non-timber stability system due to budget considerations. Using less material in the timber stability system can make the building cheaper in order for the prices to compete with the concrete or steel stability systems. In addition, decreasing the amount of material decreases the environmental load.

The type of stability system used in the building has an influence on the amount of material needed to provide enough structural stability. From literature it is found that for timber high-rise stability systems the diagrid system appears to be the most material efficient system [21] [41] [71] [62]. After this comes the external braced frame system [74] [23] [41] [71]. Both the diagrid and the external braced frame are externally braced stability systems and will be researched in this thesis. The less material efficient stability systems that will not be studied are cores, shear walls, outrigger, semi-rigid frames, and internal bracing.

1.2. Problem description

Different types of stability systems can be used to make a high-rise timber building. Since timber has a low material stiffness, putting the stability system as far away from the neutral axis of the building as possible would increase the moment of inertia [23]. Thus increasing the material efficiency. Literature attests this and states that a diagrid is the most material efficient system and after that the external braced frame. Both systems have their stability systems along the perimeter of the building. Yet, from reference projects mentioned in chapter 2 and appendix A it can be concluded that only external braced frames are realised in practice, and could even be more material efficient than a diagrid [3]. No timber high-rise building with a diagrid structure has been built as of now. This means that there is a disconnect between literature and practice. One of the reasons for this disconnect could be the connections between the timber elements. When previous studies from literature compare hypothetical stability system designs the connections are not taken into account in detail. Despite, the connections having a lot of influence on the global deformation, element size and dynamic behaviour of a timber high-rise building [41] [60] [71] [24] [33] [49]. The connections will have a connection stiffness that will decrease the global stiffness and worsen the dynamic behaviour of the stability system. The dynamic behaviour of timber stability systems for high-rise was found to be normative for the element sizes of two reference projects [12] [5]. For another reference project the connection design was found to be normative for the element sizes [69]. The influence of the connections on the global stiffness and the element sizes makes an integral comparison considering both the timber elements and connections crucial. When the connections are taken into account a fair comparison can be made on the material efficiency of the diagrid and external brace frame stability systems. From this comparison it will be possible to conclude if literature is correct by claiming the diagrid system is the most material efficient, or if practice is right in using external braced frame stability systems for timber high-rise buildings.

1.3. Goals and objectives

Goal

The goal of this research is to perform a literature, exploratory and parametric study to determine the influence of different design choices on the material efficiency of externally braced stability systems for timber high-rise buildings. In this study both the material usage of the timber elements and steel in the connections between the elements will be considered.

Objectives

The following objectives are set to accomplish the goal:

- *Determine the relevant parameters that influence the material efficiency of timber high-rise externally braced stability systems*
- *Define the relevant ranges for the parameters with exploratory studies to reduce the options that will be studied in the parametric model*
- *Develop a 3D-parametric model that can calculate the material usage of a stability system including the connections*
- *Compare results of the parametric model to conclude how different parameters influence the material efficiency of the timber stability system*

1.4. Research questions and structure

Main question

How can different design choices influence the material efficiency of an externally braced timber stability system for high-rise buildings, based on an integral comparison considering both connections and timber elements?

Report outline and sub-questions

- Chapter 1: Introduction
- Chapter 2: Externally braced timber high-rise
  - *What are the most significant design parameters and design considerations for the material efficiency of an externally braced timber stability system?*
  - *What is the most material efficient connection type for externally braced timber stability systems?*
- Chapter 3: Connection design
  - *How can a slotted-in steel plate connection be designed to be more material efficient?*
- Chapter 4: Model definition
  - *How can the global stiffness of an externally braced stability system be increased in a material efficient way?*
- Chapter 5: Parametric model
- General results
- Results per parameter
- Discussion
- Conclusion and recommendations

Structure

Figure 1.1 shows the structure of this thesis. Chapter 2 will start with a general explanation on timber externally braced stability systems. Then knowledge gained from literature combined with knowledge gained from personal correspondence on externally braced timber high-rise stability systems will be reported. Chapter 3 will discuss the exploratory study on the connection design. First the calculation method is explained and then results from the study will be shown. Finally, the parameters in the connection design will be fixed to decrease the options that will be studied in the parametric model. In chapter 4 the model will be defined. The chapter will start with defining the ranges of the parameters. From these ranges stability system designs will be made. With an exploratory study the three most material efficient designs will be chosen to study further to limit the options studied in the parametric model. Then, the loads working on the models and the flow of forces through the stability systems will be shown. The design constraints that are used to size the elements are also defined. Lastly, an exploratory study to increase the global stiffness in a material efficient way for the SLS design is performed. Chapter 5 will describe how the parametric model works and what assumptions are made. Then the results will be shown and afterwards discussed. Finally, conclusions are drawn and recommendations are made.

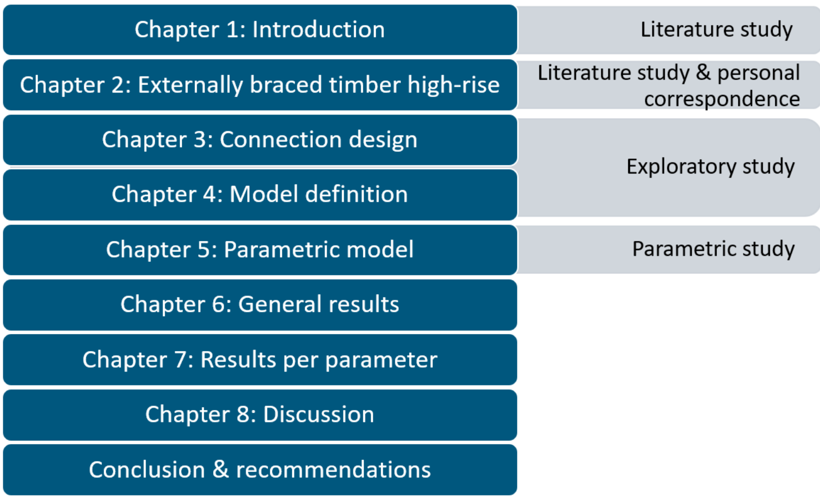


Figure 1.1: Thesis structure

1.5. Methodology

Literature study & personal correspondence

In the literature study, first the external braced stability systems are studied in general. Next, literature on the most influential parameters will be studied. Then, reference projects with externally braced timber stability systems will be studied. For the reference projects a lot of knowledge will be gathered from literature. To gain even more insight multiple structural engineers who have worked on these projects are contacted or interviewed. The information from the personal correspondence will be reported among the findings from literature.

Exploratory study

The amount of design options for the stability systems and the connections are very large. Therefore, exploratory studies will be conducted to decrease the number of options for the parametric study. First a study will be performed on the connection design in chapter 3. This is done to find how the amount of steel can be decreased while obtaining the biggest possible capacity efficiency of the timber elements. After this study many parameters in the connection will be fixed. For the other parameters that can not be fixed a method will be developed to create a connection design within the parametric model without adding extra options to the parametric model. An exploratory study will also be performed on the stability system designs. For this study simplified 2D models are compared in chapter 4 to find the most material efficient stability system designs. This is done to decrease the amount of stability system designs to three. Due to time constraints more designs are out of the scope. The last exploratory study is on increasing the global stiffness efficiently of an externally braced stability system when the SLS requirements are not met. The SLS requirements are the global displacement and along-wind acceleration of the building. In this study it will be investigated what connections or timber elements in the stability are the most material efficient to increase. Finally, it will be defined what timber elements will be increased in the parametric model to increase the global stiffness to meet the SLS requirements. Increasing the connection stiffness in the parametric model is out of the scope of this research.

Parametric study

With all the parameters and input determined a parametric study is performed. The parametric study can give preliminary designs for many different stability system designs quickly. This is useful to study the influence of different parameters. The parametric model will create a preliminary design for the stability system where the timber elements are sized and connections are designed individually for all timber elements in the stability system. The design constraints that are checked in the model are ULS timber element checks, fire safety for the timber elements, global deflection and the along-wind acceleration requirement. The output of the models will be the amount of timber required per design constraint, the amount of steel used and the global deflection and acceleration.

1.6. Scope

This study will have some scope limitations due to the time frame of the graduation thesis. These limitations are:

- **Location & building type:** The building will be a residential building placed in Rotterdam the Netherlands. Thus European Eurocodes and Dutch annexes will be used to perform the checks and determine the floor loads. The building regulations from the dutch Bouwbesluit for residences will be followed when designing the building. As well as, the foundation design being based on soil conditions of the area.
- **Material:** Only timber stability systems will be considered. These stability systems can have connections from other materials. Nevertheless, all structural elements should be made from timber because the goal is to study the feasibility and optimization of timber stability systems. The material stiffness as a parameter is out of the scope of this research and will therefore be fixed.
- **Building elements:** This study focuses on the stability system design. That is why all the elements in the stability system will be designed including connections. To make a fair comparison between the different floor systems the timber for the internal structural system is also calculated. Other elements such as facades and installations are taken into account but not designed. Only the material efficiency of the superstructure will be considered. The number of foundation piles of the substructure will be determined based on the required amount of piles to resist the maximum support reactions. Thus, the foundation will not be taken into account in the comparison of the material efficiency but the internal floors, columns and beams will be.
- **Building design:** The building will have a fixed height and only two plot sizes will be considered. Only three stability system designs will be studied in the parametric model. These designs will be determined in an exploratory study to find material efficient stability system designs. The building geometry will also be determined in the model definition.
- **Design constraints:** The timber elements are checked on axial stress, shear stress, bending stress, combined stress, buckling and fire safety. The connections are checked on the Johansen failure mechanisms, block shear of the timber, shear resistance of the bolts, bearing resistance of the steel plate and block tearing of the steel plates. The global checks that are performed are checking the global displacement and the along-wind acceleration caused by wind. The floor vibrations are not taken into consideration. For the acoustics of the floor a minimum value of 200 kg/m^2 sand is added to the floors to comply with the acoustic requirements for floors. The acoustics for walls, facade and connections are not taken into consideration.

2

Externally braced timber high-rise

In this chapter the literature review that was performed on externally braced timber high-rise buildings will be discussed. First, the externally braced timber stability systems will be explained in general. Secondly, the design parameters are studied. In this paragraph a summation of all design parameters that influence the material efficiency according to literature is given. Next, three reference projects that have a timber externally braced stability system, Treet, Mjøstårnet and Monarch will be studied. From these reference projects insight will be gained on the design process, the most important design considerations, and lessons learned during the making of the building. The three most important design considerations from the reference projects will be discussed afterwards. These considerations are the dynamic behaviour caused by wind, the fire design and the connections. All four paragraphs will be used to answer the following questions:

What are the most significant design parameters and considerations for the material efficiency of an externally braced timber stability system?

What is the most material efficient connection type for externally braced timber stability systems?

2.1. Timber stability systems

A structural system of a building must transfer the gravity forces to the ground but it must also withstand lateral forces like wind. The elements in the structure that keep the building from failing under lateral forces are called the ’stability system’ or the ’lateral load resisting system’ as can be seen in figure 2.1. In structural engineering a stable building is one that will return to its equilibrium state when it gets a small displacement. An unstable building will proceed to move away from the equilibrium, for example by tipping over or collapsing [15].

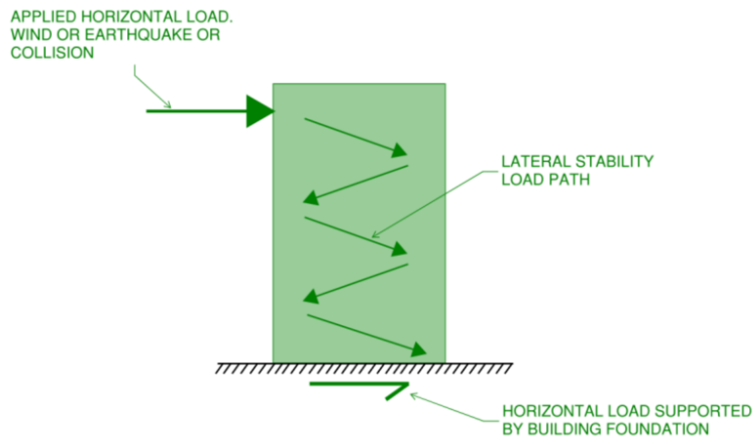


Figure 2.1: Stability system load path [67]

The stability systems in timber are made with either 2D-elements like timber panels or 1D-elements such as timber columns, beams and braces. These elements are typically made of engineered wood. Engineered means that the wood is processed after it arrives from the forest in boards. Timber is an anisotropic material meaning that properties like the strength differs in different directions [82], thus processing can create elements with a higher strength. To create 1D-elements a type of timber called glulam, which is an abbreviation for glued laminated timber, is often used. The creation of this product is shown in figure 2.2. The boards are selected on their strength and defects are removed. Then the boards are connected in the length of the element, layered, glued and heated to create whatever length, width, depth and shape is desired. When creating glulam all elements are layered in one direction to create a timber that is strongest in the length of the element [37]. It is also possible to create elements where the layers are stacked at a 90 degree rotation to their previous layer. This is done in the 2D-elements to make the strength similar in both directions of the plate. Different stability systems for timber are possible with 2D and 1D elements. Externally braced stability systems use 1D elements to resist the lateral forces. Under the externally braced systems fall the externally braced frame and diagrid stability systems.

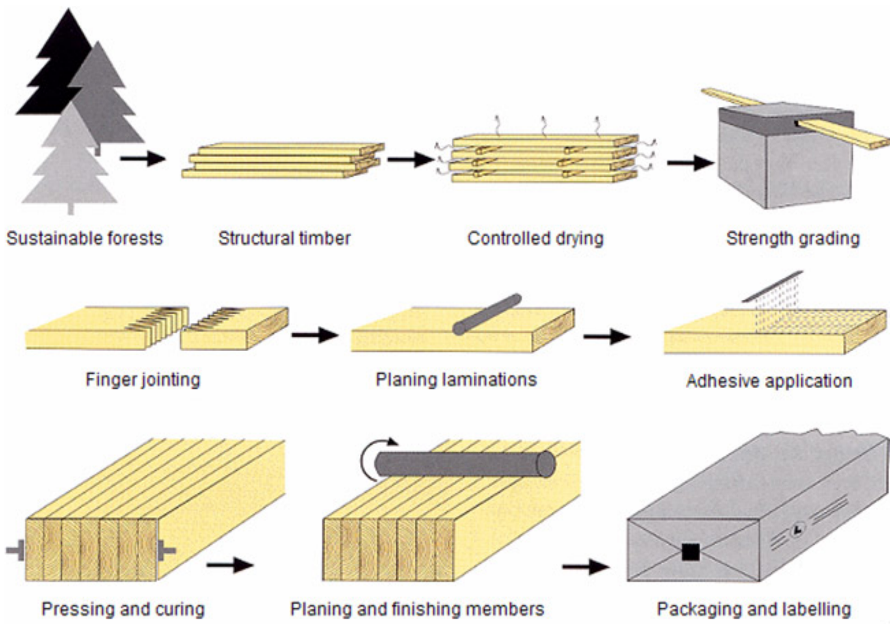


Figure 2.2: Glulam production process [26]

2.1.1. Externally braced frame

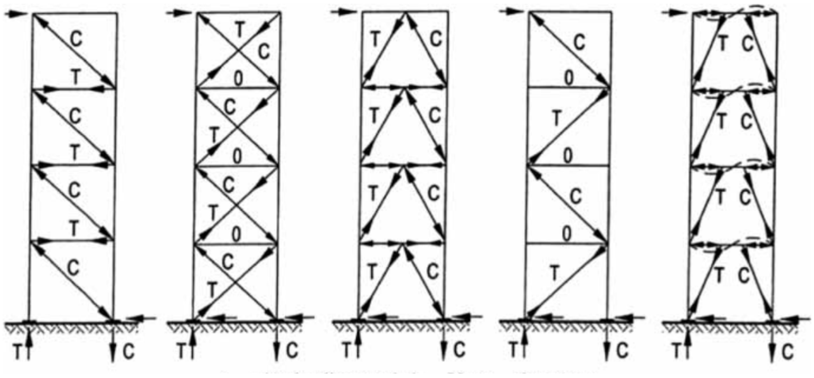


Figure 2.3: Load path under lateral load for braced frame structures [31]

A braced frame structure can be used in the interior of a building but also on the exterior of a building as an externally braced frame. It consists of beams and columns as well as, diagonal elements called braces. The beams and columns are used to transfer the vertical loads. When a braced frame is loaded laterally the horizontal forces are primarily taken up by the braces in the system. These braces are loaded mainly axially. The usage of braces can eliminate all moments present in the beams and columns of the system, depending on the placement of the brace. Figure 2.3 shows examples of bracing patterns exposed to a lateral load. If an element is under compression a (c) is shown next to the element, if it is in tension a (t) is shown, and if no force is present in the element a (0) is shown. Good examples of timber structures with a braced frame stability system are Treet and Mjøstårnet. Mjøstårnet uses an external braced frame in the facade with a few large diagonals as braces. The bracing pattern of the building is the same as the fourth bracing pattern in figure 2.3. Treet uses more small diagonals in the exterior and interior of the building as braces. The bracing pattern is a more elaborate version of the fourth bracing pattern in figure 2.3. Both Treet and Mjøstårnet will be discussed further in paragraphs 2.3.1. and 2.3.2. respectively.

2.1.2. Diagrid

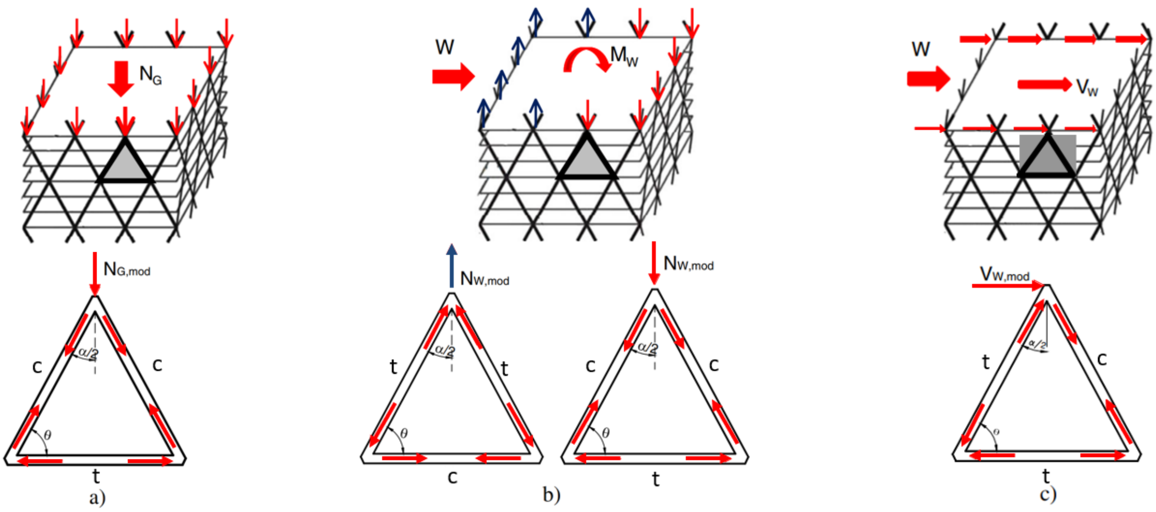


Figure 2.4: Diagrid module under (a) gravity load, (b) overturning moment and (c) shear force [64]

A diagrid system has a perimeter grid and uses triangulation to provide structural integrity. This means that the connections between the members can be hinged [62]. The diagonals can take up the vertical forces as well as the lateral forces making the use of columns unnecessary. A diagrid in its purest form has only diagonals and beams. The beams are necessary to create the structural triangulation. Figure 2.4 shows the forces present in a triangular diagrid module under gravity load, overturning moment and shear force. Next to elements under compression a (c) is written and the elements in tension are marked with a (t). Besides triangulation another defining feature of a diagrid is the ring beam that encompasses the entire facade [65]. The tension present in the ring beam keeps the diagrid structure from bulging outward. The lateral load path caused by wind forces in the plane of the facade is shown in figure 2.5. In yellow the diagonals subjected to tension are shown and in red the elements under compression. A diagrid has a uniform stress distribution across the whole facade. According to literature this uniform stress distribution is the reason why a diagrid is more material efficient than an external braced frame [41] [71]. Figure 2.6 shows the load path of the diagrid building 'Poly International Plaza' in Beijing. This building is made of steel and concrete and has shear walls in the middle. From this figure it can be seen that the lateral load between the facades is transferred partially via the floor and partially via the facade.

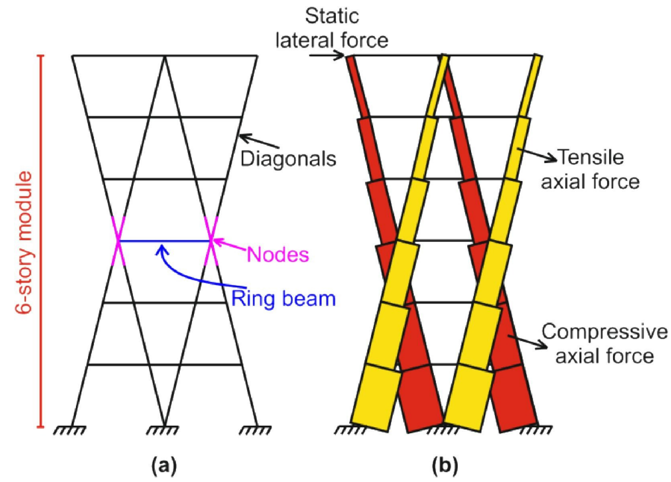


Figure 2.5: 2D lateral load path of a diagrid in plane of the facade [32]

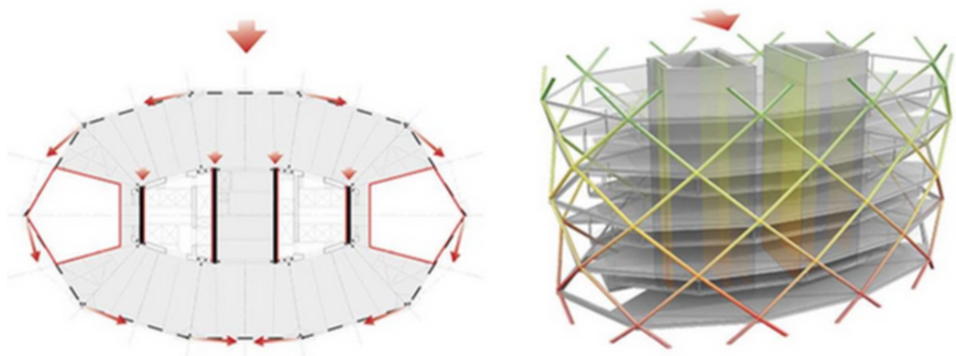


Figure 2.6: 3D lateral load path of a diagrid [63]

2.2. Design parameters

A literature study is performed on the material efficiency of timber high-rise buildings. From this literature study multiple parameters are found that influence the material efficiency of externally braced stability systems. This paragraph will give an overview of these parameters. The material strength also has an influence but this is out of the scope and will therefore not be discussed.

2.2.1. Timber element size

According to Abrahamsen (2022) using fewer large elements is more material efficient than using more smaller elements. This was learned from designing Treet and the reason why the design of Mjøstårnet featured larger elements [3]. When the fire safety checks are performed the outer-layer of the timber is not considered because of the reduced cross-section method, this method will be explained in paragraph 2.4.2. For this reason, Ramage et al. (2017) state that mega structural members are more efficient because a larger percentage of the element remains. Due to the reduced cross-section method a square cross-section is also more efficient than a rectangular one [60].

2.2.2. Building height and slenderness

The higher a building is the more dominant the lateral forces will be. The slenderness of a building is determined by dividing the height of the building by the width of the building. According to Trinh & Zhang (2020) with increasing height and slenderness the overturning moment increases with a power of two and the sway at the tip of the building with a power of four [71]. For both Treet and Mjøstårnet the lateral displacement and dynamic behaviour were normative for most element sizes. This results in a limited height for timber high-rise buildings. Currently, the highest fully timber building reaches a height of only 81 meters with a slenderness of 1:4.8. Theoretically heights of 140 meters for the external braced frame system used in Mjøstårnet, and 200 meters for a diagrid can be reached [21]. However, these maximum heights are highly dependent on the acceleration comfort requirement used for the building.

2.2.3. Foundation stiffness

The foundation stiffness can have an effect on the lateral displacement of the building and therefore also on the dynamic behaviour. In literature it is found that the rotational and horizontal stiffness of the foundation piles do not influence the global stiffness and will therefore not have an effect on the dynamic behaviour and material efficiency of the building. The vertical stiffness of a foundation pile should however be taken into account when evaluating the dynamic behaviour of a building [33] [41].

2.2.4. Angle of bracing

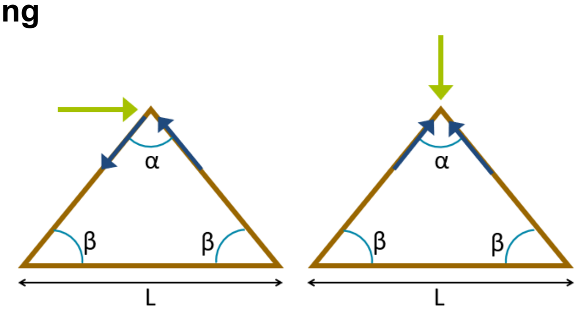


Figure 2.7: Triangular module of diagrid with forces and reaction forces

The angle of the bracing can have a large influence on the global stiffness and optimal usage of the timber elements. Trinh & Zhang (2020) describe that the smaller the size of the triangular module the smaller the lateral displacement for a diagrid structure [71]. Figure 2.7 shows a triangular module with a horizontal and vertical force in green and the respective reaction forces of the module in blue. When the width of the module (L) is constant the triangular module becomes smaller if the top angle (α) becomes larger. Felicita (2021) also concludes that the amount of material decreases with larger top angles for the brace because it works more efficient to provide stability [21].

2.2.5. Floor weight

The weight of the floors influence the dynamic behaviour of the building. Trinh & Zhang (2020) suggest that the mass of the building should be increased when the eigenfrequency is lower than 1 Hz to improve dynamic behaviour [71]. In practice it is also seen that concrete floors are added to timber high-rise buildings to meet the comfort requirements caused by the dynamic behaviour [12] [6].

2.2.6. Floor span

As stated by Ramage et al. (2017) it is important to lead as much of the vertical forces into the stability system to resist the overturning moment caused by the lateral load [60]. This can either be done by increasing the floor weight which would increase the loading on the internal structure of the building requiring more material, or it can be done by changing the floor span. A larger floor span will increase the amount of load that is taken up by the stability system in the facade. Changing the floor span will change the design of the internal structure but does not increase the loading, therefore the amount of material required is not necessarily increased.

2.2.7. Connections

The connection stiffness plays an important role in the global stability and timber element sizes of a building. Kawar (2020) declares that it is often the most critical part of the structure [41]. Slight movements in the connection can weaken the global stiffness [60] [24]. These movements can be caused by slip in the connection and this slip can worsen the dynamic behaviour [71]. Slip in a connection is the same as the axial stiffness of a connection. In a diagrid stability system the rotational stiffness of the joint is said to have no influence on the global design [28]. According to Hjohlman et al. (2020) the axial stiffness of the beam connections does not have an influence on the design for an external braced frame. However, it is mentioned that the axial stiffness in a diagonal and rotational stiffness of a beam connection in an external braced frame do effect the required amount of timber [33]. The slip and rotational stiffness of the connections were found to have an important impact on accurately predicting the dynamic behaviour of the structure [73]. For this research only the axial stiffness of the connection will be considered, the rotational stiffness of the connections is out of the scope. This choice was made because assuming the connections as hinged will give larger timber element sizes since the bending moment in the element will be larger and in the connection it will be smaller as is shown in figure 2.8. The dynamic behaviour would be improved if the connections were assumed to have a rotational stiffness therefore assuming they have no rotational stiffness will give safe results.

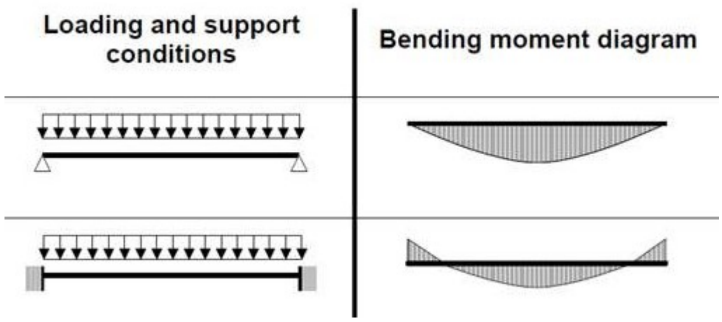


Figure 2.8: Bending moment diagram for a beam with hinged connections(top) and fixed connections (bottom)

2.3. Reference projects

Figure 2.9: Braced frame system Treet [76]

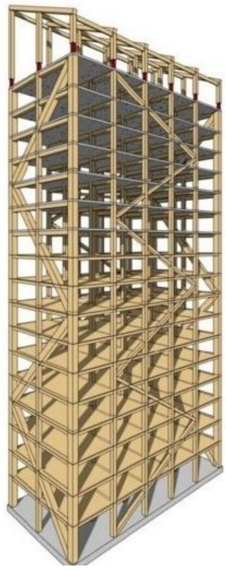


Figure 2.10: External braced frame system Mjøstårnet [4]

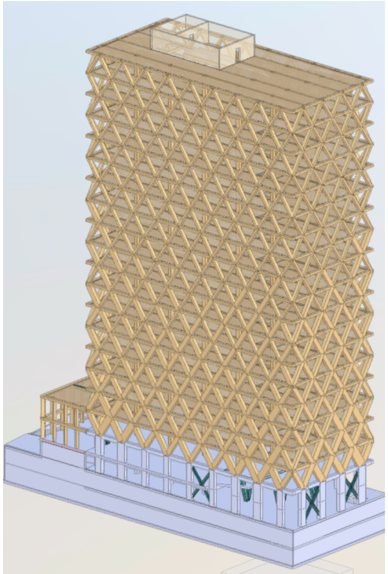


Figure 2.11: Diagrid system Monarch [69]

In appendix A a more extensive analysis of the reference projects can be found. This paragraph will only discuss the most important findings.

2.3.1. Treet, Bergen Norway

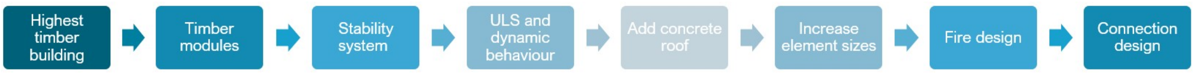


Figure 2.12: Design process Treet

Treet is a residential building that was finished in 2015. It is 45 meters high and at the time it was the highest timber building. The engineers were experienced with making timber bridges and used this knowledge for the design of the building. The design process as can be seen in figure 2.12 first started with wanting to built the highest timber building. After this the idea for stacking timber modules on top of each other was born. These modules could not be stacked more than 4 floors so concrete power storeys were made to support the groups of stacked modules [2]. The stability and vertical load transfer system are made of many small glulam elements to create an external and internal braced frame system, as seen in figure 2.9. Then the ultimate limit state (ULS) checks for the elements and the service limit state (SLS) checks for the global behaviour of the building were performed. This led to the conclusion that the dynamic behaviour of the building did not satisfy. To meet the requirements a concrete roof was added and the sizes of the timber elements were adjusted [48]. As Treet was one of the first fully timber projects a lot of research was done on the dynamic behaviour of the building. This will be elaborated further in paragraph 2.4.1. Below an overview is given of the most important properties of the reference project:

Dimensions	45 meters high with 14 floors of 3-3.3 meters high, plot size 20.7 x 22.3 m
Structural system	Three small buildings with four modules each, on top of each other and power storeys to support the small buildings
Stability system	Braced frame with many small elements, complete facade used, hinges assumed between all elements
Dynamic response	The acceleration of the building was 0.045 m/s^2, damping of 1.9% was a good estimation, building reacts stiffer than expected, concrete floors needed, no slip in connections assumed
Fire safety	Reduced cross-section, all steel elements embedded 65 mm in timber, sprinklers used and structural elements covered by modules
Connections	Slotted-in steel plates used because of experience, some elements had to be enlarged due to connections & fire design

2.3.2. Mjøstårnet, Brummunddal Norway

Figure 2.13: Design process Mjøstårnet

Mjøstårnet is a mixed-use building with a height of 81 meters and was finished in 2019. The same engineers who worked on Treet did the design for Mjøstårnet [5]. As can be seen in figure 2.13, again the ambition was to built the highest timber building. Because the building is a mixed-use building an open floor plan was required, leading to a stability system in the facade. Large glulam diagonal elements act as a brace and are used to resist the lateral loads as is shown in figure 2.10. This is different from the smaller elements in Treet. Being that, using less but bigger elements was found to be more efficient by the engineers working on Treet, especially for higher buildings. It is even thought that this stability system could make buildings up to 140 meters high [3]. After the ULS and SLS checks it was found once more that the dynamic behaviour should be improved. Therefore, the top 6 floors were made in concrete [4]. Next, the timber elements were increased due to added load from the concrete floors. The fire and connection design were the last step and just like in Treet some elements had to be enlarged [3]. For Mjøstårnet there were stricter requirements for the fire safety than for Treet due to the height of the building. As Mjøstårnet was the first fully timber building of 80 meters or higher, experiments were performed to see if using the reduced cross-section method from Eurocode 5 was applicable. In paragraph 2.4.2. this research will be explained further. The reduced cross-section method where you subtract the outer-layer of timber under an accidental fire load situation to check structural safety will also be explained in this paragraph. Below an overview is given of the most important properties of the reference project:

Dimensions	81 meters high with 18 floors of 4 meters high, plot size 17 x 37 m
Structural system	Columns and beams, grid determined by hotel rooms
Stability system	External braced frame with a few big elements, not the complete facade is used, hinges assumed between all elements
Dynamic response	The acceleration of the building was 0.066 m/s^2, determined with a damping of 1.9% afterwards a damping of 1.5% and 2.3% was determined, acceleration to high for the top floor & six concrete floors needed, no slip in connections assumed
Fire safety	Reduced cross-section, all steel elements embedded 85 mm in timber, and sprinklers
Connections	Slotted-in steel plates used because of experience, some elements had to be enlarged due to connections & fire design

2.3.3. Monarch IV, the Hague Netherlands

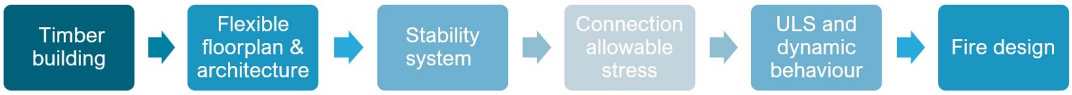


Figure 2.14: Design process Monarch

Monarch IV is a project to create rapid availability of extra square meters of sustainable office space for the dutch government in The Hague. To accomplish this a preliminary design was made for a timber diagrid stability system by RHDHV. The stability system was based on having a flexible floor plan along with architectural considerations [81]. In order to size the timber elements, first an example connection was designed. From this connection a maximum allowable stress of 5.5 N/mm^2 in the timber elements was derived. This decrease in allowable stress caused by the connection design increased the timber element sizes significantly. Afterwards the ULS element checks and SLS global checks were performed as well as reviewing the fire design. No elements in the stability system had to be enlarged due to these checks [69]. Below an overview is given of the most important properties of the reference project:

Dimensions	72 meters high with 20 floors of 3,6 meters high, plot size 20.7 x 44.7 m
Structural system	Columns and beams, grid determined by floor system and floor height
h Stability system	Diagrid with many elements, complete façade is used, hinges assumed between all elements and no columns used
Dynamic response	Requirement of 0.2 m/s^2 a lot less strict instead of 0.06-0.08 m/s^2 like in Treet and Mjøstårnet
Fire safety	Reduced cross-section, but not for connection design
Connections	Slotted-in steel plates because of fire safety and aesthetics

2.3.4. Comparison

Treet has a slenderness of 1:2 and 1:2.2, Mjøstårnet of 1:2.2 and 1:4.8 and Monarch of 1:1.6 and 1:3.4. This means that Mjøstårnet is the most slender. Where Treet and Monarch use more small elements, Mjøstårnet uses less but bigger elements. Treet and Mjøstårnet both have columns to transfer the vertical forces, Monarch only has diagonals. In table 2.1 the amount of timber used in the buildings is compared. From this table it can be concluded that the diagrid of Monarch uses more timber per square meter floor than Mjøstårnet meaning it is less material efficient. However, Monarch has a higher global stiffness and therefore less lateral displacement as can be seen from the unity check (UC) for the global deflection. Treet seems the most material efficient but this is because the timber modules add a lot of timber which is not considered here.

	Height (m)	Floor plan (mxm)	Nr. floors (-)	Glulam (m^3)	Glulam per building volume (-)	Glulam per floor area (m^3/m^2)	Global deflection (-)
Treet	45	20.7x22.3	14	475*	2.29%	0.074	UC=0.79
Mjøstårnet	81	17x37	18	1400	2.99%	0.124	UC=0.86
Monarch	72	20.7x44.7	20	2787	4.18%	0.151	UC=0.57**

Table 2.1: Comparison amount of structural timber and global deflection [69] [77] [47]
*Treet's internal structure not taken into account
** UC is H/1000 instead of H/500

For both Treet and Mjøstårnet the comfort criteria from the dynamic behaviour was the most important design criteria. Minority of the elements had to be enlarged due to fire safety and the connections. In Monarch the connection sizes were the most important even without using extra timber to embed the connections for fire safety. This could partly be explained by the fact that the dynamic requirements used in Monarch for comfort are less conservative than those used in the Norwegian buildings. Monarch also has a larger global stiffness than the other buildings because of the large amount of diagonal elements. Furthermore, the Norwegian buildings had a more extensive dynamic behaviour assessment. The dynamic behaviour of Treet was tested on-site and gave good results that were better than the expected behaviour. For Mjøstårnet the first outcome of the research based on the on-site testing was not very accurate. However, the measured accelerations were higher than the expected accelerations. This could mean that the dynamic behaviour caused by wind was underestimated. It was also found that the actual damping for the short direction of 1.5% was lower than the considered damping of 1.9%. Table 2.2 compares the dynamic behaviour of all three buildings. The maximum allowable acceleration depends on the frequency of the building that is why the unity check (UC) is different. Mjøstårnet and Treet both use the evaluation curve for wind acceleration of ISO 10137:2007 which can be seen in figure 2.16 and Monarch used the limit value curve for wind acceleration of NEN-EN 1990+A1+A1/C2/NB which can be seen in figure 4.21. The limit curve used in Monarch is less strict than the ones used in Treet and Mjøstårnet.

	Damping ratio used %	Frequency calculated (Hz)	Acceleration m/s^2	UC	Damping ratio measured %	Frequency measured (Hz)
Treet	1.9	0.75, 0.89	0.048, 0.051	0.051/0.049 =**1.04**	1.84, 1.61, 1.98	0.97, 1.12 1.12
Mjøstårnet	1.9	0.33, 0.37, 0.59	0.045, 0.066	0.066/0.062 =**1.06**	1.5, 2.3, 2.2	0.50, 0. 54, 0.82
Monarch*	-	0.66	0.2	0.2/0.2=**1**	-	-

Table 2.2: Dynamic behaviour of the reference projects
*Monarch has only been checked to see if it falls within the acceptable range as it is a preliminary design

The fire design of the buildings is also very important for the global design of the building, and especially for the design of the connections. Monarch did not use the reduced cross-section method for the design of the connections. Nonetheless, the connections were still normative for the element sizes. This could be explained by the comfort criteria being less strict. All reference projects use slotted-in steel plate connections due to either experience or fire safety and aesthetics. The allowable compression stress in the slotted-in steel plate connection of Mjøstårnet is 12.4 N/mm^2 with a force of -11500 kN and a tension stress of 5.9 N/mm^2 with a force of 5500 kN. This compression stress is much higher than the allowable stress calculated for Monarch of 5.5 N/mm^2. When re-calculating the estimated connection design for Mjøstårnet with the current Eurocode [57] a maximum allowable compression and tension force of 5039 kN is found. These calculations can be found in appendix B. The large difference between -11500 kN and -5039 kN can be explained by the assumption made in the Norwegian buildings that the column can be loaded end-grain to end-grain when it is loaded under compression. In this study it is assumed that compression capacity is determined by the connection design equal to the tension capacity and the columns will not transfer its forces end-grain to end-grain.

Another design consideration for the connections is that joints that connect two facades to each other are important as making a 3D-connection can be complicated, this will be elaborated more in paragraph 4.2.1. In paragraph 2.4.3. the connection design will be discussed further. A literature review will be performed on different connection types and this will be compared to information on the connections from the reference projects. This is done to investigate if slotted-in steel plate connections are the most material efficient.

2.4. Design considerations

From the comparison on the reference projects it can be gathered that the dynamic behaviour, fire safety and the connections are very important design considerations. Therefore they will be elaborated further in this paragraph.

2.4.1. Wind induced dynamic behaviour

In both Treet and Mjøstårnet the dynamic behaviour caused by wind was the most determining factor for the sizing of the timber elements. There are three different responses to wind load as shown in figure 2.15. According to the current Eurocode 1 on wind, the acceleration in the along-wind direction needs to be checked for the serviceability assessments [55]. The along-wind response is the behaviour for which the acceleration requirements were checked in Treet and Mjøstårnet and is the response that will be discussed in this paragraph.

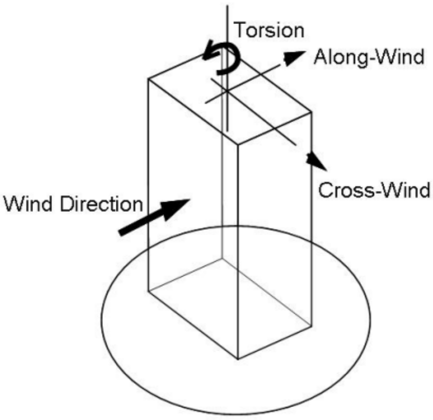


Figure 2.15: Wind response directions [50]

Dynamic behaviour of a building is caused by a temporary lateral force. This force will cause an oscillation with a natural frequency that depends on the building parameters like geometry, stiffness, weight and damping. The oscillating movement causes the building to have an acceleration that is larger at the top of the building, as the deformation is largest there. The acceleration is calculated to determine if the building meets the comfort requirements. To calculate the natural frequency the weight and the global lateral displacement of the building are used. To calculate the acceleration the weight and natural frequency are used.

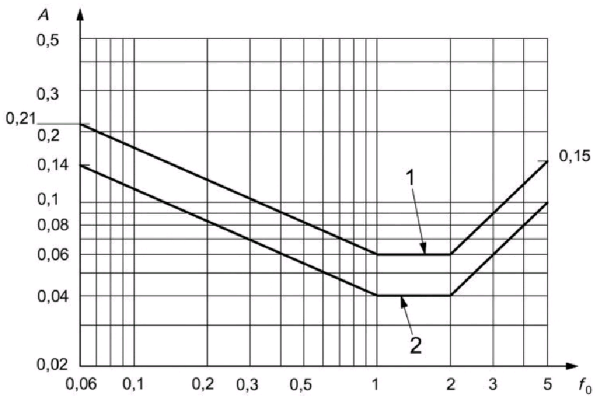


Figure 2.16: Evaluation curves for wind-induced vibrations from ISO 10137:2007 [29]

Since timber high-rise is a recent innovation not much was known about the dynamics of these buildings before designing Treet and Mjøstårnet. Timber is a very light construction material resulting in a low structural weight and proportionally high buildings. Meaning they fall within a range where natural frequency can cause discomfort. The higher the acceleration the more discomfort users experience. According to the standard ISO 10137 the perception acceleration limit for 50% of the population is 0.049 m/s^2 and the limit for nausea is 0.098 m/s^2 [36]. Figure 2.16 shows the graph of permitted acceleration according to ISO 10137:2007 where line 1 is for offices and line 2 is for residences.

To study the dynamic behaviour of Treet multiple researches were done beforehand, and afterwards the real wind-induced accelerations of the building were measured to see if the predictions were correct [12]. The results from these studies were:

- The common approximation for the eigenfrequency for buildings of $f_1 = 46/h$ is a vast underestimation [77].
- Increasing the mass will give a lower eigenfrequency and a lower acceleration [77].
- Reducing the height will give a higher eigenfrequency and a lower total mass. This reduces the acceleration [77].
- A damping ratio of 1.9% is a good estimation for timber high-rise buildings [30].

When estimating the dynamic behaviour of Mjøstårnet the damping ratio of 1.9% was used as well. In studies performed after construction a damping ratio of 1.5% was found for the short direction of the building and 2.3% for the long direction [73]. Since Monarch was only a preliminary design the dynamic behaviour was estimated by determining the global displacement from which the stiffness, eigenfrequency and acceleration were derived [69].

The acceleration of both Treet and Mjøstårnet was calculated by using annex C of NEN-EN 1991-1-4+A1+C2:2011. This method is shown in appendix D. During the calculations some important assumptions were used. These assumptions are:

- A damping ration of 1.9% is used for the exponent of the mode shape ξ
- When calculating the mass per reference area μ analytically 30% of the live load can be added to the weight of the building
- For the structural logarithmic decrement of damping δ_s a value of 0.12 for timber bridges is used.

2.4.2. Fire safety

In Eurocode 5 [57] a method is given to secure fire safety for a certain time period. This is called the reduced cross-section method. Here the timber element is checked under loading of the accidental design situation and the cross-section is reduced according to the time it needs to resist fire. The following formulas can be used to calculate the reduced cross-section, where $d_{char,n}$ is notional design charring depth and d_{ef} is effective charring depth.

$$d_{\text{char},n} = \beta_n t \tag{2.1}$$

$$d_{\text{ef}} = d_{\text{char,n}} + k_0 d_0 \tag{2.2}$$

For Mjøstårnet the main load bearing system must be able to endure a 120 minute fire and must withstand a burnout scenario. Meaning the building must stop burning before it collapses [35]. This was tested by calculating the parametric fire curve where the fire energy from the rest of the building is taken into account. It was found that the charring would stop after 40 minutes with a depth of 30 mm [35]. Another experiment was performed to see how the timber elements would burn during the cooling-down phase of the building. Here as well, it was concluded that the fire would die out. After a charring depth of 75 mm with a fire of 90 minutes the fire would stop [10]. Nevertheless, the more conservative reduced cross-section method was used.

Next to using the reduced cross-section method on the stability system, other measures are taken for both Treet and Mjøstårnet. These measures include sprinklers, fire resistant paint in escape routes and making fire stops to create compartments where the fire will remain [42] [35]. The connection design for both buildings is based on the reduced cross-section method. All steel elements are placed the effective charring depth within the timber. In this way it is assumed that the steel connections will not fail in case of a fire situation [48]. At this time the fire safety design of Monarch is based on the reduced cross-section method for the stability system but the connections are not yet placed within a protective layer of timber [9].

2.4.3. Connection

In Treet and Mjøstårnet slotted-in steel plates are used. No other connection types were considered. These types of connections were used many times before in projects with timber bridges and are shown to have big tension capacities and have performed well in the past [3]. For Monarch two connection types were considered, a slotted-in steel plate or a glued-in rod connection. Figure 2.17 shows an example of how a slotted-in steel plate could be used to connect five elements and figure 2.18 shows how a glued-in rod connection could be used to connect six timber elements. The glued-in rods have the advantage that the compression capacity is larger and that could lead to smaller timber dimensions. The slotted-in steel plate connection has a better fire safety since the steel parts are encapsulated by the timber and it is more aesthetically pleasing. The slotted-in steel plates had the architectural preference so they were chosen for Monarch [9].

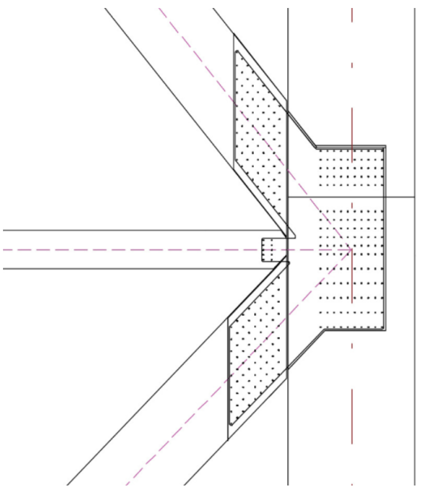


Figure 2.17: Slotted-in steel plate connection [73]

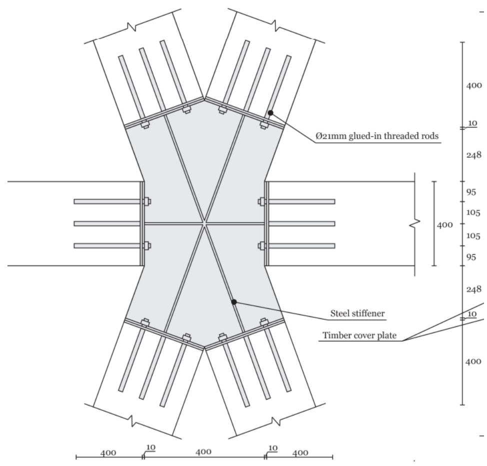


Figure 2.18: Glued-in rod connection [79]

To investigate if slotted-in steel plates and glued-in rods are the only connection types that can be used for externally braced stability systems a comparison of connection types by Van Rhijn (2020) [79] is discussed. The connection types explored are shown in figure 2.19 and are:

- **Glued-in rods:** In glued in rods steel or fibre-reinforced plastic rods are glued in the timber elements to connect them. These rods can be directly glued into the connecting timber members. Or the rods can be connected to for example a steel plate as can be seen in figure 2.19 option A and H [66]. Connection B also uses glued-in rods but these are attached to a steel profile with bolts. The tension forces of a glued-in rod connection need to be taken up by the adhesive that connects the rods to the timber. The compression strength is taken up by the timber elements end-grain to end-grain or by the steel element between the timber.

- **Nails:** Option F shows an example of a perforated plate with nails. The tension forces are taken up by the shear forces in the nails and the tension force in the steel plate. The compression in the connection is taken up by the timber elements end-grain to end-grain.

- **Steel plates with dowels or bolts:** These types of connections can either have steel plates slotted-in the timber like in option C,D and G or the steel plates can be placed on the outside of the timber. The slotted-in steel plate connections are the connections used in all reference projects. Both tension and compression forces are taken up by the connection.

- **Screws:** A connection with screws is shown in option E. The compression force is transferred end-grain to end-grain by the timber elements. The screws will take up the tension forces and can take up shear forces as well. Screws can be combined with steel elements similar to dowels and bolts.

In the study it is found that glued-in rod connections, screws and nailed connections all have a high compression strength that is the same value as the compression strength of the timber. The slotted-in steel plate connections have a lower compression strength as the compression forces are not transferred end-grain to end-grain but through the steel parts in the connection. The tension resistance and axial stiffness of screw and nail connections is very low making them unsuitable for elements that get big tension forces. The tension resistance of glued-in rod connections and slotted-in steel plates can be large depending on the design of the connection. The axial stiffness of the glued-in rod is said to be infinite as no slip can occur in the glue. The axial stiffness of the slotted-in steel plate is also dependent on the connection design and can be increased by increasing the amount of dowels used. Only glued-in rods and slotted-in steel plate connections would have enough tension capacity to be considered for externally braced stability systems. Due to increased fire safety and the fact that slotted-in steel plates have proven their good performance in practice, they are assumed to be the most suitable connection type and will therefore be used in this study. However, without making detailed calculations for many connection designs it can not be concluded if slotted-in steel plate connections are in fact more material efficient than glued-in rod connections [79].

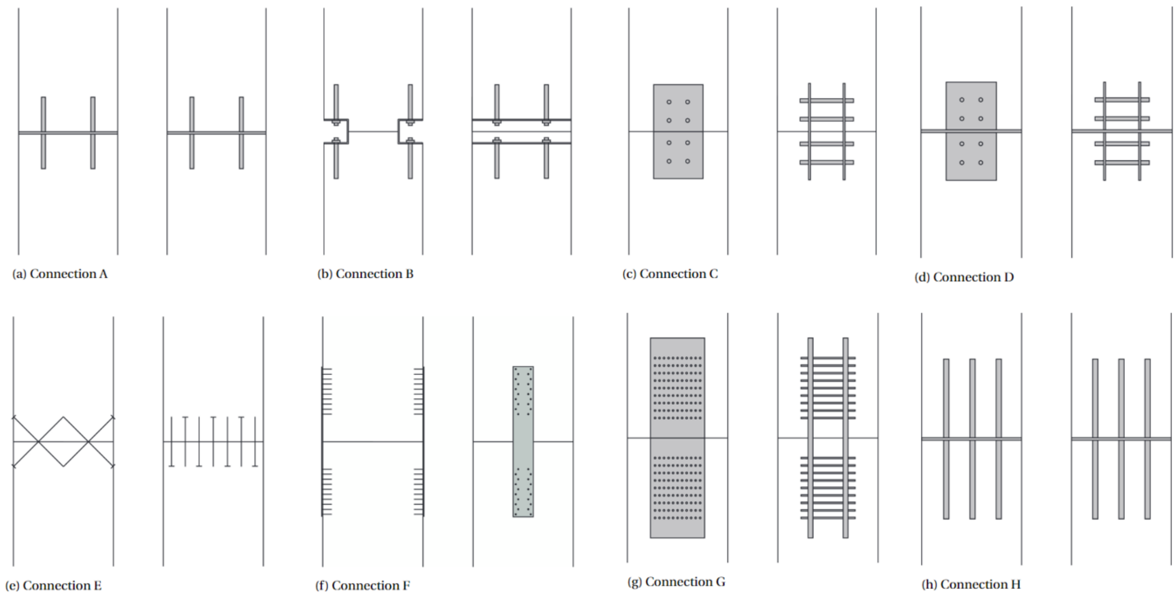


Figure 2.19: Side views of connections for which capacities are calculated [79]

2.5. Conclusion

The sub-questions that need to be answered are:

> *What are the most significant design parameters and design considerations for the material efficiency of an externally braced timber stability system?*

> *What is the most material efficient connection type for externally braced timber stability systems?*

The most significant design parameters for material efficiency from the literature review were:

- **Timber element size:** When using the reduced cross-section method larger timber elements are more efficient because a higher percentage of the element remains. However, element sizes can be limited due to architectural considerations or production.
- **Building height and slenderness:** The higher and more slender a timber building the more determining the overturning moment and dynamic behaviour becomes. Extra material will be needed to satisfy the comfort requirements. For this research only one height and two slendernesses will be studied due to the scope.
- **Vertical foundation stiffness:** The vertical foundation stiffness influences the lateral displacement and dynamic behaviour of the building. Using a stiffer foundation will improve the dynamic behaviour and decrease the lateral displacement. In this study the foundation stiffness is determined by the required foundation piles and the influence of the substructure is out of scope.
- **Angle of the diagrid:** When the lateral displacement and dynamic behaviour are determining for the element sizes the material can be decreased by applying a bigger top angle. This gives the diagonal or brace a flatter slope and more axial stiffness of the element in the lateral direction. To reduce the scope only one angle for the diagrid and one angle for the external braced frame will be modelled for the parametric study.
- **Floor weight:** Adding weight can improve the dynamic behaviour but this will increase the amount of material needed. Three different floor weights will be modelled to see how the material efficiency can be improved.
- **Floor span:** Adding extra loads to the short facade that must resist the most overturning moments and could improve the dynamic behaviour. This will also influence the amount of timber that is needed in the internal vertical load bearing system.
- **Connection stiffness:** The connection stiffness is determined by the amount of bolts in the connection. Adding more bolts would increase the global stiffness of the stability system and therefore improve the dynamic behaviour and lateral displacement but it would also increase the material usage. To simplify the parametric model the connection stiffness will be determined by calculating the amount of bolts required for the ultimate limit state axial capacity of the connection. An increase in connection stiffness will be studied in the exploratory study to increase the global stiffness in paragraph 4.5 but will not be implemented into the parametric model.

The most significant design considerations from the reference projects for the timber element sizes were the dynamic behaviour, the lateral displacement and the connection design. The fire safety design also had a large influence on the connection design. From the research on Mjøstårnet it was found that the stiffness of the connection should also be taken into account when analyzing the dynamic behaviour. The dynamic behaviour can be improved by adding more mass to the floors. Adding more mass will also require more timber and larger connections. The dynamic behaviour and lateral displacement can also be improved by adding stiffness to the stability system by increasing the timber element sizes or increasing the number of bolts in the connections. In both cases extra material is needed but without further research it can not be said what option is the most material efficient. When studying different connection types slotted-in steel plate and glued-in rod connections were found to be the only suitable connection types. Without more in depth research it can not be said what connection type is the most material efficient. However, due to increased fire safety and their good performance in practice slotted-in steel plates will be the only connections considered in this study. Chapter 3 will study how a slotted-in steel plate connection can be designed to be more material efficient.

3

Connection design

In this thesis slotted-in steel plate connections will be used to connect the timber members. This chapter will first give background information on the calculation method for slotted-in steel plates. Afterwards, the exploratory study on the material efficiency of slotted-in steel plates will be discussed. To include the connections in an efficient way in the parametric model a method must be created where the model can calculate and design connections for every timber member individually without adding extra parameters to the model. Therefore, the goal of the exploratory study is to fix as many parameters in the connection as possible or create rules that can determine the parameters based on the timber element sizes. To fix the parameters for the connection design the following sub-question must be answered:

How can a slotted-in steel plate connection be designed to be more material efficient?

3.1. Calculation method

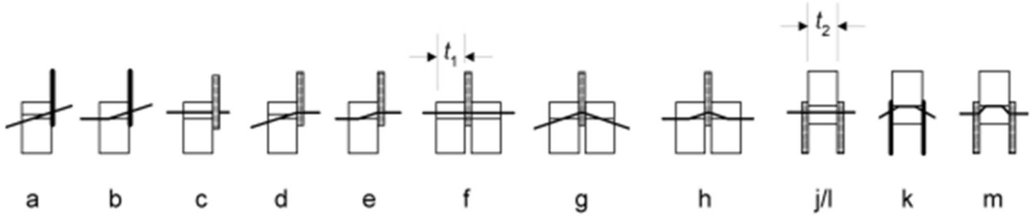


Figure 3.1: Failure mechanisms from the Johansen model

Slotted-in steel plate connections are dowel type fasteners. For these types of connections the Johansen model is used to determine the capacity of the connection. Johansen (1949) created a general theory that could predict the capacity based on the assumption that both the steel and timber elements of the connection would behave as rigid plastic materials [39]. The dowel will behave in this manner under bending stress and the timber under embedment strength. The capacity of the connection will therefore be limited to when either the embedment strength of the timber is reached or a plastic hinge is present in the dowel [13]. 11 different failure mechanisms are defined and can be seen in figure 3.1. These failure mechanisms are either for the outer parts of the connection or the parts located between the steel parts called the inner parts. For a connection with multiple plates three different failure mechanisms are possible for the outer parts, namely f, g and h. For the internal parts only mechanism l and m are possible. Figure 3.2 shows all the possible different combinations. From these six combinations the option with the smallest capacity determines the connection capacity. The embedment strength of the timber is $f_{h,k}$ and the yield moment for when a plastic hinge occurs in the dowel is $M_{y,k}$.

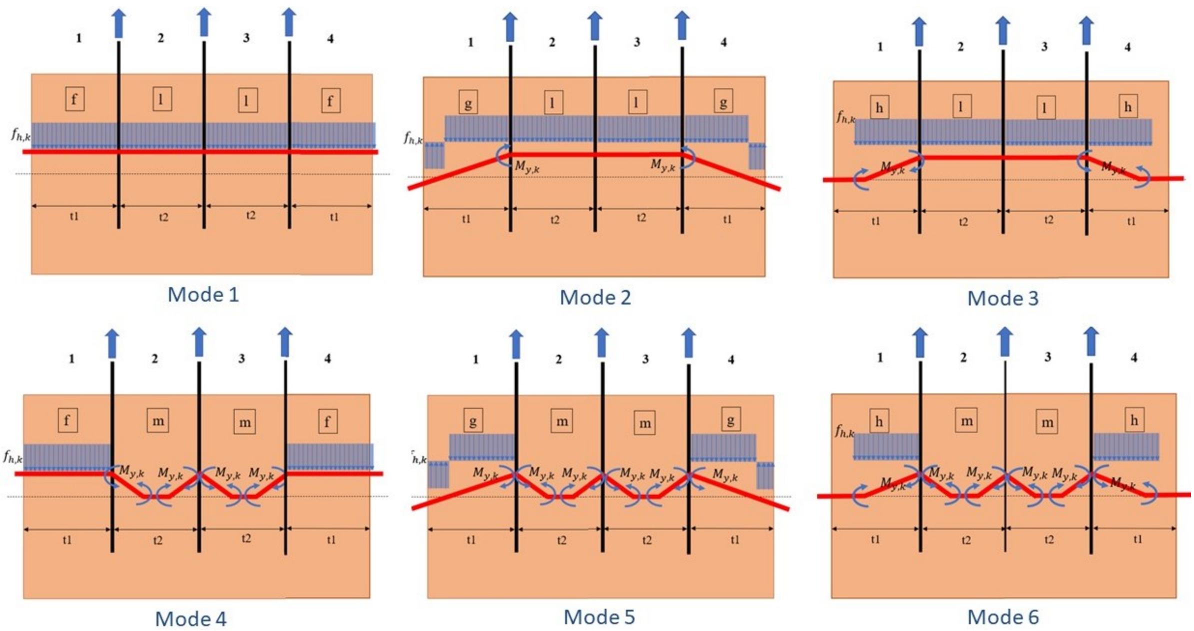


Figure 3.2: Failure modes for steel-to-timber connections with four steel plates

Next to the capacity of these failure modes of the Johansen model other capacities of the connection are calculated as well. One of them is the block shear failure for the timber parts of the connection. In the current Eurocode 5 [57] for timber this capacity is higher than in the draft version for the new Eurocode 5 [16]. This is because the calculation is made material dependent. In this study the calculation method of the draft version will be used as it is presumed to be more accurate due to the material dependence. The block shear capacity is determined by calculating the shear resistance of the side planes $F_{v,d}$ shown in figure 3.3 in yellow with the letter L and the tensile resistance $F_{t,d}$ of the head plane shown in blue with the letter H. The maximum of $F_{v,d}$ and $F_{t,d}$ is the block shear capacity of the connection.

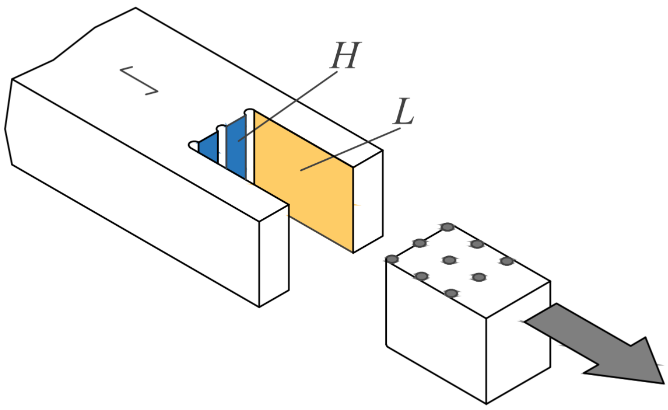


Figure 3.3: Block shear of timber element with dowels [83]

Other capacities that are checked can be seen in figure 3.4 and they are the shear resistance of the bolts, the bearing resistance of the steel plates and the block tearing of the steel plates. In the end the element with the lowest capacity will define the overall capacity of the connection. The load carrying capacity of the connection will be tested for the ultimate limit state (ULS). Next to the capacity, the stiffness of the connection is also determined. For timber dowel type fasteners the stiffness of the connection is determined using the current Eurocode 5 [57]. The connection stiffness will be determined for the serviceability limit state (SLS). In appendix B an example calculation for the capacity and axial stiffness of a timber slotted-in steel plate connection with multiple plates is shown. The calculation method used in appendix B is the calculation method used for all connections in this thesis.

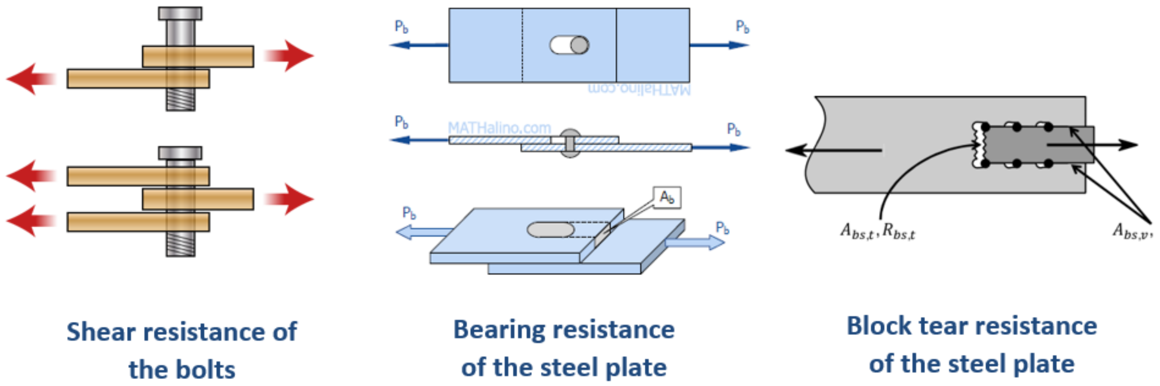


Figure 3.4: Failure mechanisms steel [68][22][20]

3.2. Exploratory study

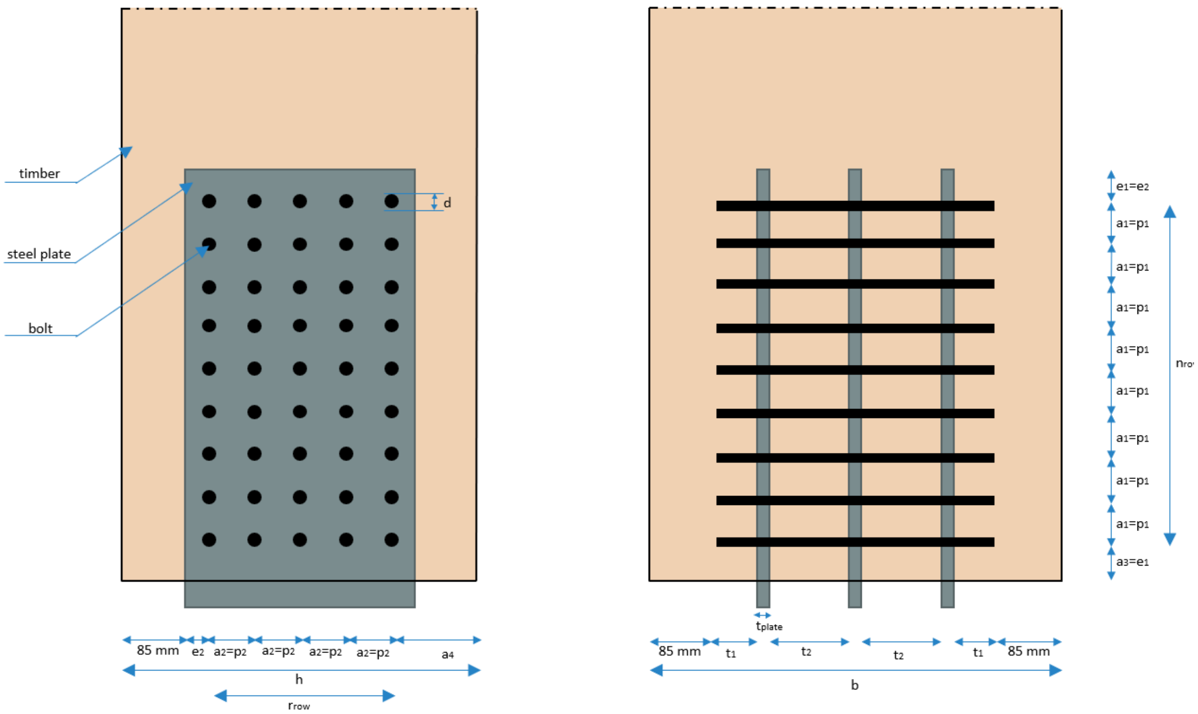


Figure 3.5: Example slotted-in steel plate connection with parameters

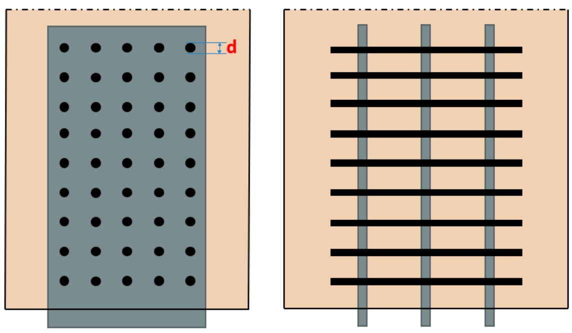
A slotted-in steel plate connection has a lot of parameters as is shown in figure 3.5. The exploratory study has the goal to define as many parameters of the connection in order to decrease the options that need to be researched in the parametric model. The parameters h and b which are the height and width of the timber element respectively, will not be defined in this exploratory study. The width of the timber element will be a parameter in the parametric model and will therefore not be defined here. The required element height of the timber will be determined by the unity checks performed in the parametric model. The number of rows parallel to the grain n_{row} will also not be defined but will be researched in this exploratory study. The component that calculates the connections will use the number of rows n_{row} to increase the capacity of the connection this will be elaborated further in paragraph 5.2.5.

The exploratory study is divided and structured per parameter. To explore the influence of different parameters on the connection two different approaches are applied. The first approach takes the connection calculated in appendix B and changes one parameter. The connection calculated in appendix B is the estimated connection design of the corner column to the foundation in Mjøstårnet. The connection has 4 steel plates and 100 dowels. The timber element has a height of 1485 mm and a width of 625 mm. In a table the capacity of the connection and the normative mechanism will be given. The first row in the table will always show the capacity for the connection design of appendix B which is 3786 kN. The second approach is to make multiple plots per parameter where the other parameters differ as well. This is done because the most material efficient choice for a parameter depends highly on the rest of the connection design. Some parameters will be dependent on the other chosen parameters when they are not being researched. These parameters are:

- e_2: the minimum value of $1.2 * d$ is used
- a_4: the minimum value of $85\ mm$ determined by the reduced cross-section method is added to e_2.
- a_2: a_4 on both sides is subtracted from the height h and then divided by the amount of bolts r_{row}
- a_3: is the minimum value of $7 * d$
- t_2: $85mm$ from the reduced cross-section method and t_1 will be subtracted from the width b for both sides as well as the plate thickness t_{plate} times the amount of plates. The remaining thickness of timber will be divided by the number of plates minus one to make t_2. Figure 3.5 helps to clarify this explanation.

The results of both approaches will be discussed per parameter. Where first a table will be shown with the results of the first approach. Afterwards, one or two plots of the second approach will be shown with an accompanying table where all parameters of the connection are shown. When one of the dependent parameters is defined it will also be documented in the accompanying table. Appendix C shows a more extensive exploratory study on the connection design. In this appendix more plots for the second approach are shown.

3.2.1. Diameter of the bolt d



It was chosen to determine a bolt diameter first as it gives the limits for a lot of other parameters.

Approach 1

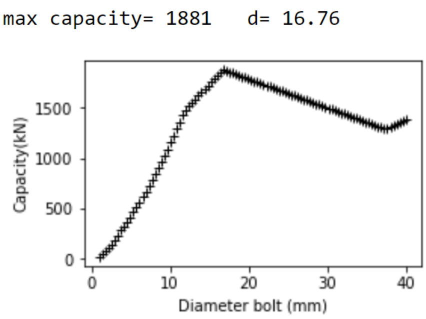
d [mm]	Capacity [kN]	Type
15	3786	Block shear timber
10	4057	Block shear timber
8	3284	Johansen
5	1682	Johansen

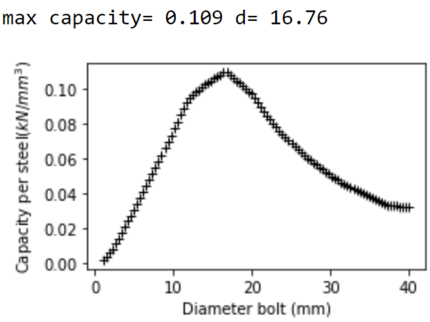
From these results it can be seen that if the diameter is small the connection will fail on the capacity of the Johansen failure modes. When the diameter is larger the block shear strength of the timber becomes normative.

Approach 2

The minimum value of parameter a_1 is dependent on d that is why the maximum value of 100 mm or $(4 * cos\alpha) * d$ is used for the following plots.

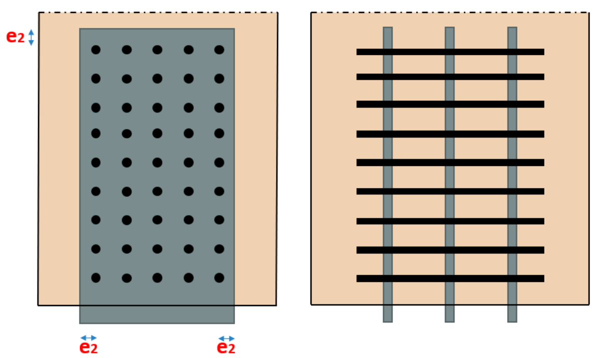
Parameters	
b	$600\ mm$
h	$600\ mm$
n_{plate}	4
t_1	$40\ mm$
n_{row}	6
r_{row}	4
t_{plate}	$15\ mm$





The first graph shows the capacity of the connection on the y-axis and the diameter of the bolt on the x-axis. The second graph shows the capacity of the connection divided by the amount of steel used in the connection on the y-axis. If the capacity per steel used is higher the connection design is seen as more material efficient. For the connection design plotted in approach 2 a bolt diameter of 16.8 mm is most material efficient and also gives the highest capacity. In appendix C 4 more connection designs are researched where the most material efficient bolt diameters are 17.9, 14, 11.2, and 17.9 mm. The average of all five bolt diameters is 16 mm and this will be used as the bolt diameter in the rest of the study. The study on the bolt diameter was not extensive enough to conclude that this is the most material efficient diameter. In Treet the bolt diameter is derived to be 12 mm, in Mjøstårnet it is derived to be 15 mm and in Monarch the diameter is 25 mm. The chosen bolt diameter of 16 mm fits within this range and is thus seen as an appropriate bolt diameter.

3.2.2. Edge distance of the steel plate e_2



e_2 should have a minimal value of $1.2 * d$ and a maximum of $4t + 40mm$. These values come from EC3 and can be seen in appendix B.

Approach 1

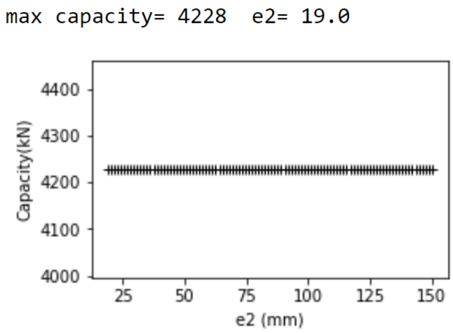
e_2 [mm]	Capacity [kN]	Type
120	3786	Block shear timber
18.2	3786	Block shear timber

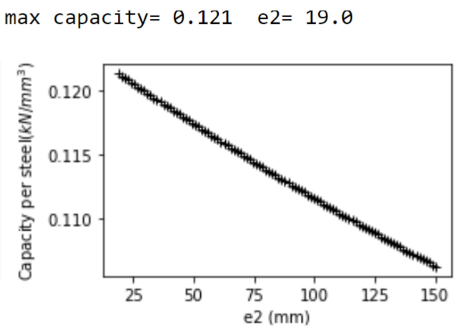
The connection from appendix B has a bolt diameter of 15 mm thus the minimum value for e_2 is 18.2 mm. From approach 1 can be concluded that e_2 seems to have no effect on the connection design.

Approach 2

The minimum value for e_2 is 19.2 mm for a connection with a bolt diameter of 16 mm. So the plots will start at e_2 is 19 mm.

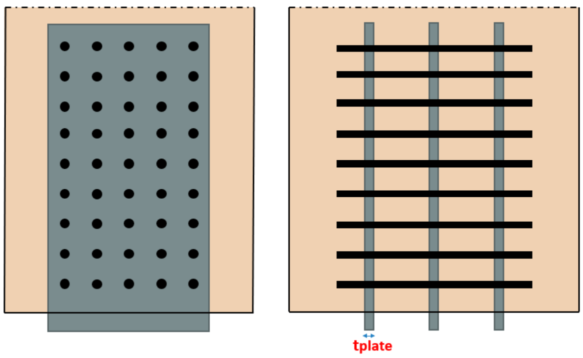
Parameters	
b	$600\ mm$
h	$1000\ mm$
n_{plate}	3
t_1	$40\ mm$
n_{row}	8
r_{row}	8
t_{plate}	$15\ mm$
d	$16\ mm$
a_1	$100\ mm$





From approach 2 it can also be concluded that e_2 has no influence on the capacity. In appendix C another connection is researched where this is also the case. This means that e_2 is the most material efficient for the minimum required value. This minimum is $1.2 * d$ or 19.2 mm. Rounding upwards a value of 25 mm is chosen to allow for production errors.

3.2.3. Thickness of the steel plates t_{plate}



Approach 1

t_{plate} [mm]	Capacity [kN]	Type
15	3786	Block shear timber
10	3993	Block shear timber
8	4075	Block shear timber

If t_{plate} increases the thickness of the timber t_1 and t_2 in the connection will decrease slightly. When the block shear of the timber is normative for the connection a decrease in timber will decrease the capacity slightly. Figure 3.6 shows how an increase from a smaller plate thickness to a larger plate thickness can influence the timber thicknesses. The old plate is shown in grey and the new plate is shown in red. In blue arrows the old sizes for t_1 and t_2 are shown and in red arrows the new reduced sizes are shown.

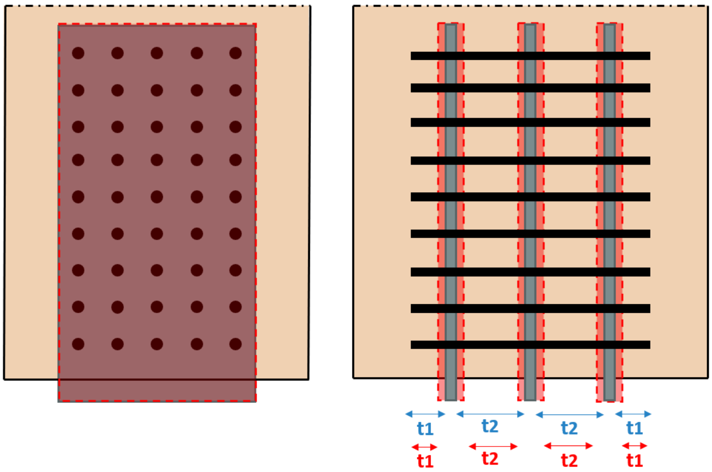
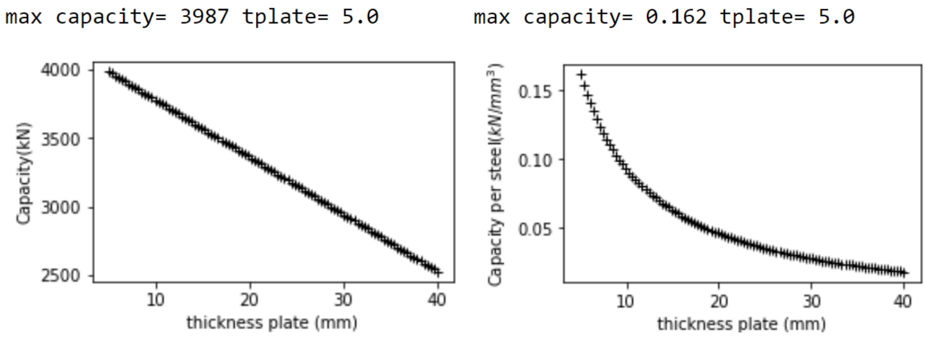


Figure 3.6: Decreasing t_1 and t_2 with an increased plate thickness t_{plate}

Approach 2

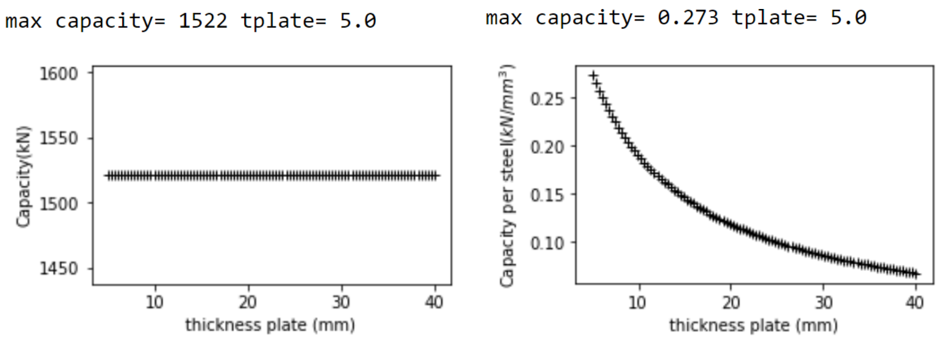
Connection 1:

Parameters	
b	$600\ mm$
h	$1000\ mm$
n_{plate}	4
t_1	$40\ mm$
n_{row}	10
r_{row}	10
d	$16\ mm$
a_1	$100\ mm$



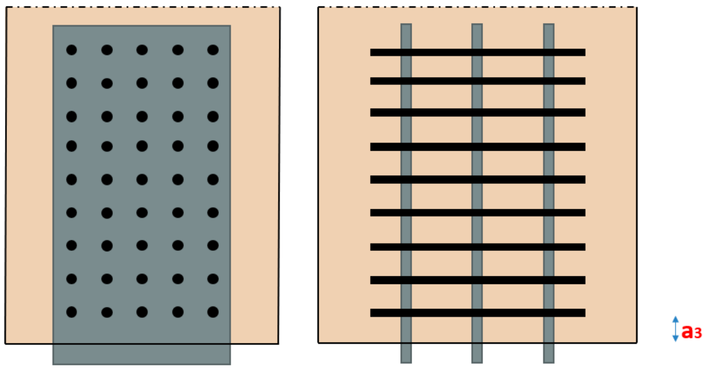
Connection 2:

Parameters	
b	$600\ mm$
h	$600\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	6
r_{row}	6
d	$16\ mm$
a_1	$100\ mm$



Here the results of approach 2 for two connection designs is shown. In appendix C three more designs are studied. All designs had the same trends as connection 1 or connection 2. In connection 1 the block shear strength is normative and in connection 2 the Johansen model is. For all connections the material efficiency is the highest for the smallest plate thickness of 5 mm. However, this is a very thin plate and in the reference projects plate thicknesses of 10, 15 and 25 mm are used. For this reason a plate thickness of 10 mm is chosen as it is the smallest thickness used in practice.

3.2.4. Edge distance of the timber a_3



The smallest required value of parameter a_3 should be the maximum of either $80mm$, $7 * d$ and $1.2 * d$. These requirements are determined in EC3 and EC5 and are shown in appendix B.

Approach 1

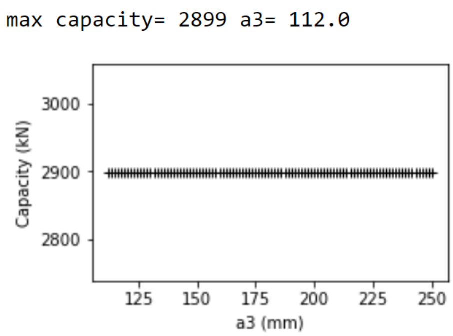
a_3 [mm]	Capacity [kN]	Type
120	3786	Block shear timber
105	3786	Block shear timber

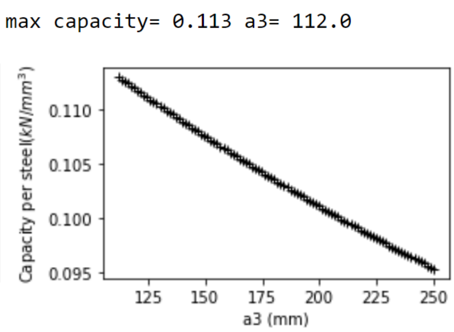
The connection of appendix B has a bolt diameter of 15 mm thus the minimum value for a_3 is 105 mm. a_3 seems to have no effect on this connection design.

Approach 2

The minimum value of a_3 is 112 mm for a connection with a bolt diameter of 16 mm so this will be the bottom value of the plots.

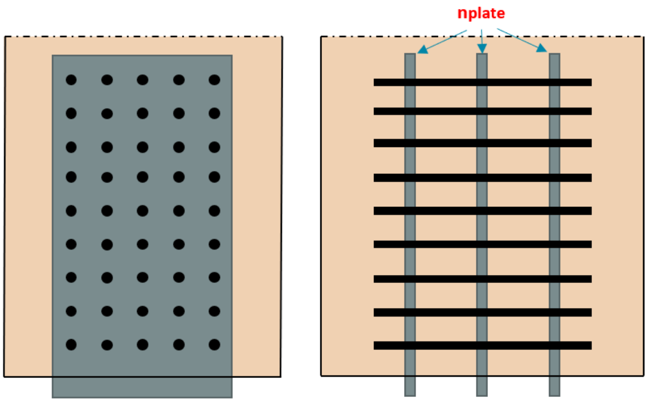
Parameters	
b	$600\ mm$
h	$1000\ mm$
n_{plate}	3
t_1	$40\ mm$
n_{row}	6
r_{row}	8
t_{plate}	$10\ mm$
d	$16\ mm$
a_1	$100\ mm$





The other connection plotted in the appendix has the exact same trend as the graphs shown here. Therefore it can be concluded from approach 1 and 2 that the value of a_3 has no influence on the capacity. A larger a_3 will increase the amount of steel used so it is chosen to use the minimum value for a_3 and that is 112 mm.

3.2.5. The number of steel plates n_{plate}



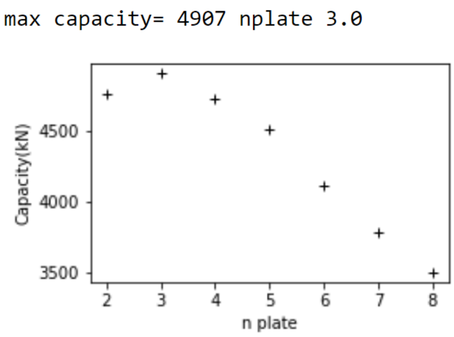
Approach 1

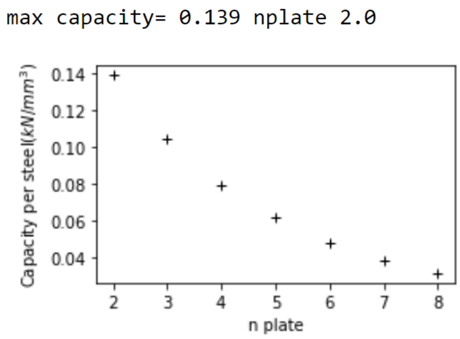
n_{plate}	Capacity [kN]	Type
4	3786	Block shear timber
3	3941	Block shear timber
2	3657	Johansen

The capacity for the connection can be increased slightly by decreasing the number of plates. This is because of the block shear failure of the timber. With less plates the thickness of the timber increases a little therefore increasing the capacity.

Approach 2

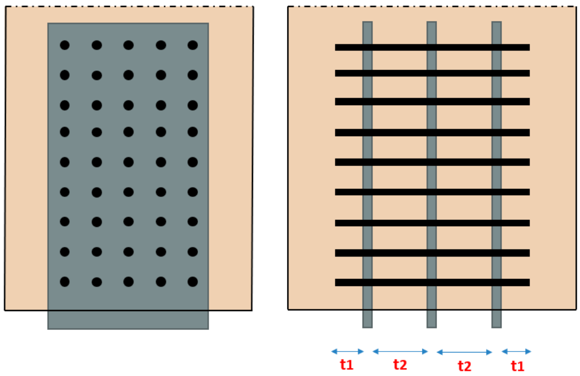
Parameters	
b	$500\ mm$
h	$1500\ mm$
t_{plate}	$10\ mm$
t_1	$40\ mm$
n_{row}	10
r_{row}	10
d	$18\ mm$
a_1	$100\ mm$





In a timber element with small dimensions the block shear will be normative more often than in a larger timber element with the same number of plates. When block shear is normative adding steel plates will decrease the capacity slightly as can be seen in approach 1. In the graphs in approach 2 it can be seen that the capacity of the connection increases when going from two to three steel plates. When using two steel plates the Johansen model is normative and from three plates and up the block shear becomes normative. For this particular connection design the capacity per steel is the largest for two steel plates. The maximum width of a timber element in the parametric model will be 650 mm, this will be defined in paragraph 4.1.2. For connections with this timber width the optimal number of plates is always two or three. For the researched elements with a width or height smaller than 500 mm, two plates give the highest capacity. In all three reference projects there are a lot of elements with either a width or height smaller than 500 mm. Hence, it is expected that in the researched parametric models there will also be large numbers of elements with a size smaller than 500 mm. This together with the fact that for some connections the most material efficient connection design is with two plates while the maximum capacity is with three as shown in approach 2, is the reason why two steel plates will be used in the parametric model. Nonetheless, a connection design with three steel plates is more material efficient for larger timber elements for which the capacity is not determined by the block shear strength. As well as, using more steel plates will increase the connection stiffness and this will increase the global stiffness. The exact number of large timber elements versus the number of small elements in the parametric models is unknown so it can not be concluded if using two steel plates is actually more material efficient than using three plates. The number of plates will not be input in the model as a parameter as this is out of the scope of the research.

3.2.6. Thickness of the timber parts t_1 & t_2



t_1 is the thickness of the timber outside of the steel plates, t_2 is the thickness of the timber between the steel plates. These parameters are both dependent on the width of the timber. If t_1 gets larger t_2 get smaller.

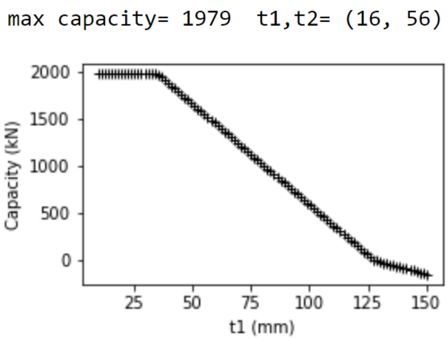
Approach 1

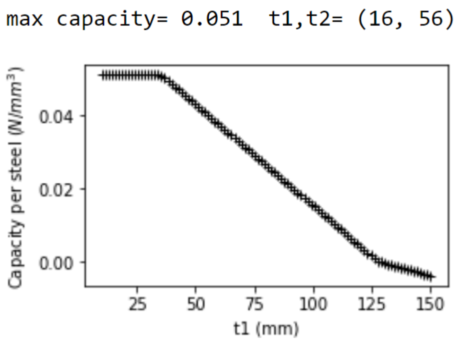
t_1 [mm]	t_2 [mm]	Capacity [kN]	Type
40	105	3779	Block shear timber
60	91.7	3642	Block shear timber
20	118	3930	Block shear timber

For the connection from appendix B the capacity increases when t_1 decreases and t_2 increases. This occurs when the block shear strength of the timber is normative for the capacity.

Approach 2

Parameters	
b	$500\ mm$
h	$800\ mm$
n_{plate}	5
t_{plate}	$15\ mm$
n_{row}	8
r_{row}	5
d	$15\ mm$
a_1	$100\ mm$





t_1 [mm]	0	50	100	150
t_2 [mm]	63.8	38.8	13.8	-11.2

Changing t_1 and t_2 adds no steel to the connection since only the location of the steel plates is changed, consequently the graphs for capacity and capacity per steel are equal. When the block shear strength of the timber is normative for the capacity, the smallest value t_1 gives the highest capacity. This can be explained by the calculation method of the capacity. The effective thickness of the inner parts between the steel plates is always t_2. For the outer parts the effective thickness is $t_{ef} = t_1 * 0.65$ when brittle failure of the timber is normative and not the ductile failure of the bolt. For most of the connections the block shear strength is normative and this is a brittle failure. This means that the effective thickness of the entire timber element can be increased by increasing t_2 and decreased by increasing t_1 as t_1 is reduced with brittle failure. In figure 3.7 the effective thickness of two different connection designs is shown. The red dotted lines in the figure signifies the reduced effective thickness of the outer part. In the top connection the total effective thickness of the inner and outer parts is smaller than that of the bottom connection which has a larger t_2. All connection designs researched in appendix C with a timber element width within the allowed range, give the highest capacity for a t_1 of 15.6 or smaller. There is no bottom value for t_1 and due to the calculation method having the smallest possible t_1 seems logical. However, t_1 must have a certain length to transfer the forces to the outer timber part. In both Treet and Mjøstårnet a t_1 of 40 mm is used. For that reason it is assumed that a value of 40 mm is an appropriate value for t_1 and will be used in the parametric model.

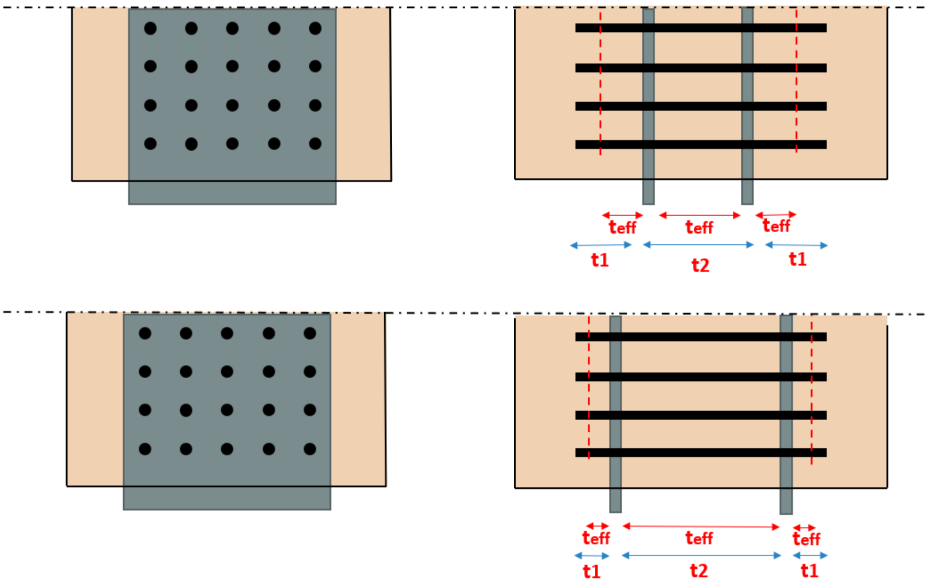
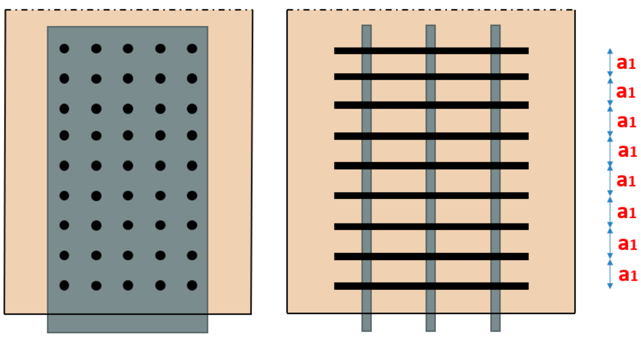


Figure 3.7: Effective timber thickness during block shear

3.2.7. Distance between bolts in direction of the grain a_1



The minimum value of a_1 is the largest value of $1.2 * d$ or $5 * d$. The maximum value is the smallest of $14 * t_{plate}$ or 200 mm.

Approach 1

a_1 [mm]	Capacity [kN]	Type
100	3786	Block shear timber
75	3786	Block shear timber

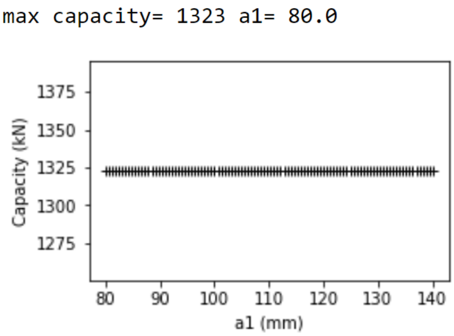
The minimum value for the connection from appendix B is 75 mm as the bolt diameter is 15 mm. The capacity does not change when changing a_1.

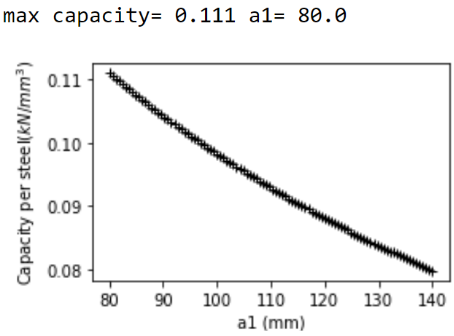
Approach 2

With a bolt diameter of 16 mm and plate thickness of 10 mm the range for a_1 is 80-140 mm.

Connection 1:

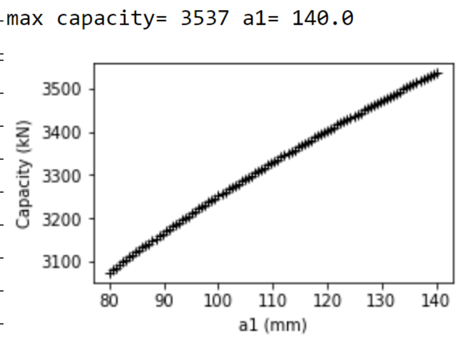
Parameters	
b	$600\ mm$
h	$600\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	10
r_{row}	6
d	$16\ mm$
t_{plate}	$10\ mm$

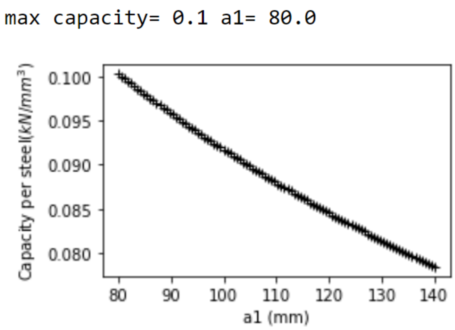




Connection 2:

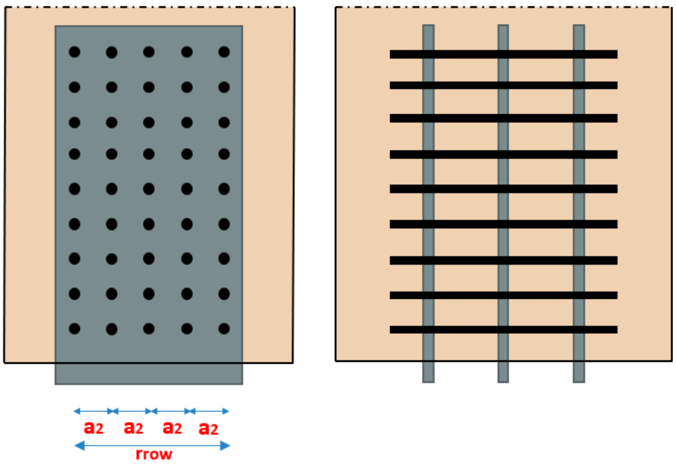
Parameters	
b	$650\ mm$
h	$1500\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	10
r_{row}	10
d	$16\ mm$
t_{plate}	$10\ mm$





Connection 1 fails on block shear of the timber and connection 2 on the Johansen model. This means that a_1 has no effect on the capacity when block shear occurs. When the capacity of the Johansen model is normative increasing a_1 can increase the capacity slightly. However, the capacity per steel usage is always the largest with a minimum value of a_1. Therefore, a minimum value of $5 * d$ is chosen which is 80 mm for $d = 16\ mm$.

3.2.8. Number or rows perpendicular to the grain and distance between the bolts perpendicular to the grain r_{row} & a_2



The number of rows of bolts r_{row} determines the distance between the bolts a_2 since the height of the beam is not a parameter. The minimum value of a_2 is $4 * d$ or $2.4 * d$ the maximum value is $14 * t_{plate}$ or 200 mm.

Approach 1

r_row	a_2 [mm]	Capacity [kN]	Type
10	85	3786	Block shear timber
12	70	3636	Block shear timber
8	109	3955	Block shear timber

The minimum value for a_2 for the connection calculated in appendix B is 60 mm as the bolt diameter is 15 mm. Decreasing the number of rows can increase the capacity for this connection. This is caused by the block shear being normative. When calculating the block shear strength of the timber the tensile failure resistance determines the capacity for this connection. The head tensile plane is shown in blue and indicated with an H in figure 3.3. The length of the head tensile plane is b_{net} and this is calculated with the formula $b_{\text{net}} = (a_2 - d_n) * (r_{row} - 1)$ and shown in figure 3.8. Decreasing the rows of bolts will increase the area of the timber that can resist the tensile stress and hence increase the capacity.

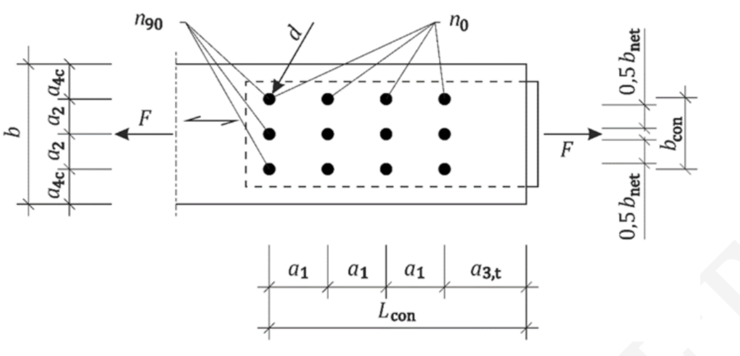


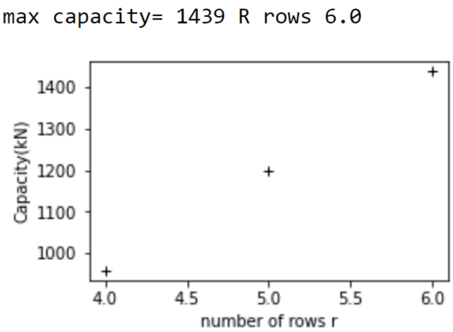
Figure 3.8: Length of head tensile plane b_{net} from the Eurocode 5 draft [16]

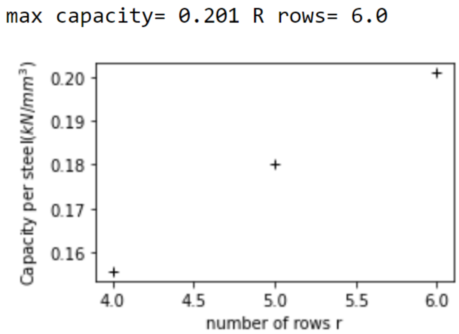
Approach 2

For a connection with a bolt thickness of 16 mm and a plate thickness of 10 mm we get a range for a_2 of 64 mm and 140 mm.

Connection 1:

Parameters	
b	$600\ mm$
h	$600\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	6
d	$16\ mm$
t_{plate}	$10\ mm$
a_1	$80\ mm$

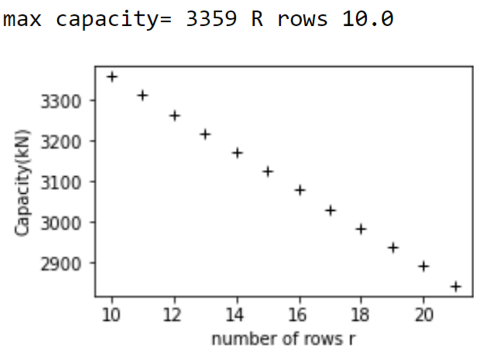


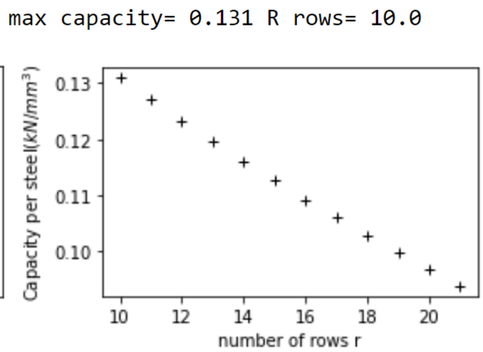


r_{row}	4	5	6
a_2 $[mm]$	130	98	78

Connection 2:

Parameters	
b	$400\ mm$
h	$1500\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	10
d	$16\ mm$
t_{plate}	$10\ mm$
a_1	$80\ mm$

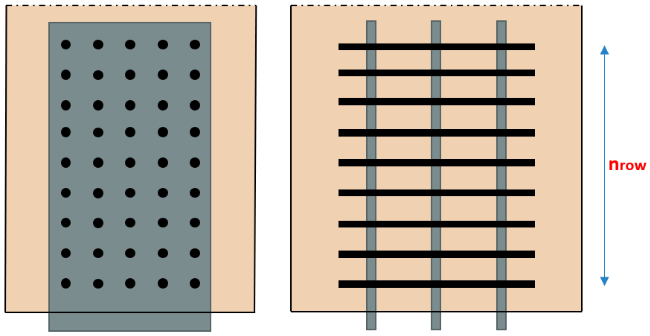




r_{row}	10	11	12	13	14	15	16	17	18	19	20	21
a_2 $[mm]$	144	130	118	108	100	92	86	81	76	72	68	65

Connection 1 shows that the capacity of the connection increases when the number of rows increase. The normative failure mechanism in this connection is the Johansen model. In connection 2 the capacity decreases when rows are added. The capacity of this connection is determined by the block shear strength of the timber. As discussed in approach 1 adding more rows will decrease the area of timber that can resist the tensile strength. In the parametric model some of the connection designs will be determined by the block shear and others by the Johansen model. Consequently it is hard to define the best value for a_2 and r_{row}. Therefore, it is chosen to make the limits for the value of a_2 leading. The minimum value is $64\ mm$ with a bolt diameter of $16\ mm$ and the maximum is $140\ mm$ with a plate thickness of $10\ mm$. The number of rows in the connection design will be determined by dividing the available space for the bolts by $100\ mm$ and then round the result and add one to give the number of rows. In this way the value for a_2 will stay within the limits but it will differ per connection design.

3.2.9. Number of rows in direction of the grain n_{row}



The exact number of rows in the direction of the grain will not be defined in this exploratory study. This parameter will be used to be able to increase the capacity of the connection without increasing the element size of the timber. A range for n_{row} will be defined.

Approach 1

n_{row}	Capacity [kN]	Type
10	3786	Block shear timber
8	3786	Block shear timber
6	3786	Block shear timber
4	3231	Johansen
30	3830	Block shear timber

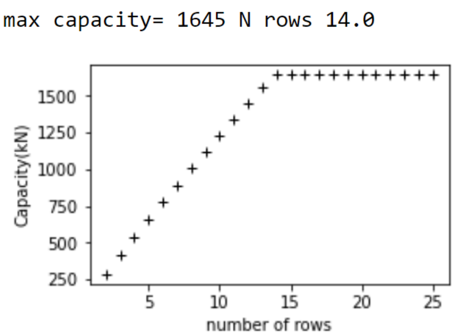
For this connection the capacity stays the same for 10, 8 and 6 rows. This is because the tensile capacity F_{td} of the block shear strength of the timber is normative. For 4 rows of bolts the capacity decreases and the Johansen failure mechanisms become normative. With 30 rows of bolts the capacity increases as the capacity of the side shear planes F_{vd} becomes larger than that of the tensile head. The block shear strength of timber is determined with the following formula:

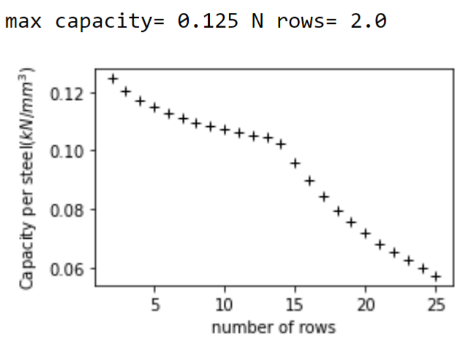
$$F_{bs,Rd} = \max \begin{cases} F_{td} = k_t * b_{net} * t * f_{t,0,d} \\ F_{vd} = 2 * k_v * L_{con} * t * f_{v,d} \end{cases}$$

Where b_{net} and L_{con} are shown in figure 3.8. k_t and k_v are material dependent factors, t is the thickness of the timber part and $f_{t,0,d}$ and $f_{v,d}$ are the material design strengths. If the connection is determined by the block shear increasing the rows of bolts does not increase the connection capacity up until the point the capacity of the side shear planes F_{vd} becomes larger than the tensile capacity F_{td}.

Approach 2

Parameters	
b	$650\ mm$
h	$650\ mm$
n_{plate}	2
t_1	$40\ mm$
r_{row}	4
t_{plate}	$10\ mm$
d	$16\ mm$
a_1	$80\ mm$





When the block shear strength becomes normative for the connection the capacity will not increase when the rows of bolts n_{row} increase. In the plots from approach 2 this happens at 14 rows. Adding bolts can however increase the stiffness of the connection. The range for the number of rows n_{row} will start at one row and end at 15 rows. According to Bjertnaes (2022) a connection design is good when the head tensile plane determines the connection capacity. When the side planes are normative the connection is too long [11]. In approach 1 the design with 30 bolts is a design that is too long as the side shear planes are normative. In approach 2 the capacity only increases up to 14 rows of bolts and the material efficiency decreases with every bolt. In Treet and Mjøstårnet the connection between the corner column and the foundation gets the largest tension force in the building and consequently have the largest connection designs of the building. The connection in Treet has seven rows and in Mjøstårnet the connection has 10 rows. Since the connection should not be too long, the graph in approach 2 stops increasing capacity after 14 rows, the material efficiency decreases for every row added and the reference projects have seven and 10 rows a maximum number of rows of 15 is chosen.

3.3. Conclusion

In this chapter the connection design of slotted-in steel plate connections was discussed. First the calculation method was explained where the load carrying capacity is determined for ULS and the connection stiffness for SLS. Afterwards, an exploratory study was performed. The goal of the exploratory study was to define as many parameters as possible to simplify the parametric model. To define the parameters in the exploratory study the following sub-question needs to be answered:

How can a slotted-in steel plate connection be designed to be more material efficient?

From the exploratory study it can be concluded that a material efficient design is dependent mostly on what failure mechanism is normative. All connections researched were either determined by the Johansen model or the block shear strength of the timber. When the block shear strength is normative adding more plates and bolts does not increase the capacity of the connection. When the Johansen model is normative increasing the number of plates and bolts can increase the capacity of the connection. The block shear strength is normative more often for smaller timber elements and the Johansen model for larger elements. The parametric models will have large timber elements and small timber elements so it will differ what failure mechanism will be normative for which member.

However, the material efficiency of the steel is largest mostly when using less steel. Therefore, it is chosen to use only two plates, a distance between the bolts in direction of the grain a_1 of $80\ mm$ and a maximum of 15 rows of bolts n_{row}. These parameters needed to be defined to reduce the scope but it cannot be concluded that they are more material efficient for the entire parametric model without further research. For the number of rows perpendicular to the grain r_{row}, a middle ground was chosen where the distance between the rows a_2 is determined by the allowed range of the value $64 - 140\ mm$. This is done by dividing the space for the bolts by $100\ mm$ and then rounding to give a distance of a_2 that is within the range. This does not optimize the material efficiency but will give correct connection designs. For some parameters the minimum value gives the most material efficient design as these parameters do not influence the capacity. This is the case for the edge distance of the steel plate e_2 and edge distance of the timber a_3. For the parameters t_{plate}, t_1 and t_2 there are no minimum values given. For both t_{plate} and t_1 the smallest values gave the highest capacity and material efficiency due to the calculation method. However, in practice these values can not be endlessly small. Therefore, it was chosen to define these parameters based on the smallest values from the reference projects to ensure a connection design that could be used in practice. The entire exploratory study is based on the bolt diameter d of 16 mm. If another bolt diameter were chosen the most material efficient design of the connection would be different. So it must be noted that this choice has a lot of influence.

The final defined parameters for the connection are:

- $d = 16\ mm$
- $e_2 = 25\ mm$
- $t_{plate} = 10\ mm$
- $a_3 = 112\ mm$
- $n_{plate} = 2$
- $t_1 = 40\ mm$
- $a_1 = 80\ mm$
- $a_2 = 64 - 140\ mm$
- $n_{row} = 1 - 15$

4

Model definition

This chapter will discuss how the parametric model is defined. First ranges will be defined for the design parameters. Then, the three stability system designs that will be studied in the parametric model are defined in an exploratory study. This will start with some design considerations caused by the connections. Based on the design parameters and design considerations for the connections externally braced stability systems will be designed. From these stability systems one option for the diagrid and two for the external braced frame will be chosen based on material efficiency of the design.

After the stability system designs are defined the input of the loads working on the parametric model will be discussed together with the flow of forces through the stability systems. Later, the design constraints used in the parametric model are treated.

Lastly, the final exploratory study on the design for the serviceability limit state (SLS) is discussed. In this study it is investigated how the global stiffness of the stability system can be improved in the most material efficient manner. The study will consider increasing the stiffness of the timber elements as well as that of the connections. However, it is chosen to only increase the stiffness of the timber in the parametric model and not increase the connection stiffness as this would add a lot of options to the model. With this exploratory study the last sub-question will be answered:

How can the global stiffness of an externally braced stability system be increased in a material efficient way?

4.1. Ranges of the global design parameters

In this chapter the ranges for the parameters that will be input for the parametric model will be defined. The conclusions are partially based on literature, partially on personal correspondence on the reference projects and partially on personal correspondence with experts from RHDHV.

4.1.1. Material

In the model the material GL28c will be used for all timber elements. The material properties are given in table 4.1.

$f_{m,g,k}$	$28\ N/mm^2$
$E_{0,g,mean}$	$12.5\ kN/mm^2$
$\rho_{g,k}$	$390\ kg/m^3$
$P_{g,mean}$	$420\ kg/m^3$
$f_{t,0,g,k}$	$19\ N/mm^2$
$f_{t,90,g,k}$	$0.5\ N/mm^2$
$f_{c,0,g,k}$	$24\ N/mm^2$
$f_{c,90,g,k}$	$2.5\ N/mm^2$
$f_{v,g,k}$	$3.5\ N/mm^2$
$E_{g,0.05}$	$10.4\ kN/mm^2$
$E_{g,90,mean}$	$0.3\ kN/mm^2$
$G_{g,mean}$	$0.65\ kN/mm^2$
$E_{g,0.05}$	$0.54\ kN/mm^2$

Table 4.1: Properties GL28c [17]

4.1.2. Timber element size

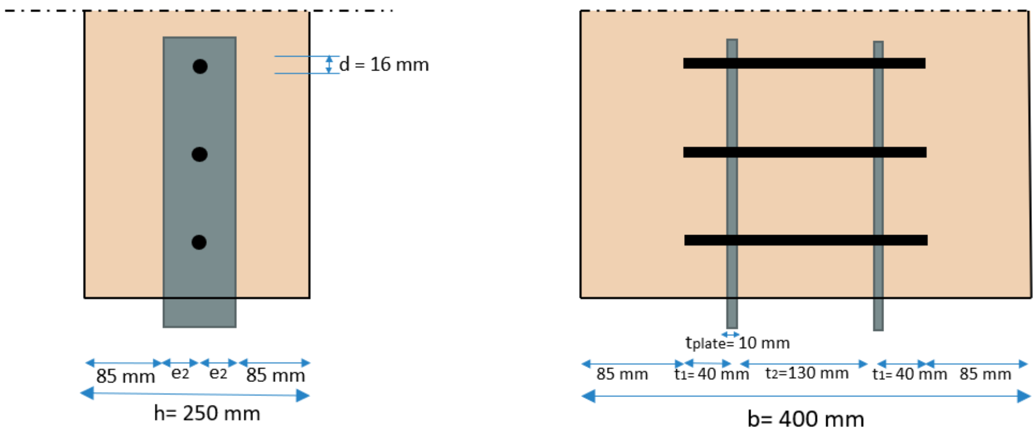


Figure 4.1: Smallest timber element

All cross-sections in the model will have the same width to give a uniform facade design. The heights of the elements will be determined in the parametric model. When the facade is thicker due to the elements being wider there is less usable floor area in the building. This means that the width of the elements should be limited. Since the largest element width in Monarch is 650 mm, and in Mjøstårnet 625 mm, a maximum width of 650 mm is used in this research. The minimum width will also be defined to reduce the scope. The minimum width of the timber will be 400 mm. The width can not be too small as this would give very high timber elements that can block to much of the facade area. Also, the element must be wide enough to ensure sufficient capacity in the connection. With a width of 400-650 mm the capacity for a single bolt in the connection design is 47 kN when the width decreases to 350 mm the capacity decreases to 36 kN. That is why the range of 400-650 mm is applied. The minimum height of the elements is determined by the connection design. In figure 4.1 the smallest timber element size is shown. The smallest height is 250 mm. When adding the 85 mm of the reduced cross-section method to the edge distance e_2 of 25 mm the height is 210 mm. As the timber sizes will be increased in steps of 50 mm the smallest height will be 250 mm. The maximum length of a timber member is 16 meters as this is the largest size available at producers, and that can be shipped.

4.1.3. Building height

For this research only one building height will be chosen to reduce the scope. When looking at residential towers in Rotterdam we find much taller buildings with concrete or steel used in the stability system than with timber. Since in practice no height above 81 meters with 18 floors is reached for an external braced frame timber structure, and a height of 70 meters with 20 floors for a diagrid structure seems feasible in Monarch, a building height of 70 meters with 20 floors was chosen, every floor level had a height of 3.6 meters. Since Monarch is an office and the buildings in this research are residential the floor height could be decreased to 3.4 meters which gives a building height of 68 meters. This height of 68 meters will give relevant results because it is within a buildable range for timber high-rise but is also challenging to realise fully in timber. As can be seen in the completed building Haut in Amsterdam with a height of 73 meters and designs for the building Sawa in Rotterdam with a height of 50 meters. From these buildings it can be seen that a concrete core is used to provide the lateral stability due to financial considerations [19] [54].

4.1.4. Slenderness of the building

To reduce the scope of this research only two plot sizes for the building will be considered with their respective slendernesses. Table 4.2 shows the dimensions and slendernesses of the highest timber buildings for whom all dimensions could be found. Slenderness ratios in timber buildings range anywhere between 1 and 2.4 for the long side of the building and between 1.6 and 4.8 for the short side. With a height of 68 meters the long side of the building ranges between 68 meters and 28 meters. The short side will range from 43 meters to 14 meters. The chosen plot sizes are 27.2 x 27.2 meters and 40.8 x 27.2 meters. The slenderness for 27.2 meters is 2.5 and for 40.8 meters it is 1.67. The plot sizes are based on common plot sizes in residential buildings.

Timber building	Height (m)	Length floor plan (m)	**Slenderness**	Width floor plan (m)	**Slenderness**
Treet	45	22.3	**2.0**	20.7	**2.2**
Mjøstårnet	81	37	**2.2**	17	**4.8**
Haut	73	31	**2.4**	16.6	**4.4**
Brockcommons	53	56	**1.0**	14	**3.8**
Stadthaus Murray Grove	29	18	**1.6**	17	**1.6**
Monarch	70	44.7	**1.6**	20.7	**3.4**

Table 4.2: Slenderness of timber buildings

4.1.5. Angle of bracing

Due to the scope only one angle of bracing for the diagrid and one angle for the external braced frame will be considered. In paragraph 2.2.4. it was stated that a larger top angle of the triangular module would cause a smaller triangular module and therefore decrease the lateral displacement. As well as, a larger top angle using less material since it is more efficient in providing stability. That is why a top angle of 90° is chosen for the diagrid in this research. The angle of the external braced frame designs will be determined in the exploratory study on the stability systems in paragraph 4.2.3.

4.1.6. Floor weight

The floor system consists of timber beams and timber box floors. These timber floors carry a lot of noise so extra measures need to be taken when using such floors to meet comfort the requirements. To decrease the noise sand will be put into the box floors. The sand increases the weight which will decrease the sound transfer. The minimal required weight for a residential building is 200 kg/m^2, this value is based on personal correspondence with a building physics expert from RHDHV. This value will be used as the minimum added permanent floor load. The other added loads are 380 kg/m^2 and 520 kg/m^2. The latter is the maximum amount of sand that fits within the largest box floor of Lignatur [46]. Figure 4.2 shows the three different floor designs per added floor load. The free height in residences should be at least 2.6 meters [14]. Hence, the total floor should not be bigger than 800 mm. For permanent floor load 3 the floor with installations is higher than the allowed 800 mm, however the installations can be placed in the hallways of the residences where the minimum free height is 2.3 meters.

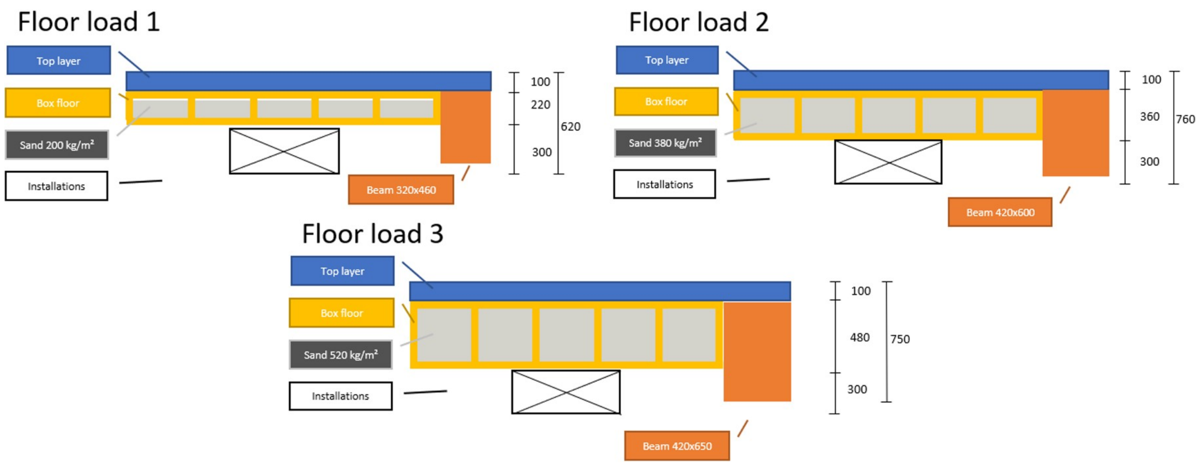


Figure 4.2: Floor designs for the three floor loads, sizes in mm

4.1.7. Floor span

Since the overturning moment will be critical for the short facade the floors will span in the direction that will transfer the loads onto this facade. Figure 4.3 shows the four different floor plan designs. The columns are shown with black squares and the beams that carry the box floors are shown in green. The box floors span in the direction of the red arrows. The 'short' facade will now be referred to as the head facade which is indicated with a H in figure 4.3 and the 'long' facade will now be referred to as the side facade which is indicated with a S. This is done to avoid confusion when discussing the square floor plan. A larger floor span will result in higher beam elements and the span of the box floors is also limited when under heavy loading. Floor spans of 3.4 meter and 6.8 meter are chosen to investigate as larger floor spans would result in higher floors and are not very common in practice.

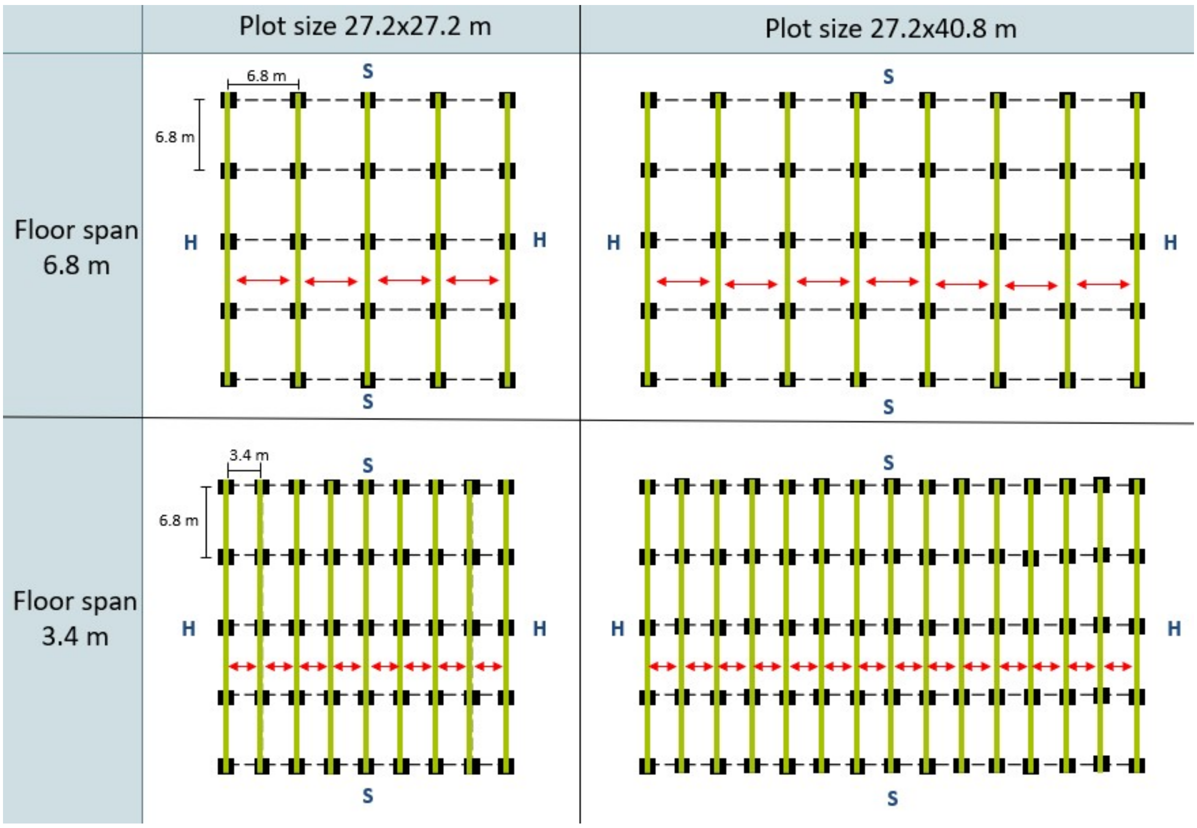


Figure 4.3: Top view four floor plan designs. Columns portrayed as black square, floor beams in green and span direction in red

4.1.8. Foundation stiffness

The vertical foundation stiffness in the building will be based on the vertical stiffness provided by the required foundation piles. Information provided by RHDHV states that a regular foundation pole has a capacity of 3000 kN and a corresponding vertical stiffness of roughly 150 MN/m^2 for Rotterdam. All stability system designs for the three floor loads, two plot sizes and two floor spans are examined and the maximum support forces are determined. From these maximum support forces the required amount of foundation piles per support are defined. Figure 4.4 shows the amount of piles per support for the stability system. When one foundation pile is required it is shown in grey, two piles in blue and three in green.

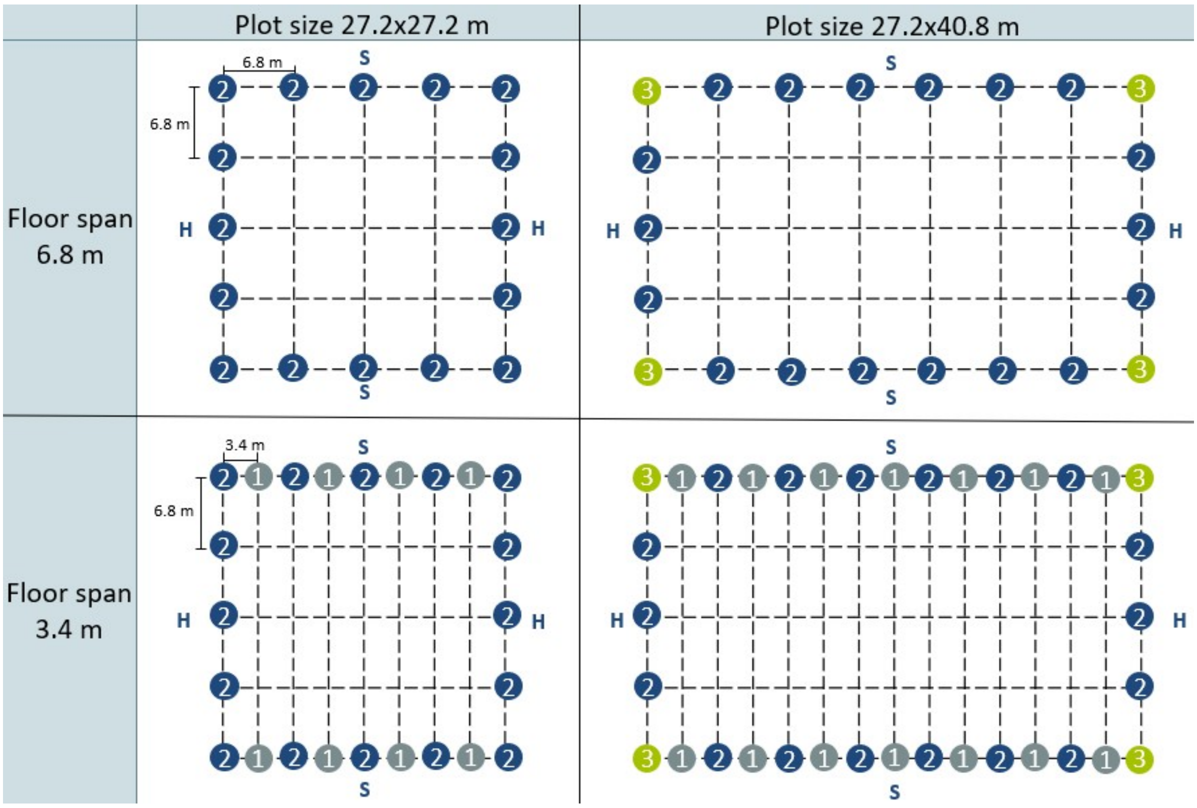


Figure 4.4: Number of foundation piles per support

4.2. Exploratory study stability system design

In this exploratory study the most material efficient stability system designs for a diagrid and external braced frame are researched. A system is seen as material efficient when the global lateral displacement is smaller with the same amount of material used. A smaller displacement will cause a smaller acceleration. The acceleration can also be decreased by adding weight but this will always increase the required amount of material. First, some design considerations caused by the connections are discussed as they influence the possible stability system designs.

4.2.1. Design considerations connections

The slotted-in steel plate connections give some considerations that need to be taken into account when designing the stability systems.

Diagrid

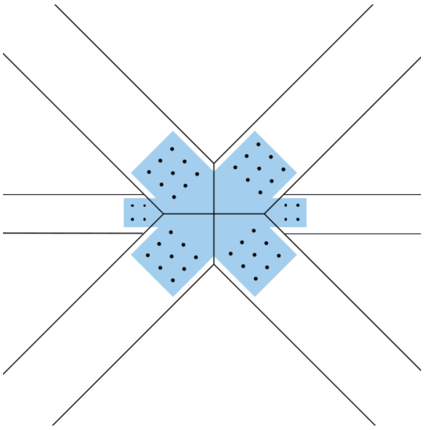


Figure 4.5: Diagrid with split-up members

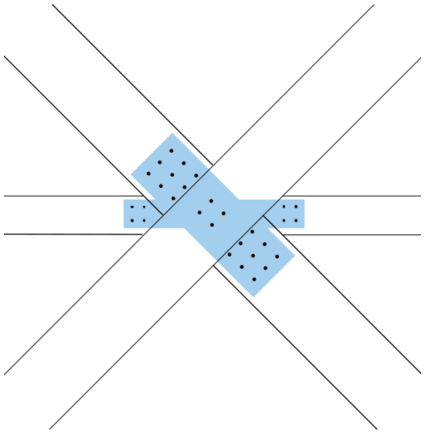


Figure 4.6: Diagrid with continuing member

The connection design for a diagrid will have less steel when one member continues, figure 4.6, then when all the members stop at the intersection, figure 4.5. In a diagrid there are diagonals, beams, and some corner columns that support the overhanging floors. The diagonals get the largest forces, then the beams, and the corner columns get the smallest. Since slotted-in steel plate connections use more steel with higher forces it is chosen to let the member with the largest forces be continuous. In this way the most amount of steel can be saved. Consequently, the diagonals will continue where possible, then the beams and finally the columns.

External braced frame

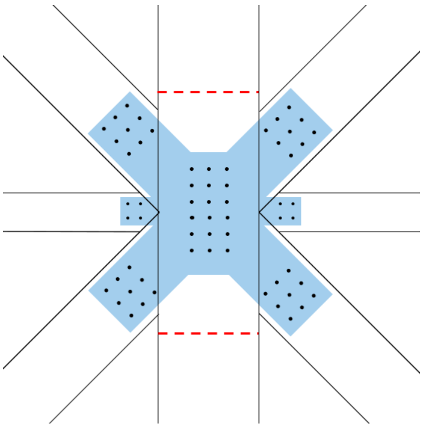


Figure 4.7: External braced frame connection with a slit

For an external braced frame, the forces are the largest in the columns, then in the brace, and the smallest in the beams. This means that the columns will have the first priority to be continuous then the brace and finally the beams. During assembly the steel plates are slid into slots in the timber and then secured with dowels. As can be seen in figure 4.7, a connection where there are four diagonals requires a large slot in the timber, this is indicated with a red dotted line. This will leave spaces in the timber that need to be filled up. These empty spaces will decrease the capacity of the column. Due to extra difficulty during assembly and decreased capacity this type of connection will not be used. And therefore no crossed braces will be considered. Another consideration for the external braced frame is that it is possible to make connections where one column meets two beams and two diagonals in one point, figure 4.8. This type of connection will use less steel because there are less parts where the steel plates need to cross the adjoining member, figure 4.9.

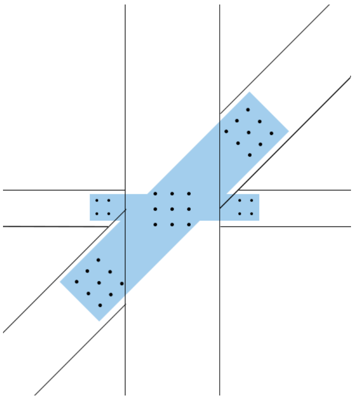


Figure 4.8: Connection design when members meet in one point

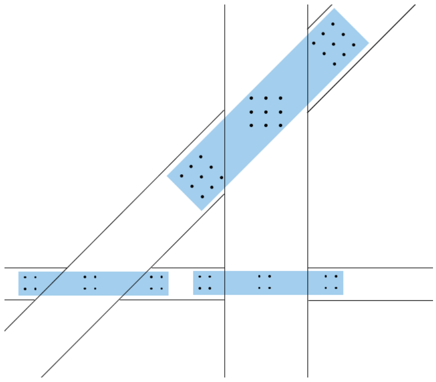


Figure 4.9: Connection design when members do not meet in one point

Corner columns

Making connections in the corner columns is quite a challenge when trying to minimize the amount of steel. When there is a connection that has four diagonal elements that meet both a column and beams you must make a 3D-connection that will result in a steel box. A good example of this is the connection of the Koning Willem I college shown in figure 4.10 and 4.11. In Mjøstårnet they avoided this problem by not making the diagonals reach the corner columns in the longer facade. In Treet the connections from the diagonals do not meet in one point. As can be seen in figure 4.12. To decrease the amount of steel, making complicated 3D-connections must be avoided as much as possible. However, in the diagrid stability system, corner columns are needed on some levels to support the overhanging floor. As well as, the diagonals and beams from either facade are required to meet in one point for a diagrid to work properly. When the corner columns have small forces and are not continuous the connection design could be made simpler than a steel box. In this research it is assumed that the columns in the diagrid can be connected with regular slotted-in steel plate connections without a large steel box.

Figure 4.10: Photograph corner connection Koning Willem I college [44]

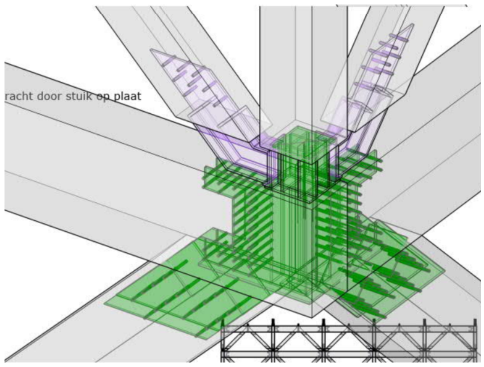


Figure 4.11: Detail corner connection Koning Willem I college [38]

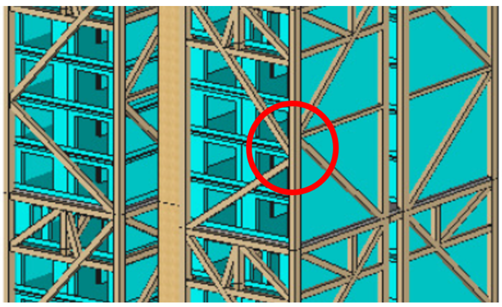


Figure 4.12: Corner connections Treet [8]

4.2.2. Diagrid

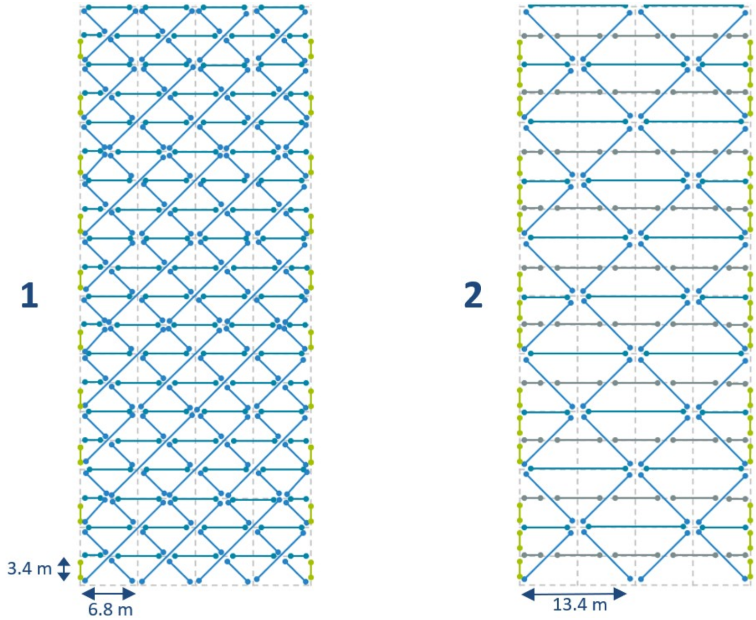


Figure 4.13: Diagrid designs

The diagrid design will have a top angle of 90° as discussed in paragraph 4.1.5. This gives the following two diagrid designs that are shown in figure 4.13. Option 1 has a finer mesh with ring beams on every floor and option 2 has ring beams every other floor. In the figure the hinges of the members are shown as round ends. In option 1 it can be seen that the diagonals spanning from bottom left to upper right continue across multiple floors while the diagonals spanning in the other directions are split into members only spanning one floor. The maximum element length of 16 meters only allows for the diagonals to span three floors. This option requires columns every other floor to support the floors in the corners of the building. For option 2 all diagonals span two floors and here columns are required three out of four floors to support the floors. Only one design will be researched further due to the scope and time constraints.

A 2D model is made in SCIA engineer to research the effect of the wind force on the different diagrid designs. On every floor a wind load of 76 kN is placed which is shown in figure 4.15. All member sizes in both models are 200x300 mm and are made of GL28c. The displacement under wind loading is shown in appendix F. In table 4.3 the total length of all diagonals is shown together with the total length of the columns and the displacement per option.

	Diagonals(m)	Columns(m)	Displacement(mm)
1	769	68	769
2	385	102	1926

Table 4.3: Displacement and element lengths of diagrid options

The displacement of option 1 is 2.4x smaller than the displacement of option 2. Option 1 has twice as many diagonals and 2/3rd of the columns of option 2. Both options have the same amount of beams. This means that option 1 has less than 2.4x the amount of material of option 2. Therefore, option 1 is seen as more material efficient and will be the only diagrid option that will be researched further in the parametric study.

4.2.3. External braced frame

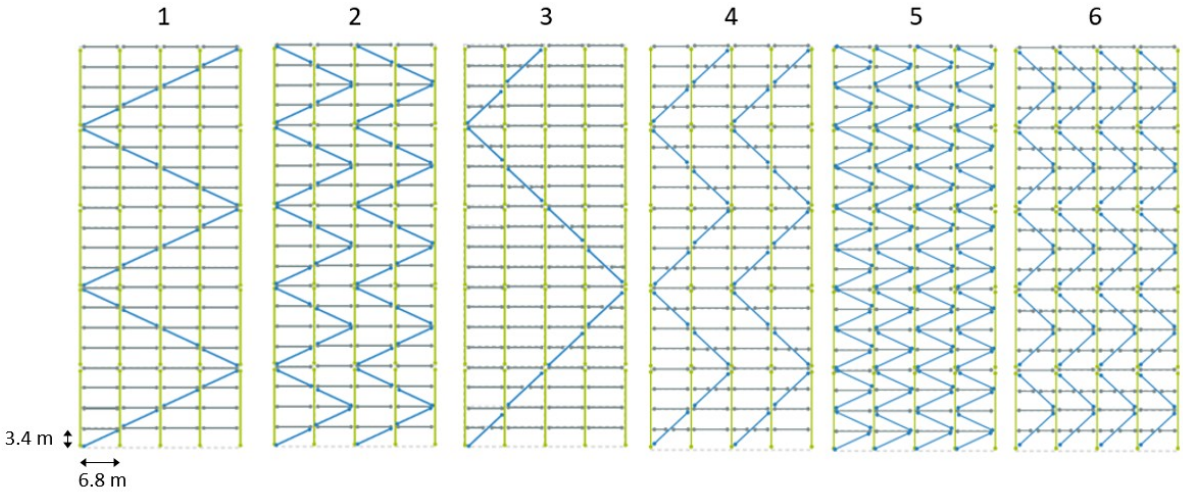


Figure 4.14: External braced frame designs

The external braced frame design is based first on the assumption that the brace should connect in a point where a column and beam meet, in order to decrease the amount of steel as discussed in paragraph 4.2.1. In Mjøstårnet the slope of the brace on the short facade is 1:1.2 and on the long facade the slope of the brace is 1:1.9. Multiple iterations of a slope of 1:1 and 1:2 are studied since these are similar to Mjøstårnet and comply with the assumption of a brace, column and beam meeting in one point. Figure 4.14 shows the different designs. The columns are shown in green, the beams in grey and the braces in blue. The columns will span four floors and the brace will be split every time it crosses a column. The beams are split when they cross a column or a brace. From these options only two designs will be studied further in the parametric model due to time constraints.

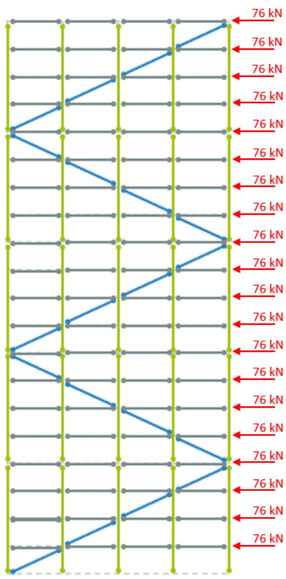


Figure 4.15: Forces on SCIA model

Again a 2D model is made in SCIA engineer to test the effect of the wind force on the different stability systems. Figure 4.15 shows how the forces are placed on the model, this is the same for every option. The results for the horizontal global displacement, normal force, moment, and support forces are shown in appendix F. All members have the same size of 200x300 mm and are made of GL28c. The larger the horizontal global displacement the larger the along-wind acceleration. From the literature research on Treet and Mjøstårnet in paragraph 2.3 it was concluded that the along-wind acceleration was determining for stability system design. The most material efficient design will therefore be the design that has the least amount of material and the smallest lateral global displacement. As mentioned in paragraph 2.1.2 a uniform stress distribution is the reason why a diagrid is the most material efficient [41] [71]. That is why when evaluating the options the ones with a more uniform stress distribution are seen as more material efficient. In table 4.4 the slope of the brace, the length of the braces, the displacement, the largest normal force in the column, the largest normal force in the brace and the largest moment are shown.

	Slope	Brace(m)	Displacement(mm)	$N, max(kN)$ col	$N, max(kN)$ brace	$M_y, max(kNm)$
1	1:2	152	428	1889	1688	16.7
2	1:2	304	297	1730	968	2.9
3	1:1	96	455	1880	2115	31.9
4	1:1	192	310	1671	1282	20.8
5	1:2	608	239	1633	524	4.4
6	1:1	384	243	1570	767	21.7

Table 4.4: Forces in external braced frame options

Options 3,4 and 6 all have a slope of 1:1. These options have shorter braces, higher moments in the columns as well as moments in the braces. The normal force distribution is less uniform than for the other three options. Next to literature stating that a more uniform stress distribution in material efficient there is another reason why a uniform stress distribution would be beneficial. Since the stability system will have elements who all have the same thickness a more uniform force distribution would be more material efficient. Because of the more uniform force distribution and the smaller moments, options with a slope of 1:2 are chosen to investigate further. When looking at options 1 and 2 we see that with a doubling of the braces the deformation decreases with 31% . From 2 to 5, the braces are also doubled but the deformation decrease is only 20%. That is why option 1 and 2 are chosen to study further.

4.3. Loads

This section will describe the loads working on the model and the load combinations. Afterwards the flow of forces through the different stability system designs will be shown.

4.3.1. Permanent loads

The permanent loads on the model are the self weight of the floor and the facade. There are three different floor types for the model which were shown in paragraph 4.1.6. In the tables below the permanent floor loads of these types is shown:

Load 1	q_k (kN/m^2)
Wooden floor	0.5
Topping	1
Sand	2
Total weight	3.5

Load 2	q_k (kN/m^2)
Wooden floor	0.5
Topping	1
Sand	3.8
Total weight	5.3

Load 3	q_k (kN/m^2)
Wooden floor	0.5
Topping	1
Sand	5.2
Total weight	6.7

These floor loads are withing the range of permanent floor loads used in Mjøstårnet which were 2.5 kN/m^2 for the timber floors and 8.5 kN/m^2 for the concrete floors [11]. The chosen weights are limited by the minimal required floor weight for acoustic requirements and the maximum size of the box-floors as was explained in paragraph 4.1.6. How the floor loads work on the model is shown in figure 4.16. In the figure a line load is displayed in green and the point loads are displayed with red arrows. For both external braced frame designs the floor load is input as a line load on the head facade and as a point load caused by the internal beam on the side facade. In the external braced frame the point loads are transferred directly to the columns present in the facade. For the diagrid system the floor load is also input as a line load on the head facade. The point loads caused by the internal beams either transfer directly into the joint between diagonals or in the middle of the beam, as is shown in figure 4.17. The weight of the facade is 1.0 kN/m^2. The load of the facade is multiplied by the height of a floor which is 3.4 meters. This gives a line load of 3.4 kN/m. This line load is put on all beams in the head facade and the side facade.

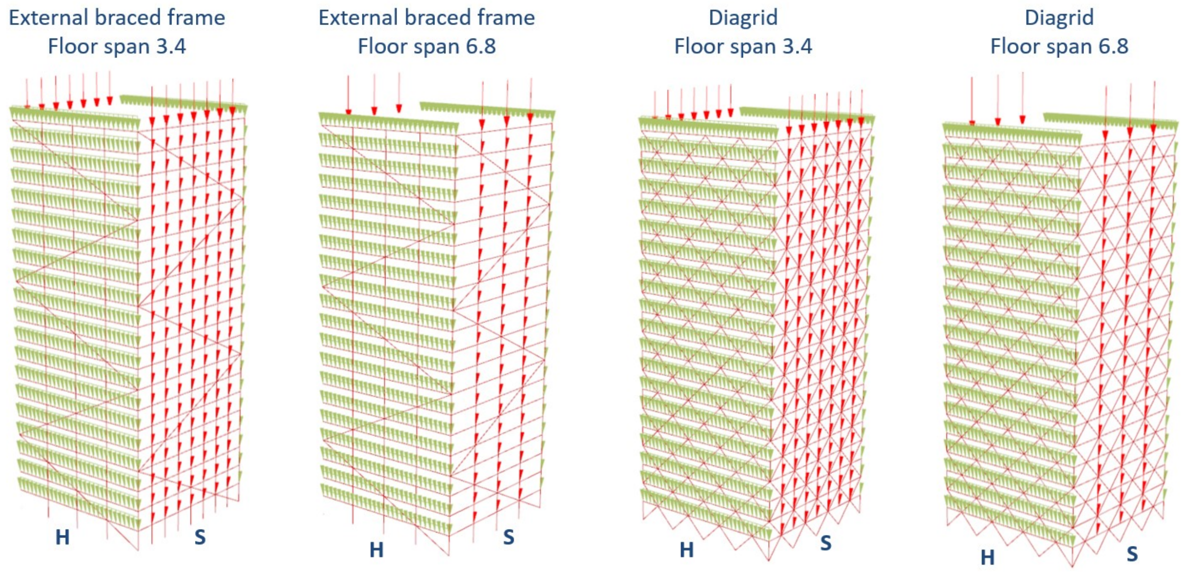


Figure 4.16: Floor loads working on the model

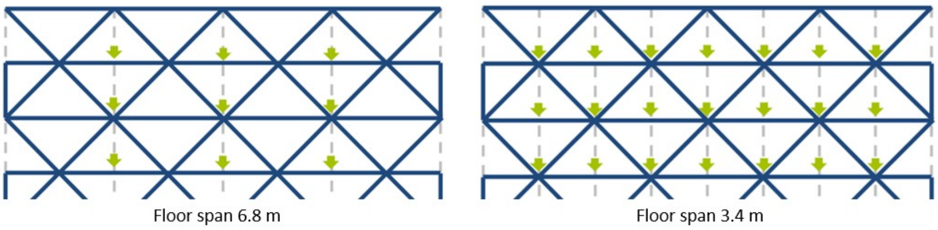


Figure 4.17: Diagrid side facade, load transfer of the floor beams

4.3.2. Variable loads

The variable loads working on the model are live loads, wind loads and moveable walls. For this research the loads caused by snow and rain are not taken into account. The live loads are imposed loads residencies for non-common floors of 1.75 kN/m^2 from NEN-EN 1991-1-1+C1+C11:2019/NB:2019. These loads work on every floor and the roof, this is done to simplify the model. The movable walls have a weight of 1.2 kN/m^2 and work on every floor. The wind load is determined in appendix D and works on the facades of the building. The ψ-factors per load are given in table 4.5.

	$q_k(kN/m^2)$	ψ_0	ψ_1	ψ_2
Imposed loads residencies, non-common floors	1.75	0.4	0.5	0.3
Imposed load movable walls	1.2	1	1	1
Wind	2.0	0	0.2	0

Table 4.5: ψ-factors

The live loads and moveable walls transfer their loads in the same manner as the floor loads in figure 4.16. The wind loads are put into the model as point loads. Figure 4.18 shows how the wind load is put on the stability systems. For the external braced frame designs the wind is put on every joint where a column meets a beam. This means that when the span of the loaded facade is 6.8 meters the wind load is put in every 6.8 meters, and with a span of 3.4 meters every 3.4 meters as can be seen in figure 4.18. For the diagrid design the wind load does not change when the floor span changes.

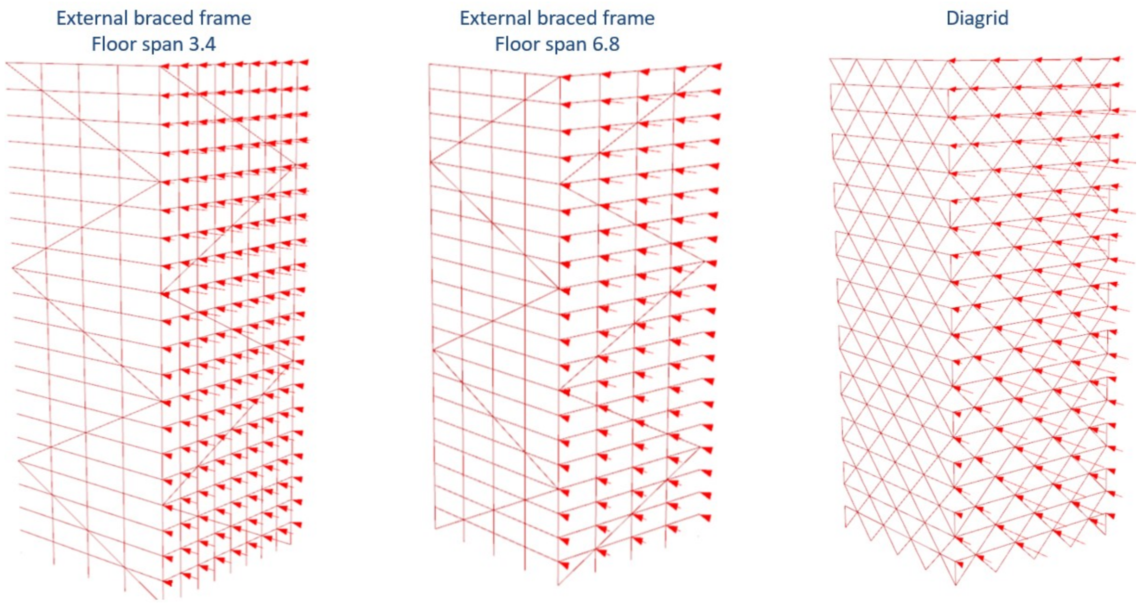


Figure 4.18: Wind loads working on the model

4.3.3. Load combinations

The following classes and load combinations are determined according to NEN-EN 1990+A1+A1/C2/ NB:2019. The consequence class of the building is CC2 as the building height is 68 meters, the indicative design working life is 50 years and has class 3. The load combinations are given below for the ultimate limit state (ULS) and service limit state (SLS). Where the permanent loads are called perm, the imposed loads residencies for non-common floors are called live, the movable walls are called walls and the wind loads are wind. In the parametric model the wind is tested from all four sides to ensure that all the elements are sized correctly.

ULS:

6.10.a:	LCO	perm*1.35 + live*1.5*0.4 + walls*1.5
6.10.b:	LC1	perm*1.2 + live(top 2 floors)*1.5 + live(rest floors)*1.5*0.4 + walls*1.5
	LC2	perm*1.2 + wind*1.5 + live*1.5*0.4 + walls*1.5
	LC3	perm*0.9 + wind*1.5 + walls *1.5
6.11:	LC5	perm*1 + wind*1*0.2 + var*1*0.3 + fire reduction + walls *1.5
	LC5	perm*1 + wind*1*0.2 + fire reduction + walls *1.5

SLS

char:	LC6	perm*1 + wind*1 + walls *1.5
freq:	LC7	perm*1 + wind*1*0.2 + walls *1.5

4.3.4. Flow of forces

Figure 4.19 shows how the flow of normal forces is under vertical loading for the three stability systems. In green the line load on the beams is shown. When a member is under compression the force is shown in orange and a member in tension is shown in blue. In the external braced frame options the vertical load is transferred to the foundation predominantly by the columns and a small share is taken up by the braces. In the diagrid the diagonals are under compression and transfer the vertical loads to the supports. The beams are in tension, this is because of the triangulation of the diagrid module as was discussed in paragraph 2.1.2. In figure 4.20 3D images are shown of the normal forces in the systems under wind loading. The wind load is portrayed with red arrows. One of the head facades is loaded with wind in the figure. Again, a member with a compression force is shown in orange and one with a tension force in blue. The scale of the normal forces is the same for all three stability systems shown. The facade called the head facade is indicated with a H and the side facades with a S. The external braced frame with a single brace transfers the loads to the foundation almost completely through the side facades. In the external braced frame with the double brace the figure shows that the head facade also takes up some of the normal forces. For the diagrid stability system the contribution of the head facade is even more.

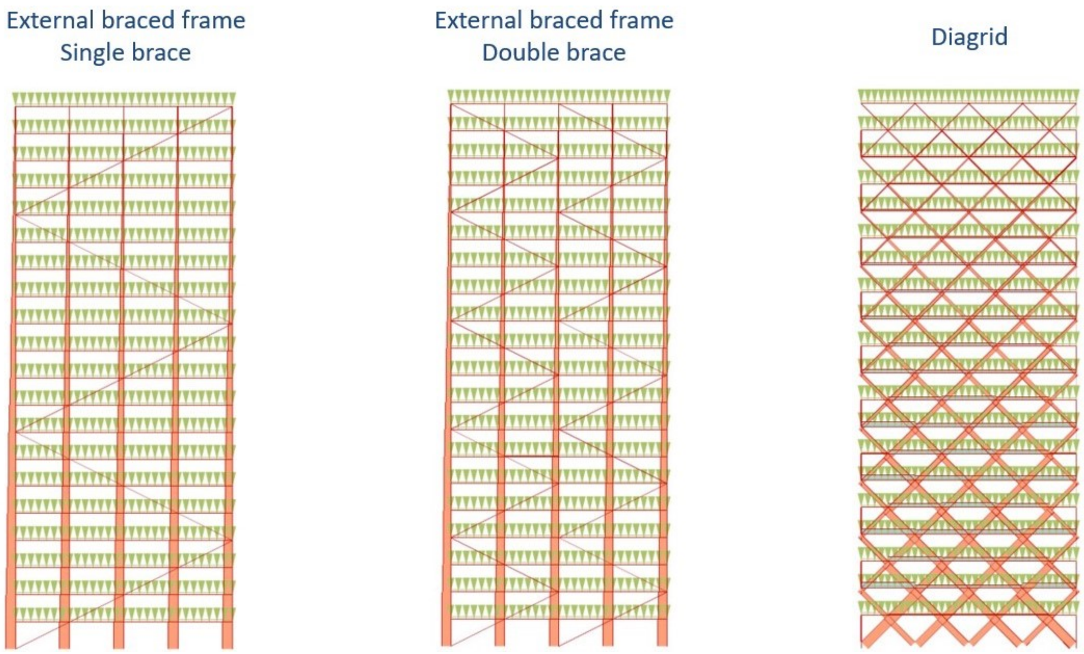


Figure 4.19: Flow of normal forces in stability systems under vertical loading

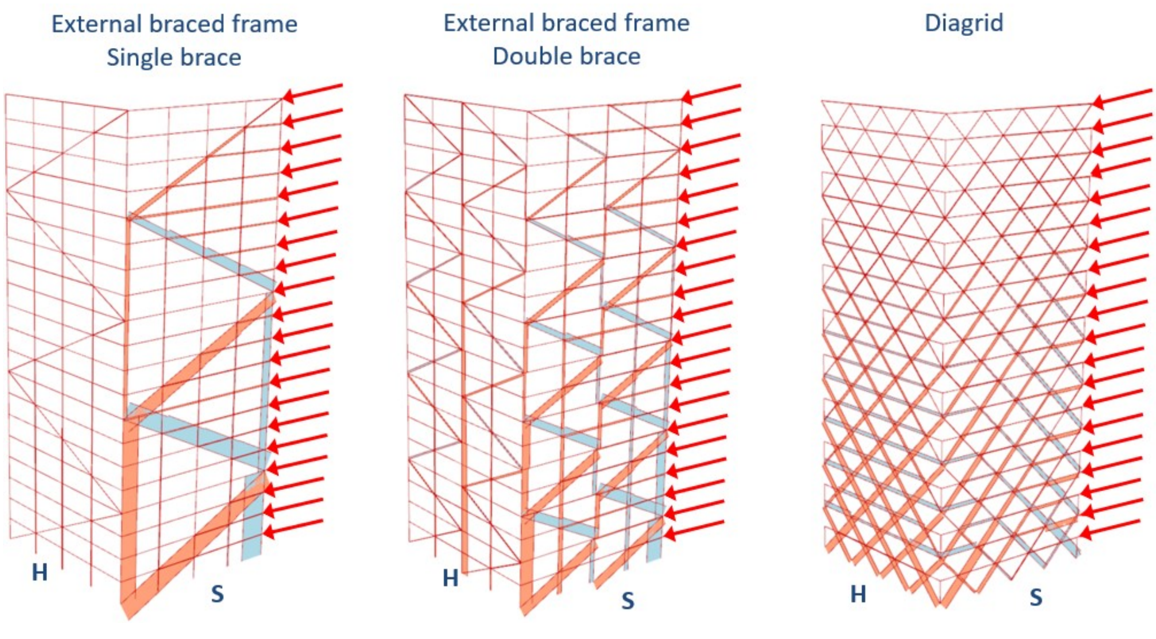


Figure 4.20: Flow of normal forces in stability systems under wind loading

4.4. Design constraints

The sizes of the elements are based on the design constraints. These constraints can be divided in element checks where the ultimate limit state (ULS) loads are applied and global checks where the service limit state (SLS) loads are applied.

4.4.1. Element checks

The full calculations for the element checks of the timber are shown in appendix E. The timber elements are checked on:

- Axial stress
- Shear stress
- Bending stress
- Combined axial and bending stress
- Buckling
- Fire safety

The fire safety of the elements is determined by using the reduced cross-section method. The cross-section of the elements is reduced with 85 mm on every face. For the capacity of the timber the design strength in fire is used. With this reduced cross section and altered strength the timber elements are tested for axial strength, shear stress, bending stress, combined axial and bending stress and buckling under the load combinations for fire.

The connections are checked according to the formulas in appendix B. The fire safety of the connections is provided by encapsulating all steel elements within 85 mm of timber from the reduced cross-section method.

4.4.2. Global checks

The global checks under the service limit state load combinations that are performed are the global horizontal displacement and along-wind acceleration. The along-wind acceleration is determined analytically with the natural frequency. The natural frequency is also determined analytically. The calculations for the natural frequency and along-wind acceleration can be found in appendix D.

Global displacement

The global displacement should be smaller than :

$$u_{max} = h/500 = 68/500 = 0.136\ m = 136\ mm \tag{4.1}$$

Natural frequency

The natural frequency (n) can be determined with the wind load (q), height of the building (h), mass of the building (m) and global horizontal displacement (u_{max}). Oosterhout (1996) developed this method for timber high-rise buildings [78].

$$n = f(\alpha h) * \sqrt{\frac{q}{m} \cdot \frac{h}{u_{\max}}} \tag{4.2}$$

$f(\alpha h)$ is a function that depends on the type of deformation behaviour. For bending $f(\alpha h) = 0.198$ and for shear $f(\alpha h) = 0.176$. As both diagrid and braced frame structures can deform in bending and shear [51] [80] the average of the two values is chosen to determine the natural frequency $f(\alpha h) = 0.187$.

Along-wind acceleration

The along-wind acceleration can be determined with the following formula from NEN-EN 1991-1-4+A1+C2:2011 annex C.

$$a_{\max}(y,z) = c_{\mathrm{f}} \cdot \rho \cdot I_{\mathrm{v}}(z_{\mathrm{s}}) \cdot v_{\mathrm{m}}^2(z_{\mathrm{s}}) \cdot R \cdot \frac{K_{\mathrm{y}} \cdot K_{\mathrm{z}} \cdot \Phi(y,z)}{\mu_{\mathrm{ref}} \cdot \Phi_{\max}} \cdot k_p \tag{4.3}$$

The along-wind acceleration of the building should stay below the limit value of acceleration given by the national annex NEN-EN 1990+A1+A1/C2/NB. This limit value is shown in figure 4.21. For residencies the value is that of gebruik 2 and gebruik 1 is for offices.

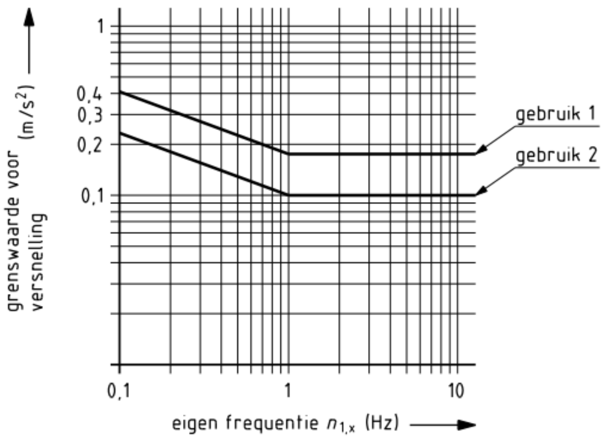


Figure 4.21: Limit value for wind acceleration for occupied spaces in buildings [59]

The maximum acceleration allowed is dependent on the natural frequency of building. That is why the limit is approached with a formula in the parametric model:

$$max(0.1 * n^{-0.34}, 0.1) \tag{4.4}$$

4.5. Exploratory study design for SLS

The final step of the model definition is to define how the global stiffness will be increased in the parametric model when the SLS requirements are not met. The SLS requirements exist of the global displacement and the acceleration. To meet the global displacement requirement the global stiffness of the stability system needs to be increased. To meet the acceleration requirement the global stiffness can be increased but also the weight of the building can be increased. When material is added to the stability system only to increase the weight and not the global stiffness it is not seen as an efficient use of the material. As well as, increasing the weight of the building is already accounted for with the parameter of the permanent floor load. That is why an exploratory study is performed to find how the global stiffness can be increased in a material efficient way. In this study the effect of increasing the timber height, which increases the stiffness of the timber member, is researched per element type. The increase of the connection stiffness by increasing the number of bolts is also studied per element type. However, it is chosen to only increase the timber element height in the parametric model and not the connection stiffness as this would increase the options in the parametric model significantly.

This exploratory study is done by taking some first results of the parametric model. The single brace, double brace and diagrid stability system design are all considered. The results are taken from the design options with a plot size 27.2 x 27.2 m, floor span 6.8 m, permanent floor load 1 & 2 and element width of 400, 450, 500, 550, 600 and 650. So, per stability system design 12 (2x6) options are collected. In the tables with results the average value of the 12 options is shown. First the average displacement and material usage for the 12 design options after the ULS member checks is determined. Then the decrease in displacement (Δ d) and increase in material (Δ Steel or Δ Timber) are determined for each options with increased stiffness. To find the influence of the decreased displacement per increased material the Δ d is divided by the Δ Steel or Δ Timber. A higher number indicates more material efficiency.

The options with increased material that are studied are:

- Multiplying the number of rows of bolts (n_{row}) in the braces (in the external braced frame) or the diagonals (in the diagrid) by 2
- Multiplying the number of rows of bolts (n_{row}) in the beams by 2
- Multiplying the number of rows of bolts (n_{row}) in the columns by 2
- Multiplying the number of rows of bolts (n_{row}) in all elements by 1.4, these numbers are rounded
- Multiplying the timber element height (h) of the braces or diagonals by 2
- Multiplying the timber element height (h) of the beams by 2
- Multiplying the timber element height (h) of the columns by 2
- Multiplying the timber element height (h) of all elements by 1.33

4.5.1. Single brace

Increase rows of bolts

	Brace * 2	Beam * 2	Column * 2	All * 1.4
Δ d (mm)	0	0.64	0	0.28
Δ Steel (m^3)	0.91	0.38	0.88	0.79
Δ d / Δ Steel	0	1.71	0	0.36

Increase timber element height (h)

	Brace * 2	Beam * 2	Column * 2	All * 1.4
Δ d (mm)	11.12	0.09	6.50	8.90
Δ Timber (m^3)	155	364	347	288
Δ d / Δ Timber	0.072	0	0.019	0.031

In the single brace increasing the rows of bolts of the brace and column does not influence the displacement. Increasing the timber element height of the beam also has a very small influence of the displacement. To give the single brace a higher global stiffness the connections of the beam or the element size of the brace and column can be increased.

4.5.2. Double brace

Increase rows of bolts

	Brace * 2	Beam * 2	Column * 2	All * 1.4
Δ d (mm)	4.93	0.51	0	4.05
Δ Steel (m^3)	0.94	0.35	0.94	0.868
Δ d / Δ Steel	5.26	1.45	0	4.67

Increase timber element height (h)

	Brace * 2	Beam * 2	Column * 2	All * 1.4
Δ d (mm)	7.71	0.09	4.40	5.56
Δ Timber (m^3)	216	377	356	316
Δ d / Δ Timber	0.036	0	0.012	0.018

In the double brace the increase of the connection stiffness of the columns and the increase of the beam height have little or no influence on the displacement. To increase the global stiffness the connections of the brace and beam and the timber element height of the brace and column can be increased.

4.5.3. Diagrid

Increase rows of bolts

	Diagonal * 2	Beam * 2	Column * 2	All * 1.4
Δ d (mm)	8.23	0.20	0	4.67
Δ Steel (m^3)	10.59	4.60	0.04	7.49
Δ d / Δ Steel	0.78	0.04	0	0.62

Increase timber element height (h)

	Diagonal * 2	Beam * 2	Column * 2	All * 1.4
Δ d (mm)	8.36	2.42	0	5.36
Δ Timber (m^3)	216	377	36	316
Δ d / Δ Timber	0.039	0.0064	0	0.017

In the diagrid increasing the connection of the column and increasing the element height of the column does not change the global displacement. To increase the global stiffness the connections of the diagonals and the element heights of the diagonals and columns can be increased. The increase of the beam connections also decreases the displacement. However, its influence per steel increase is 19.5 times smaller than that of the diagonal connection, so the effect is seen as insignificant.

4.5.4. Discussion

Now the results of the exploratory will be discussed for a single braced frame design to demonstrate the relationship between the timber element stiffness and connection stiffness. In figure 4.22 a timber element is shown with a steel connection. The timber element will act as a spring with a stiffness (K_{timber}) and the connection will also act as a spring with a stiffness $(K_{connection})$. When the timber stiffness is higher than the connection stiffness, the connection will cause more elongation of the element and global displacement than the timber.

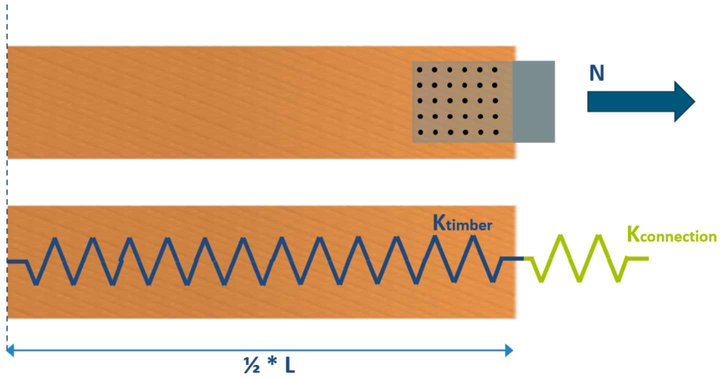


Figure 4.22: Timber element with steel connection acting as springs

Figure 4.23 shows three average designs for a brace, beam and column. These average designs are taken from the single brace design with a plot size of 27.2 x 27.2, floor span 6.8, permanent floor load 2 and element width 400 mm. The average design of a brace has a height of 950 mm and 63 bolts, a beam has a height of 700 mm and 8 bolts and a column has a height of 1650 mm and 120 bolts.

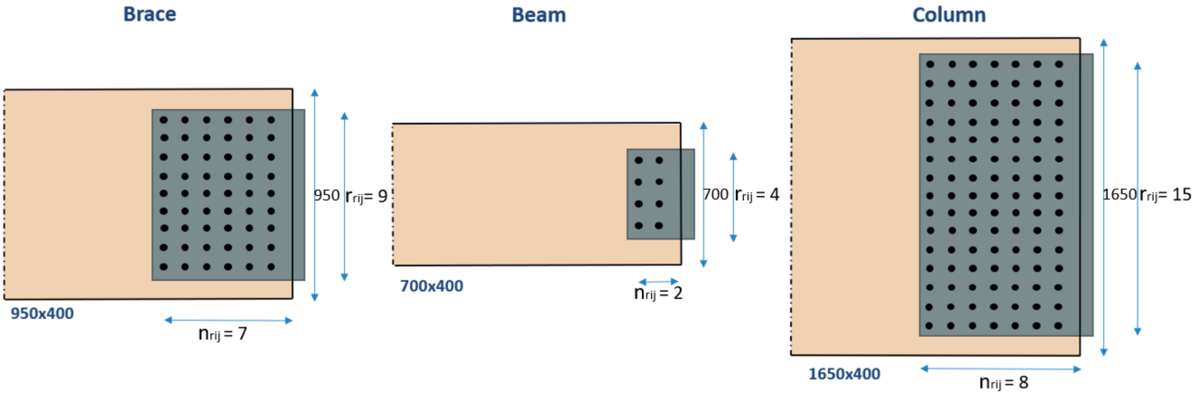


Figure 4.23: Average element designs for a single brace design option

To calculate the spring stiffness of the connection the following formula from Eurocode 3 part 8 chapter 3 [56] can be used:

Stiffness per dowel per shear plane:

$$K_{connection} = \frac{\rho_{\text{mean}}{}^{1.5} * d}{23} * 2 = \frac{430^{1.5} * 16}{23} * 2 = 12406 \text{ N/mm} \tag{4.5}$$

There are 4 shear planes per dowel:

$$K_{connection} = 12406 * 4 = 49623 \text{ N/mm}$$

To calculate the axial stiffness of the timber the following formula can be used:

$$K_{timber} = \frac{E * A}{L} \tag{4.6}$$

For timber GL28c $E = 12500 N/mm^2$. The half length of a brace is 3800 mm, a beam is 3400 mm and a column 6800 mm. Using the formulas, the amount of dowels per element, element sizes and lengths we get the following stiffnesses:

Brace	
Timber	1250000 N/mm
Connection	13126262 N/mm

Beam	
Timber	1029412 N/mm
Connection	396986 N/mm

Column	
Timber	1213253 N/mm
Connection	5954784 N/mm

From the exploratory study for the single brace in paragraph 4.5.1 it was seen that the an increase of the connection stiffness of the brace has no influence on the displacement. Increasing the timber element height of the brace did have a positive influence on the displacement. This is in agreement with the results shown in the table above where it can be seen that the connection stiffness is around 10x larger than the timber stiffness. For the beam the opposite behaviour was observed, where increasing the connection stiffness had an influence and increasing the timber stiffness had no influence. It can be seen that the timber of the beam has a 2.5x larger stiffness than the connection. In the column again increasing the timber stiffness has an influence similar to the brace. Here the connection stiffness is almost 5x larger than the timber stiffness.

4.6. Conclusion

In this chapter the input for the parametric model was defined. First the global design parameters were defined and based on that stability system designs were created. From these stability system designs the three most material efficient were chosen. Then, the loads and design constraints that are put in the model are discussed. Lastly, the approach for increasing the global stiffness for the SLS checks is defined by answering the sub-question:

How can the global stiffness of an externally braced stability system be increased in a material efficient way?

Parameters

The global design parameters can be divided in fixed design choices and parameters.

The fixed design choices are:

- Timber element size: Minimum height of 250 mm
- Building height: 68 m with 20 storeys
- Foundation stiffness: Determined by the capacity of the foundation piles not a parameter in the model
- Angle of bracing: 90°for the diagrid and a slope of 1:2 for the external braced frame designs

The parameters in the model are:

- Timber element size: Six options for the width: 400 mm, 450 mm, 500 mm, 550 mm, 600 mm and 650 mm
- Slenderness of the building: Two plot sizes: 27.2 x 27.2 m and 27.2 x 40.8 m

- Floor span: Two floor spans: 3.4 m and 6.8 m
- Permanent floor load: Three permanent floor loads: 3.5 kN/m^2, 5.3 kN/m^2, 6.7 kN/m^2. The permanent floor load of 3.5 kN/m^2 will be called load 1, the load of 5.3 kN/m^2 will be referred to as load 2 and the permanent floor load of 6.7 kN/m^2 will be called load 3.

Stability system designs

The three chosen stability system designs are shown in figure 4.24. The first design is a single brace with slope 1:2. The second is the double brace with a slope of 1:2 and the last design is a diagrid with ring beams every floor. In the figure the stability systems are shown all with different parameters for the plot size and floor span.

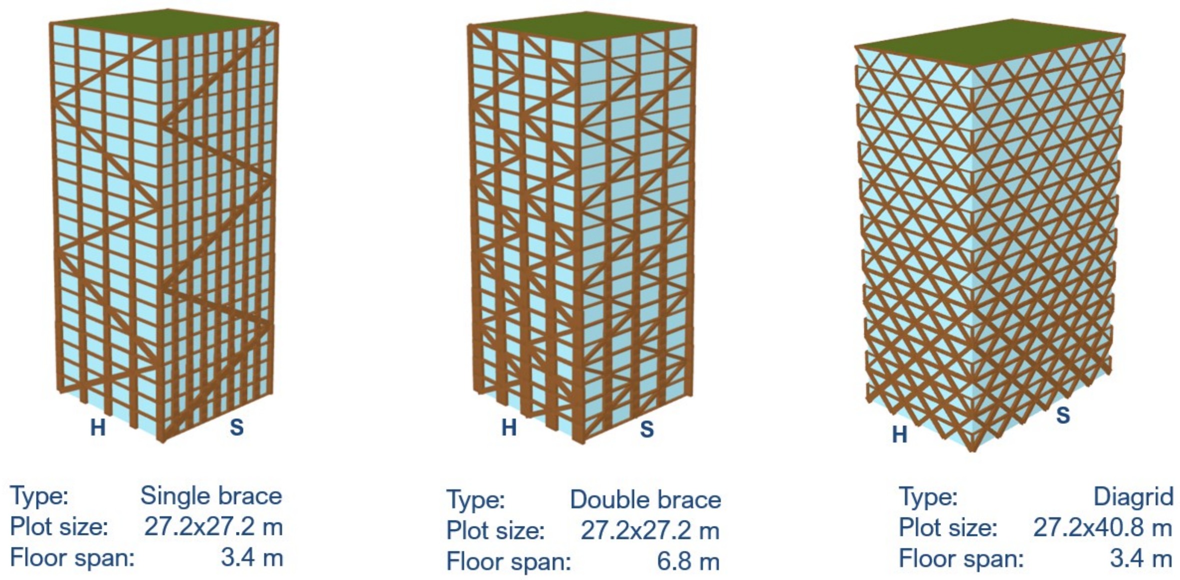


Figure 4.24: Three examples of final designs

Increase global stiffness for SLS

The global stiffness of an externally braced stability system can be increased either by increasing the connection stiffness or increasing the stiffness of the timber elements. It is found that in the single braced frame increasing the stiffness of the beam connection, or increasing the timber stiffness of the brace and column would increase the global stiffness. For the double braced frame increasing the stiffness of the brace and beam connection and increasing the timber stiffness of the brace and column would improve the global stiffness. In the diagrid increasing the connection stiffness of the diagonals and increasing the timber stiffness of the diagonals and beams increase the global stiffness.

For this research it was chosen to only increase the timber element sizes in the parametric model. To increase the global stability in a material efficient way the timber elements that do not influence the global displacement, and therefore do not increase the global stiffness, will not be increased in the parametric model. For the external braced frame systems increasing the beam stiffness had no influence on the displacement but did add a lot of timber. For this reason only the brace and columns will be increased. In the diagrid increasing the columns had no influence so only the diagonal and beam elements will be increased.

Overview parametric model

In figure 4.25 an overview is given of the parametric model with all parameters and design constraints for which checks will be performed. The amount of design options is 3x stability system design, 6x timber element size, 2x plot size, 2x floor span and 3x permanent floor load. This gives us 216 design options that will be studied in the parametric model.

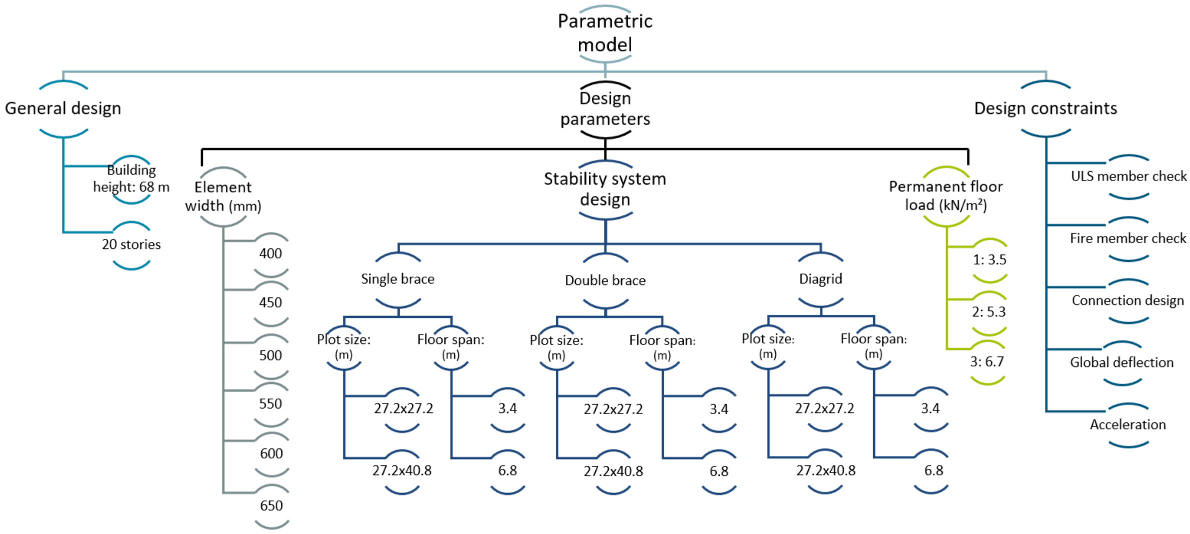


Figure 4.25: Parametric model overview

5

Parametric model

A parametric model is made to be able to study and compare many different building designs and see what parameters will give the most material efficient design. Parametric studies are useful as they can generate preliminary designs for stability systems quickly. The parametric model is made in Grasshopper which is a visual programming software for the 3D modeling program Rhino. In the model a grasshopper plug-in for structural engineering called Karamba 3D is used that can analyze entire structural models. The plug-in Beaver that can analyze timber elements, and the plug-in Colibri that can collect results for a parametric study are used as well. To create the component that can calculate the connection design and to calculate the along-wind acceleration a python plug-in is used.

The parametric model that will be made for this study will size the timber members of the stability system and generate a slotted-in steel plate connection design for every timber element individually. In figure 5.1 the design process of the parametric model is shown. First, all the parameters are selected to create the building geometry. Then, the stability system is designed based on the selected parameters. Next the timber elements will be sized and the connection designs will be created. Afterwards the global SLS checks will be performed and if the requirements are not met the timber element sizes will be increased.

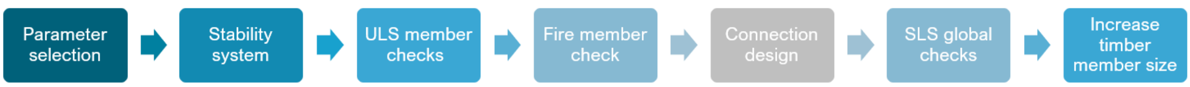


Figure 5.1: Design process parametric model

In this chapter the workflow of the parametric model will be explained in detail together with all the assumptions made. The workflow of the parametric model is divided into nine steps that will be discussed individually. Before the workflow will be discussed a table with all the assumptions made per step will be shown together with where it is explained in the thesis.

5.1. Assumptions

Step	Assumption	Described in paragraph
Step 1: Geometry & assemble	- All connections are modelled as hinges with an axial stiffness	2.2.7
	- Timber material is GL28c, steel plates are S355 and dowels are 8.8	4.1.1
	- Floor load transfers to head façade as line load	4.3.1
	- Internal beams transfer floor loads as point loads to side façade	4.3.1
	- 1.2 kN/m^2 imposed load from moveable walls	4.3.2
	- All live loads are imposed loads residencies for non-common floors of 1.75 kN/m^2	4.3.2
	- Façade load as line load on the beams with value of 3.4 kN/m	4.3.1
	- Wind load as point load on joints	4.3.2
	- Wind load tested from all four sides	4.3.3
	- Accidental load combination for fire situation with reduced cross-section method	4.4.1
	- In external braced frame models the columns continue for four floors, the braces are split when they meet a column, and the beams are split if they meet a column or beam	4.2.1
	- In diagrid the longest diagonals span 3 floors, the beams are split when they encounter a diagonal, and the columns only span one floor	4.2.2
	- The internal structure with beams and floors is not modelled	
	- The head facades are the same but mirrored around the plane perpendicular to the facade	
	- The side facades are the same but mirrored around the plane perpendicular to the facade	
Step 2: Optimize cross-section Karamba	- The unity check is set to 1.4	5.2.2
	- The timber element height is increased in steps of 50 mm	5.2.2
Step 3: ULS member check	- The bending and normal force component from the plug-in beaver is used	5.2.3
	- The largest normal force and moment are determined from all ULS-combinations	5.2.3
	- The timber element height is increased with steps of 50 mm	5.2.3
Step 4: Fire member check	- The bending & normal force component from the plug-in beaver is used	5.2.4
	- The largest normal force and moment are determined from the load combination for the accidental fire situation	5.2.4
	- The reduced cross-section method with a reduction of 85 mm from all four element sides is used	E.2
	- The material design strength for a fire situation is used	E.2
	- The timber element height is increased with steps of 50 mm	5.2.4
Step 5: Connection design	- The draft version of Eurocode 5 is used to determine the block shear capacity	3.1 & B
	- All steel is embedded within 85 mm of timber	4.1.2 & 5.2.5

	- Connection design is based on the normal force, small vertical forces on beams also considered as normal force	5.2.5
	All connections have the following defined connection parameters: - Bolt diameter of $d = 16\,mm$ - Edge distance of the steel plate $e_2 = 25\,mm$ - Two steel plates are used $n_{plate} = 2$ - Plate thickness of $t_{plate} = 10\,mm$ - Edge distance of the timber $a_3 = 112\,mm$ - Timber thickness of the outer part is $t_1 = 40$ mm - Distance between the bolts in direction of the grain $a_1 = 80\,mm$ - Timber thickness of the inner par t_2 is derived from the left-over space between the steel plates - The distance between the bolts in direction perpendicular to the grain a_2 is derived by dividing the width of the steel plate by 100 mm and then rounding this number to find how many rows of bolts r_{row} fit - The maximum number of rows in direction of the grain $n_{row} = 15$ - The timber element height is increased with steps of 50 mm - The axial connection stiffness is determined for the SLS situation	3.2
Step 6: Reassemble Karamba model	- The foundation stiffness based on the required amount of foundation piles. One pile is a vertical stiffness of 150 MN/m^2	4.1.8
	- Average axial connection stiffness determined per element group and facade	5.2.6
Step 7: SLS global checks	- Natural frequency determined analytically with the formula	4.4.2
	- Annex C of NEN 1991-1-4+A1+C2:2011 used to determine the acceleration of the building	2.4.1
	- Damping ration of $\xi = 1.9\%$ & structural logarithmic $\delta_s = 0.12$	2.4.1
	- 30% of live load added to total weight of the building	2.4.1
	- Acceleration requirement for the Netherlands used	4.4.2
	- Acceleration requirement approached with formula $\max(0.1 * n^{-0.34}, 0.1)$	4.4.2
	- Slip in connections considered	2.5
	- Only the timber element heights will be increased	4.5
	- In plot size 27.2 x 27.2 m both facades will be improved	5.2.7
	- In plot size 27.2 x 40.8 m only the head façade with length 27.2 m will be improved	5.2.7
	- In the external braced frame, the columns and braces will be increased together in steps of 10%	4.5
	- In the diagrid the diagonals and beams will be increased together in steps of 10%	4.5
Step 8: Generate results	- The timber and steel in the internal structure are calculated for the different design options. These values are added up to the amount of steel and timber in the stability system to compare the different plot sizes, floor spans and floor loads	5.2.8
	- The steel in the 'crossing' parts is considered	5.2.8

5.2. Model workflow

The parametric model is divided into nine steps. How these steps are connected can be seen in figure 5.2. In the first step the model will be assembled in a karamba model. In the second step the timber members will be sized individually within the karamba model by a karamba component. After this step the karamba model will be disassembled. For each member the size and forces are gathered from the disassembled model. These will be used to perform the ULS element checks in step 3,4 and 5. Then in step 6 the complete model is reassembled to perform the global SLS checks. In step 7 the global checks are performed. All the results of the designs will be collected after the global checks to see what stability system need to have their global stiffness increased after the ULS design. The options that do not satisfy then have their element heights increased in steps of 10% until they meet all requirements. Then in step 8 the results are generated in grasshopper. All results will be collected with colibri and afterwards excel is used to group the correct results.

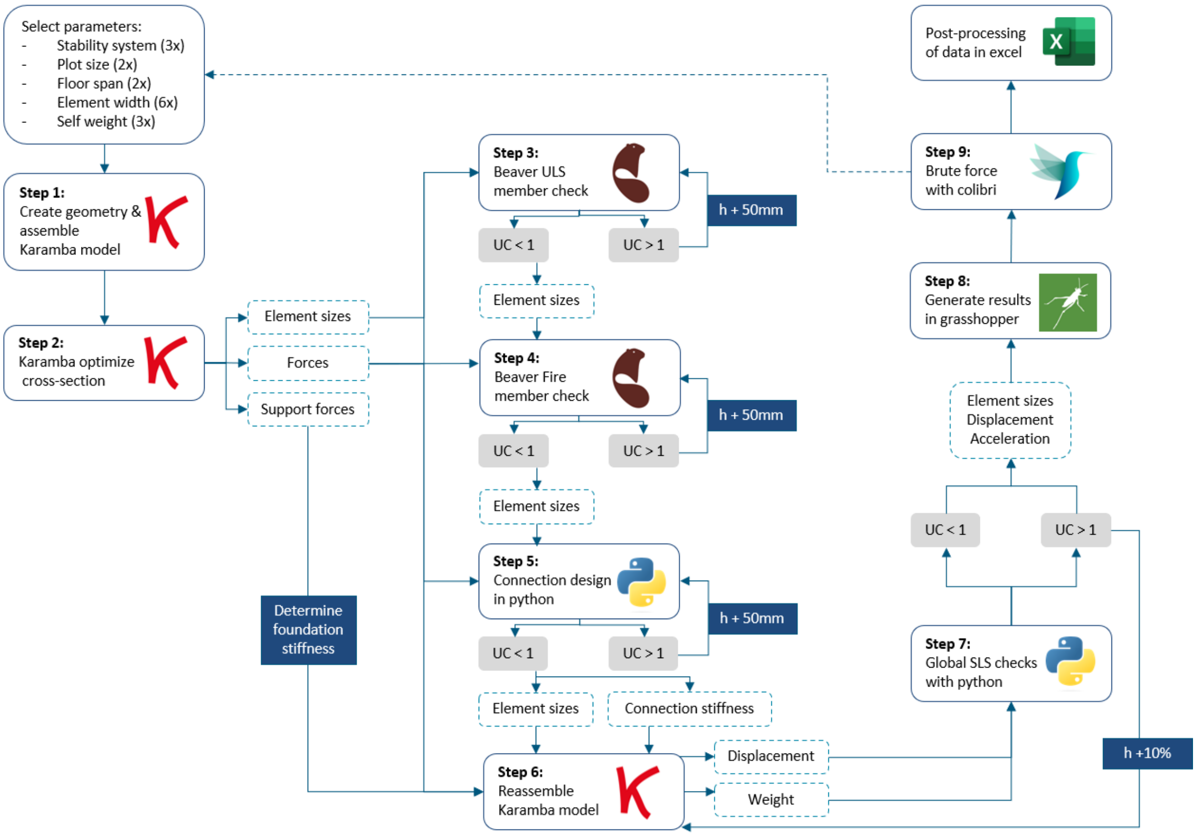


Figure 5.2: Parametric model workflow

5.2.1. Step 1: Geometry & assemble

In the first step the geometry of the chosen design option is created. This geometry together with all the load combinations, the timber cross-sections list and the material properties of GL28c are assembled in a 3D karamba model. Karamba can then analyse the model to give the forces present in each member. Figure 5.3 shows how first the model is assembled and afterwards analysed. The last part of the figure shows step 2.

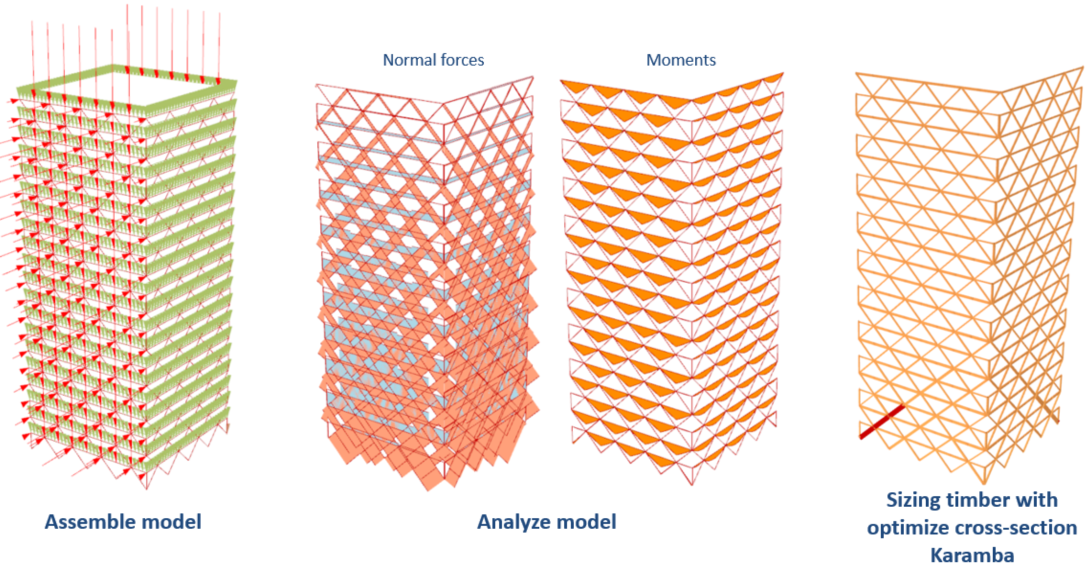


Figure 5.3: Step 1 and 2 of the parametric model

5.2.2. Step 2: Optimize cross-section Karamba

Next the model is put through the optimize cross-section tool of Karamba. This tool checks the members on axial stress, shear stress, bending stress, combined stresses and buckling for all load combinations. This component uses Eurocode 3 for steel to determine if the members satisfy the unity checks but with the material properties of GL28c timber. The maximum utilisation of the component is set to 1.4 to make sure the timber elements are not overdimensioned. This step is useful as it gives starting member sizes that are closer to the required member sizes. Consequently, the speed of the model is increased.

5.2.3. Step 3: ULS member check

Figure 5.4: Member forces

The karamba model is disassembled and for every member the maximum normal force and moment for the ULS load combinations is found. Figure 5.4 shows an example of this for a long diagonal member that spans three floors. The maximum normal force, maximum moment, member size and buckling length are put in to the bending and normal forces component of Beaver. This component checks the axial stress, shear stress, bending stress, combined stresses and buckling for timber elements based on Eurocode 5 for timber. The formulas used in these checks can be found in appendix E.1. Per member the height from step 2 is made into a list of 15 values where the height is increased with 50 mm for each value. Figure 5.5 shows an example where a member with a height of 500 mm is made into a list. The member heights from this list are then checked with the Beaver component which will give a unity check value. If the value is 1 or lower the member height satisfies. From this list of unity checks per member the smallest height that satisfies the unity check is selected as the new member height. In the example in figure 5.5 this would be 600 mm.

h(mm)	UC
500	1.2
550	1.1
600	1.0
650	0.9
700	0.8
750	0.7
800	0.6

Figure 5.5: Element height (h) list and unity check (UC) value

5.2.4. Step 4: Fire member check

The maximum moments and normal forces are determined for the accidental load combination in the case of fire. For this step the beaver component for bending and normal forces is used once more. The material stiffness is altered for the fire situation. The design strength during fire is determined by using a modification factor for fire as shown in appendix E.2. The member sizes are reduced by 85 mm from the reduced cross-section method on all four sides of the member. The member heights minus the reduced cross-section from step 3 are again made into lists where 50 mm is added per step and the smallest height that satisfies the unity checks is selected. This is the same method as used in step 3.

5.2.5. Step 5: Connection design

The connection design in the parametric model is made by a python component. The input for this component are the ULS forces (N), timber element heights from step 4 (h) and the timber element widths are used. For the columns, braces and diagonals only the normal force is checked to get the maximum load on the connection. For the beams in the model the normal force as well as the shear force are checked to determine the maximum load on the connection as shown in figure 5.6. However, all loads are seen as normal forces in the axial direction of the element when calculating the connection design. Normally when the force working on the connection is under an angle the characteristic embedment strength of the timber is reduced. The maximum shear forces in the beams are never larger than 156 kN. This maximum force results in fairly small connections with 6 bolts or less, depending on the timber member size. Overall this assumptions is expected to cause a small underestimation of the amount of steel used for the beams as the connection designs affected use a relatively small amount of steel so the possible increase would also be small.

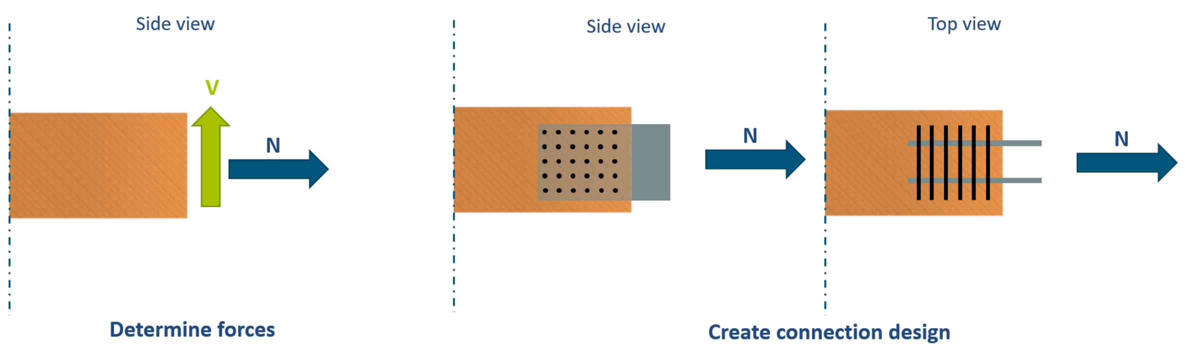


Figure 5.6: Step 5 of the parametric model

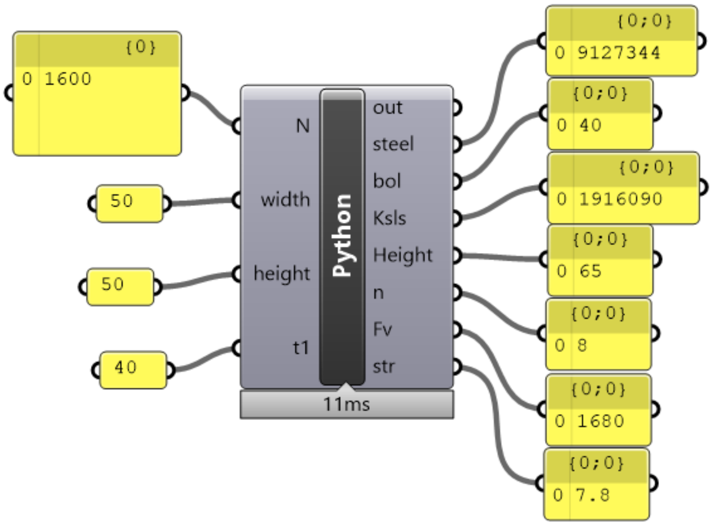


Figure 5.7: Connection design python component

The parametric model will have to determine the connection design for every timber element and be able to increase the element height of the timber if the largest connection that fits has too little capacity. The component is created with the programming language python. The formulas used in the python script are all shown in appendix B. To calculate the block shear capacity only the draft version from Eurocode 5 will be used [16]. Figure 5.7 shows the python component with an example calculation. This calculation will be discussed to explain the component. In table 5.1 the input for the component is shown with a description and unit. In table 5.2 the same is shown for the output of the component.

	Description	Value	Unit
N	Normal force	1600	kN
width	Timber width	50	cm
height	Timber height	50	cm
t_1	Thickness outer timber part	40	mm

Table 5.1: Input of the connection design component

	Description	Value	Unit
steel	Amount of steel in connection	9127344	mm^3
bol	Number of bolts	40	-
Ksls	Axial stiffness of the connection	1916090	N/mm
Height	Timber height	65	cm
n	Number of rows n_{row}	8	-
Fv	Connection capacity	1680	kN
str	Allowed stress in the timber element	7.8	N/mm^2

Table 5.2: Output of the connection design component

Steps of the connection design component:

1. **Input general:** The material properties of timber GL28c, steel plate S355 and steel dowels 8.8 are input. The modification factors $k_{mod} = 0.9$ for wind and $\gamma_m = 1.3$ for connections are input and the design strengths are calculated.
2. **Input connection design:** The connection parameters chosen in the exploratory study are input in the script. These values are $d = 16\ mm$, $e_2 = 25\ mm$, $t_{plate} = 10\ mm$, $a_3 = 112\ mm$, $n_{plate} = 2$, $t_1 = 40\ mm$, $a_1 = 80\ mm$.
3. **Calculate t_2:** As can be seen in figure 5.8 the thickness of the inner timber part is calculated by subtracting the plate thicknesses, the $85\ mm$ from the reduced cross-section and t_1 of $40\ mm$ this gives a timber thickness of $230\ mm$ for t_2.

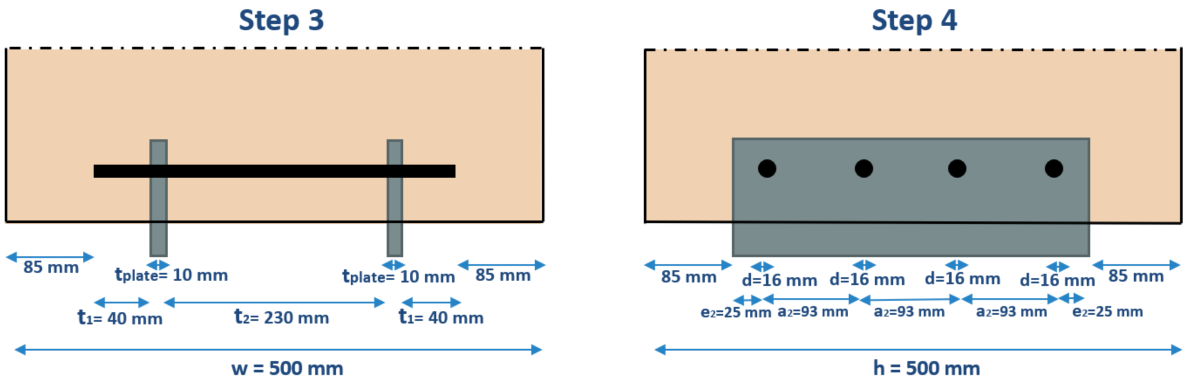


Figure 5.8: Step three and four of the connection design component

4. **Calculate r_{row} & a_2:** The component will first check if the element height is smaller than $320\ mm$ as this will give only one row r_{row}. This is checked since the calculation will differ slightly as there is no value for a_2. In the case of the example connection the element height is larger than $320\ mm$ so now the number of rows r_{row} will be calculated. This is also shown in figure 5.8. The $85\ mm$ from the reduced cross section method and the end distance e_2 are subtracted from each side of the element. For an element with a height of $500\ mm$ this left over space is $280\ mm$. The left over space is divided by $100\ mm$ giving 2.8 and then rounded to 3. This means that the area should be divided in to three parts meaning that there should be 4 rows of bolts so $+1$ is added. Then the left over space is divided by 3 giving a value of $93\ mm$ for a_2.

5. **Johansen model check:** The failure modes of the Johansen model are calculated according to the formulas used in appendix B.

6. **Timber block shear check:** The timber block shear capacity is calculated according to the draft for Eurocode 5. These calculations can be found in appendix B. In this step the effective thickness of the outer parts is dependent on the normative failure modes of the Johansen model. If the normative failure mode is failure mode H from figure 3.1 $t_{ef} = t_1$ and if another failure mode is normative $t_{ef} = 0.65 * t_1$.

7. **Steel elements check:** The shear resistance of the bolt, the bearing resistance of the plate and the block tearing resistance of the plate are calculated according to the formulas in appendix B.

8. **Capacity:** The smallest, and therefore normative capacity of all the checks is determined. The unity check of the connection is then performed. If the capacity of the connection $F_{connection}$ in figure 5.9 is smaller then the force on the connection N the unity check will be larger than 1 meaning that the rows of bolts needs to be increased. If the unity check is smaller than 1 the connection design satisfies and no rows will be added. For the connection from the example the number of rows needs to be increased.

9. **Increase number of rows n_{row}:** Increasing the number of rows is the innerloop of the component as seen in figure 5.9. The number of rows the component can choose from is 1,3,5,8,12 and 15. Not all rows from 1 to 15 are chosen as this would decrease the speed of the model. The capacity of the connection is calculated for all number of rows. If the unity check is larger than 1 with 15 rows the element height needs to be increased. For the connection from the example this is the case.

10. **Increase timber element height h:** The element height is increased in steps of $50\ mm$ and can be increased 40 times. This is done to ensure a connection design with a sufficient unity check. In the example connection the output element height is $650\ mm$ which means that the element needed to be increased in height three times. A unity check of 0.95 is reached with eight rows of bolts n_{row} and a total of 40 bolts meaning r_{row} is five.

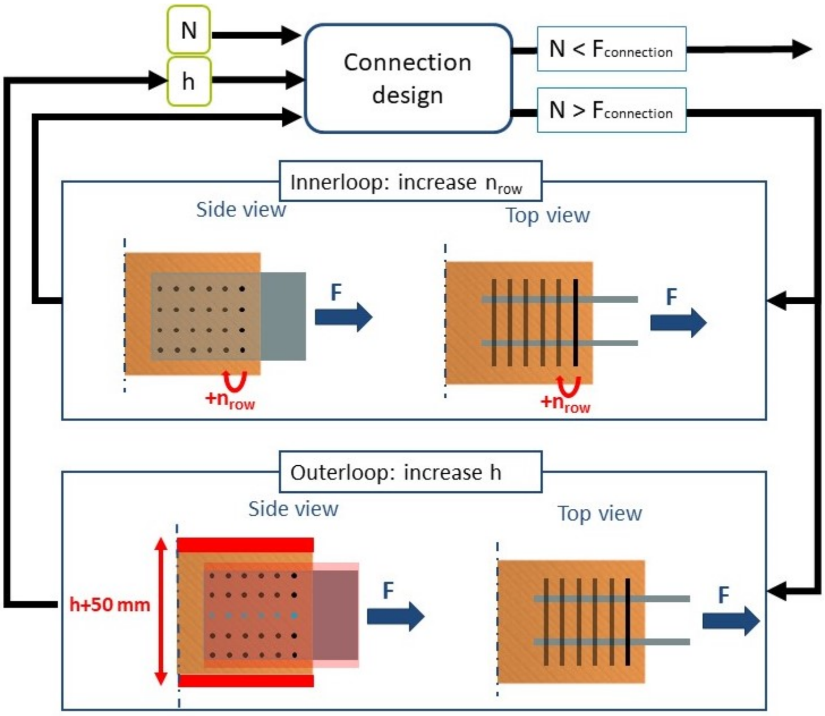


Figure 5.9: Double loop process of step 9 and 10 of the connection component

11. **Axial stiffness of the connection:** When the connection design has sufficient capacity the axial stiffness or slip of the connection is calculated for the SLS situation according to appendix B.

12. **Collect results:** The final step of the component will gather the results for every connection design. The total amount of steel in the connection is calculated based on the dimensions of the steel plates and the number of bolts. The axial stiffness of the connections and the new element heights will serve as input for the next steps in the parametric model.

5.2.6. Step 6: Reassemble Karamba model

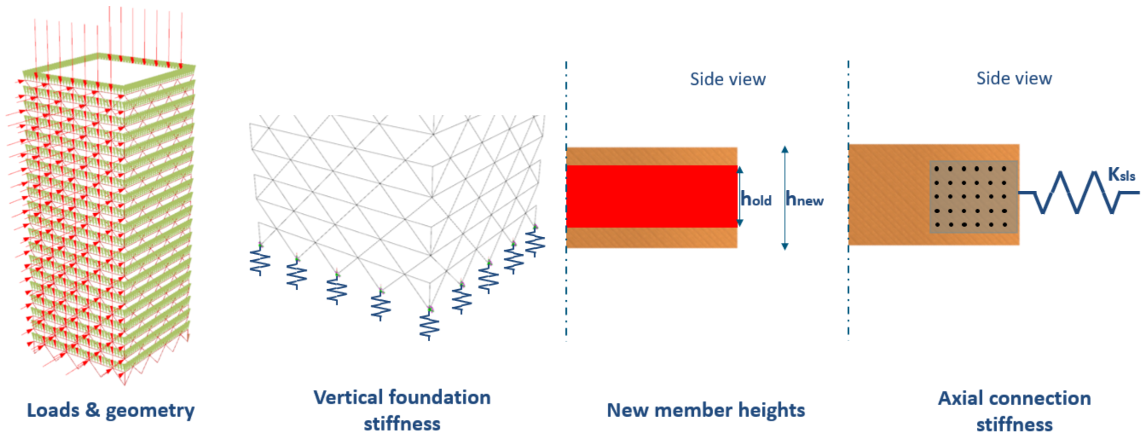


Figure 5.10: Step 6 of the parametric model

After performing the element checks the karamba model needs to be reassembled to perform the global SLS checks. As can be seen in figure 5.10, first the geometry and loads are gathered from the disassembled karamba model from step 2. Other information gathered from the model is the maximum reaction forces at the supports. Based on the reaction force in z-direction it is determined how many foundation piles with a capacity of 3000 kN are needed. Per foundation pile a vertical foundation stiffness of 150 MN/m^2 is applied and this stiffness is put in to the model. Paragraph 4.1.8. showed how many piles are used at what location. Next the member heights are altered to the new member heights found after step 5. The connection stiffnesses for the new model are determined by calculating the average connection stiffness per element group. The element groups are divided into head and side facade and per element type. The element types for the diagrid are beams, diagonals that span three floors, diagonals that span two floors, diagonals that span a single floor, columns and the ends of the beams. The ends are pointed out with red arrows in figure 5.11. For the external braced frame the element types are columns, beams and braces. After this step there is a 3D-karamba model with the defined loads & geometry, new foundation stiffness, new element heights that are sized based on the ULS element checks and average connection stiffnesses for each element group.

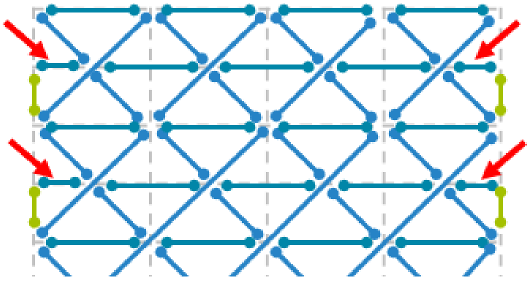


Figure 5.11: Beam ends in diagrid

5.2.7. Step 7: SLS global checks

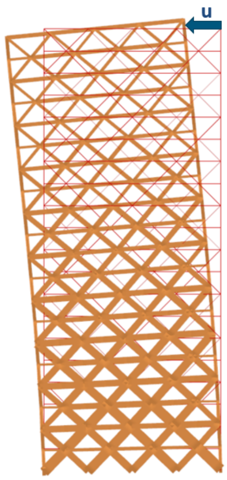


Figure 5.12: Global displacement

In this step the assembled karamba model is analyzed with the karamba component called analyze. First the global displacement of the structure is determined. This displacement (u) is checked to see if it is smaller than the maximum allowed displacement of $68000/500 = 136\ mm$. The weight of the stability system is gathered from the reassembled model in step 6. To calculate the modal mass of the building 30% of the live load, the facade weight and the self weight of the floors including sand, topping and moveable wall loading are added to the weight of the stability system. The weight of the timber in the internal structure is also added by multiplying the amount of used timber by the density of GL28c. The total mass of the building that is added to the mass of the stability system in the facade is given in table 5.3. The amount of timber in the internal system used to calculate the mass will be discussed in the next paragraph 5.2.8.

Plot size - floor span	Floor load 1	Floor load 2	Floor load 3
27.2x27.2 - 3.4	$8.62 * 10^6$	$1.16 * 10^7$	$1.41 * 10^7$
27.2x27.2 - 6.8	$8.59 * 10^6$	$1.14 * 10^7$	$1.37 * 10^7$
27.2x40.8 - 3.4	$1.28 * 10^7$	$1.73 * 10^7$	$2.07 * 10^7$
27.2x40.8 - 6.8	$1.26 * 10^7$	$1.70 * 10^7$	$2.04 * 10^7$

Table 5.3: Mass of the building excluding the stability system in kg

With the mass of the building, the wind load and the displacement, the natural frequency of the building is determined for the along-wind direction. Afterwards the acceleration is calculated with the natural frequency and the modal mass. The method to calculate the natural frequency and the acceleration are discussed in paragraph 4.4.2 and appendix D. The acceleration is then checked to see if it is smaller than the acceleration limit.

When one of the two SLS requirements is not met the element heights will be increased in steps of 10%. For the external braced frame options only the heights of the columns and braces will be increased. For the diagrid only the heights of the beams and diagonals will be increased. In the square plot size of 27.2 x 27.2 m the elements of both the side and head facades will be increased. For the rectangular plot size of 27.2 x 40.8 m only the short head facades will be increased as they are determining for the maximum global displacement.

5.2.8. Step 8: Generate results

The results of the parametric model will be the amount of material that is used in the buildings as well as the displacement, acceleration and acceleration requirement. The displacement, acceleration and acceleration requirement can be gathered directly from step 7. How the material usage is determined will be discussed now.

Timber

The results for the amount of material used will include the amount of timber in the stability system after each sizing step of the parametric model. These steps are the ULS member check, the fire member check, the connection design and the SLS global check. These results are determined by adding all the timber element sizes together after the step. The total amount of timber in the building including the internal structure will also be a result to provide a fair comparison between the different design options. The timber usage of the internal structure is determined by calculating the amount of timber of the internal beams, columns and floor elements for every different geometry and floor weight. In appendix G an example calculation for the material used in the internal structure is shown. Table 5.4 shows the amount of timber per option in m^3.

Plot size - floor span	Floor load 1	Floor load 2	Floor load 3
27.2x27.2 - 3.4	2151	2902	3416
27.2x27.2 - 6.8	2077	2481	2983
27.2x40.8 - 3.4	3291	4431	5208
27.2x40.8 - 6.8	3210	3831	4595

Table 5.4: Amount of timber inside the building in m^3

Steel

The amount of steel used in the building will also be part of the results. To calculate the amount of steel in the facade steel needs to be added to the steel calculated by the connection design component. In figure 5.13 and 5.14 a diagrid and external braced frame connection are shown, respectively. The steel is only calculated for the blue parts in the figure by the connection design component. To account for the red parts of the steel plate and bolts some simplified calculations are done.

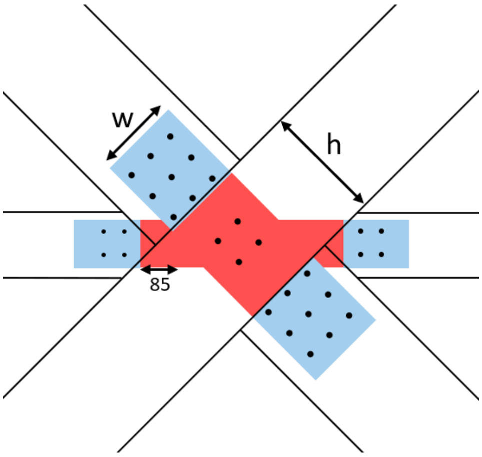


Figure 5.13: Diagrid connection with steel in crossing part

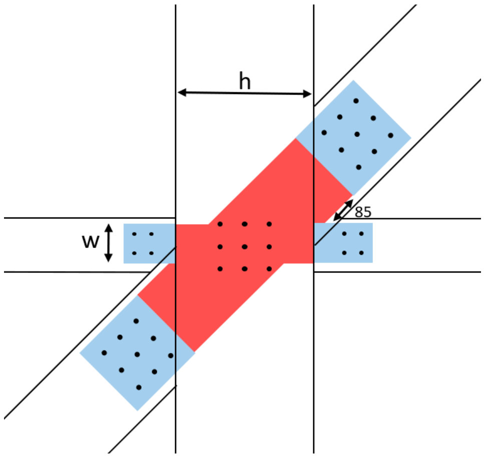


Figure 5.14: External braced frame connection with steel in crossing part

For the diagrid designs these calculations are:

Where one diagonal crosses a continuing long diagonal like in figure 5.13, the average height (h) of the long diagonals in that facade is multiplied by the average plate width (w) of the short diagonals in that facade and then multiplied by the number of plates and the plate thickness. Where a column or a beam meets the diagonals 85 mm is added to the plate in order for the plate to reach through the reduced cross-section zone of the diagonal. The bolts in the red section must keep the elements in place and take up shear forces from the beams. These shear forces are a maximum of 300 kN which can be taken up by four bolts. From detail drawings of Koning Willem 1 college and Mjøstårnet there are always more bolts present in the red areas so four bolts is assumed to be the minimum amount of bolts allowed. For each of these meeting points the area of four bolts is multiplied by the length of the bolt. The length of the bolt is the timber element width minus the 85 mm from the reduced cross-section.

For the external braced frame designs these calculations are:

Where a beam crosses a continuing column as shown in figure 5.14, the average height (h) of the columns in that facade is multiplied by the average width (w) of the plate of the beams and then multiplied by the number of plates and the plate thickness. For every brace connection 85 mm is added to the plate in order for the plate to reach through the reduced cross-section zone of the beam. Four bolts are also added to every meeting point in order to keep the elements in place and take up shear forces similar to the diagrid.

To calculate the total amount of steel in the building the steel of the internal structure is added to the steel in the stability system. The steel in the internal structure is calculated for every different geometry and floor weight. The steel in the internal structure is from the connections between the internal columns and beams. Table 5.4 shows the amount of steel in the internal structure for each design option. In appendix G an example of how the steel is calculated for the internal structure is given. This might be a slight underestimation as reference projects use a higher number of bolts for these types of connections.

Plot size - floor span	Floor load 1	Floor load 2	Floor load 3
27.2x27.2 - 3.4	1.21	1.36	2.23
27.2x27.2 - 6.8	0.98	1.09	1.54
27.2x40.8 - 3.4	1.91	2.13	3.51
27.2x40.8 - 6.8	1.63	1.81	2.57

Table 5.5: Amount of steel inside the building in m^3

5.2.9. Step 9: Brute-force

Brute-force implies that all possible combinations of the parameters are collected. To do this the software colibri is used within the grasshopper script. Colibri will collect the results for all the 216 design iterations in an CSV excel file.

Two sets of results will be gathered with the brute-force method. The first set will be after the ULS element checks, meaning that the timber elements have not been increased yet due to the global SLS checks of step 7. From these results it can be gathered what designs already satisfy the SLS requirements based on the ULS design.

The second set of results gathered by colibri will be after the designs also satisfy the global SLS requirements. These results will give the final designs that meet all requirements for all possible combinations. The CSV excel file with the results can then be ordered correctly and read either in excel or in python.

5.3. Conclusion

This chapter described the workflow and explained the workings of the parametric model. In the first paragraph all assumptions made for the model were described. Then all the steps within the parametric model were explained. After the final step of the model the results are generated and processed so that they can be discussed and shown in the following chapters.

6 General results

In the following chapters the results from the parametric model will be discussed. In appendix H all the results collected by the model are shown. In this chapter the general results of the model will be discussed. This is done to understand how the different stability system designs are sized within the parametric model. In chapter 7 the results per parameter will be discussed. In that chapter the influence of the different parameters on the material efficiency of the stability system will be shown.

The current chapter will first show the results for the ULS design. These results are the unity checks of the displacement and acceleration SLS requirements. It will show what designs satisfy the SLS requirements after the elements are sized for the ULS checks. Then, results are shown of the design for SLS. Here the options that did not meet the SLS requirements have their timber element heights increased. In the results the increase in timber will be compared to the new unity checks for displacement and acceleration. Afterwards, the increase in timber for each performed check will discussed to get an understanding of how the element sizes are increased. Lastly, the final designs of the stability systems are discussed where timber element sizes and connection sizes will be shown.

6.1. ULS design

The first set of results is gathered after sizing all the elements with the ULS element checks. From these results it can be seen that some stability system designs already meet the SLS requirements and others do not. In table 6.1 the unity checks are given for all the design options. The value in the table is the average unity check of all six element widths. Green cells represent that all six element widths satisfy the requirements. In red the options are marked that do not satisfy the requirements. One cell is marked orange for the diagrid with plot size 27.2 x 27.2, floor span 3.4 and floor load 2. Here, the acceleration requirement was only met for element widths of 400 and 450 mm. In the figure floor load 1 is the permanent floor load of 3.5 kN/m^2, load 2 is 5.3 kN/m^2 and load 3 is 6.7 kN/m^2.

Stability system	Plot size	Floor load	Floor span 3.4 m		Floor span 6.8 m	
			Displacement	Acceleration	Displacement	Acceleration
Single brace	27.2 x 27.2	1	1.07	1.52	0.82	1.43
		2	1.05	1.22	0.81	1.17
		3	1.04	1.13	0.80	1.02
	27.2 x 40.8	1	1.09	1.04	1.06	1.04
		2	1.07	0.84	1.04	0.84
		3	1.06	0.73	1.03	0.73
Double brace	27.2 x 27.2	1	0.76	1.41	0.57	1.31
		2	0.75	1.14	0.55	1.07
		3	0.73	1.06	0.54	0.94
	27.2 x 40.8	1	0.73	0.96	0.68	0.94
		2	0.71	0.77	0.66	0.76
		3	0.70	0.67	0.66	0.67
Diagrid	27.2 x 27.2	1	0.51	1.25	0.47	1.22
		2	0.47	1.00*	0.43	0.99
		3	0.44	0.93	0.41	0.86
	27.2 x 40.8	1	0.55	0.88	0.51	0.86
		2	0.50	0.70	0.47	0.69
		3	0.47	0.61	0.44	0.60

Table 6.1: Average unity checks for displacement and acceleration
*Only element width 400 & 450 mm satisfy the requirement

What can be gathered from the table is that all designs for the single braced frame with the plot size of 27.2 x 27.2 m have a displacement and acceleration that are too large. The designs for the single braced frame with plot size 27.2 x 40.8 m all do not meet the displacement requirement but the designs with permanent floor load 2 and 3 do meet the acceleration requirement. All double braced frame and diagrid designs satisfy the displacement requirement. The designs with plot size 27.2 x 40.8 m and a double braced frame or diagrid also satisfy the acceleration requirement. The double braced frame with plot size 27.2 x 27.2 m has a too large acceleration with permanent floor load 1 and 2, with load 3 only the designs with a floor span of 6.8 meters satisfy. For the diagrid designs with plot size 27.2 x 27.2 m and permanent floor load 1 the acceleration requirement is not met. With load 2 all designs with a floor span of 6.8 meters, and the designs with floor span 3.4 meters and element width 400 & 450 mm satisfy the acceleration requirement. For permanent floor load 3 all designs meet the requirements.

The unity checks for the single brace designs are the largest and the unity checks for the diagrid designs are the smallest. This means that the single braced frame has the lowest global stiffness then the double brace and the diagrid has the highest global stiffness. The unity checks for displacement are always larger for the plot size of 27.2 x 40.8 m than that of plot size 27.2 x 27.2 m. For the acceleration it is opposite where the unity checks are always larger for the 27.2 x 27.2 m plot size. When the permanent floor load increases the unity check for the displacement decreases slightly, but the unity check for the acceleration decreases significantly. The unity checks for the options with a floor span of 3.4 meters are always larger, or the same in the case of the single braced options with plot size 27.2 x 40.8 m.

6.2. Design for SLS

In table 6.2 the element heights have been increased up until the point that the global SLS requirements are met. Again, the results will be shown as the average for all six element widths. In table 6.2 the average unity check for the displacement is given in the column (UC d), the average unity check for the acceleration is given in the column with (UC a). In the column (ΔUC a) the decrease of the unity check from the unity check for the ULS design given in table 6.1 is shown. In the column (Timber) the increase of the total amount of timber in the stability system in the facade is shown in percentages. To show the material efficiency of the increase in timber for the different options the (ΔUC a) is divided by the increase in timber and shown in percentages. A higher percentage represents more material efficiency, since it means the unity check is lowered more with less added timber. The results for the double brace and diagrid with plot size 27.2 x 40.8 m are not shown as all options already met the SLS requirements. From table 6.2 information can be gathered on the unity check of the displacement (UC d), the increase in timber (Timber) and the material efficiency of the increase ((Δ a/ Timber). The findings will now be presented per item.

Stability system	Plot size	Floor load	Floor span 3.4 m					Floor span 6.8 m				
			UC d	UC a	ΔUC a	Timber	Δ a/ timber	UC d	UC a	ΔUC a	Timber	Δ a/ timber
Single brace	27.2 x 27.2	1	0.66	1.0	0.52	+760%	0.068	0.46	1.0	0.43	+650%	0.066
		2	0.70	1.0	0.22	+414%	0.053	0.52	1.0	0.17	+210%	0.081
		3	0.73	1.0	0.13	+235%	0.055	0.73	1.0	0.02	+15%	0.133
	27.2 x 40.8	1	0.89	1.0	0.04	+26%	0.15	0.90	1.0	0.04	+27%	0.148
		2	0.99	0.83	0.01	+7.0%	0.14	1.0	0.83	0.01	+5.6%	0.18
		3	1.0	0.72	0.01	+5.4%	0.19	0.99	0.72	0.01	+5.0%	0.2
Double brace	27.2 x 27.2	1	0.64	1.0	0.41	+528%	0.078	0.34	1.0	0.31	+385%	0.081
		2	0.65	1.0	0.14	+264%	0.053	0.45	1.0	0.07	+53%	0.13
		3	0.66	1.0	0.06	+137%	0.044	0.54	0.94	-	-	-
Diagrid	27.2 x 27.2	1	0.37	1.0	0.25	+319%	0.078	0.34	1.0	0.22	+250%	0.088
		2	0.45	1.0	0.01	+10%	0.1	0.43	0.99	-	-	-
		3	0.44	0.93	-	-	-	0.41	0.86	-	-	-

Table 6.2: Decrease acceleration unity check and increase timber in the stability system

Unity check displacement (UC d)

- For the larger plot size the unity checks for the displacement are larger than for the smaller plot size
- With a higher floor load the unity check for the displacement is larger
- The single brace has the highest unity checks and the diagrid the smallest
- The options with a small plot size and brace have a larger unity check with floor span 3.4 m than with floor span 6.8 m

Increase in timber (Timber)

- For the larger plot size the increase in timber is smaller than for the smaller plot size
- With a higher floor load the increase in material is smaller
- The single brace has the highest increase in material and the diagrid the smallest
- The options with a small plot size and brace have a larger increase with floor span 3.4 than with floor span 6.8

Material efficiency of the increase (Δ a/ Timber)

- For the larger plot size the efficiency seems the largest, however the increase for floor load 2 and 3 is small so the results for these options is not very accurate
- With a higher floor load the efficiency increases, except for the options with a small plot size, brace and small floor span
- The options with a small plot size, brace and floor span of 3.4 decrease in efficiency when the floor load increases
- The single brace seems to have the least efficient increase and the diagrid the most
- On average the larger floor span has a higher efficiency than the small floor span

6.3. Timber increase per check

When all the elements have been sized based on the three ULS checks including the ULS member check (blue), the fire member check (orange), the connection design (green) and the SLS global checks (red) the final designs of the stability systems are determined. To show how much each check increases the timber usage in the stability system three graphs are shown for the single brace with a plot size of 27.2 x 27.2 m and floor span 6.8 m. Three graphs are shown in figure 6.1 for the three permanent floor loads. The graphs show the amount of timber in the stability system after each check per element width. The three graphs have different y-axes.

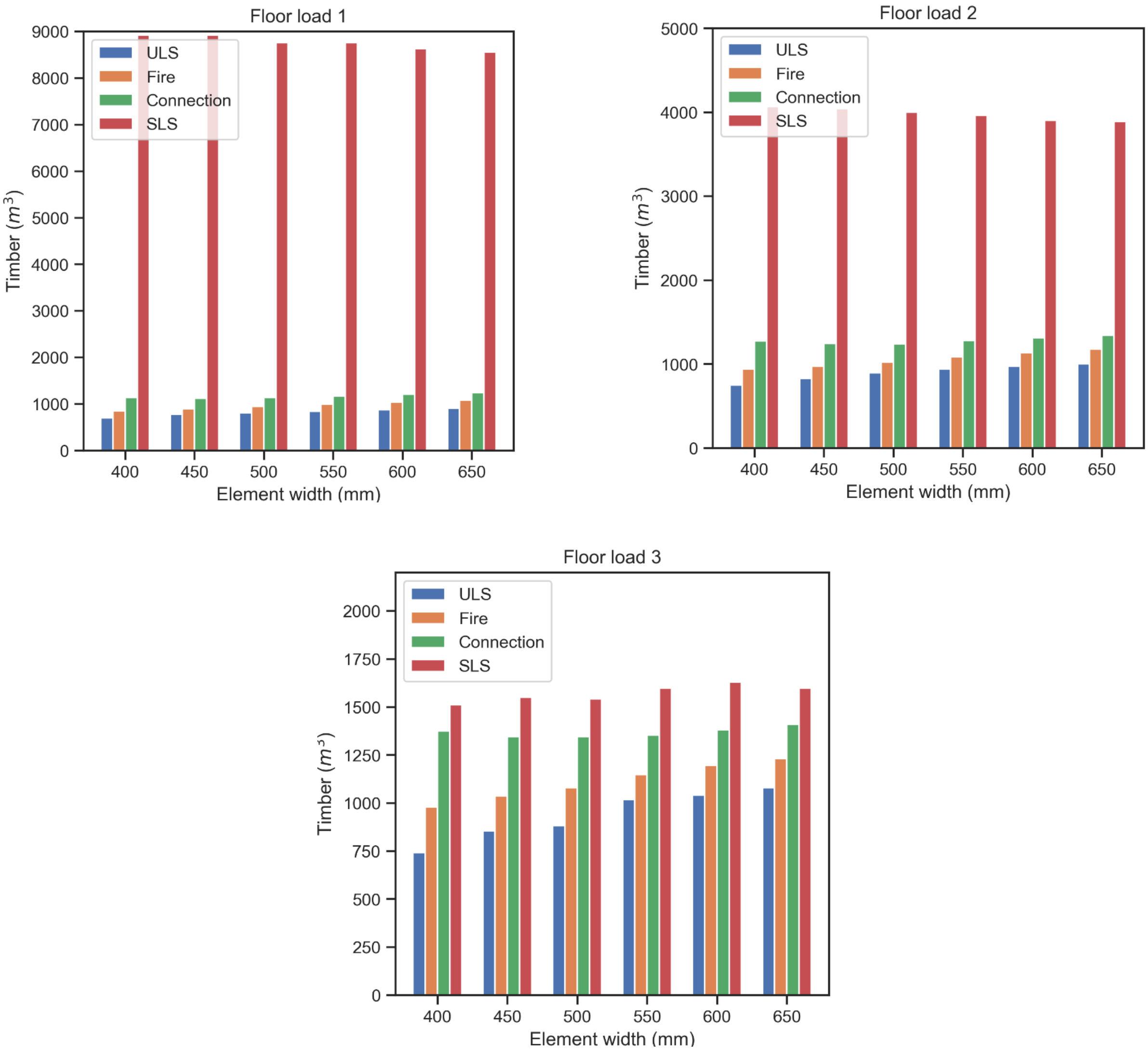


Figure 6.1: Timber in the stability system after checks, results for single brace with plot size 27.2 x 27.2 m and floor span 6.8

The three graphs show clearly that the increase in timber from the connection design to the SLS design is higher with a lower permanent floor load. This was also seen in table 6.2. The relationship between the increase in timber for the ULS checks which include the ULS member check (blue), the fire check (orange) and connection design (green) does not change with increased permanent floor load. It does however change with the element widths. The ULS and fire designs increase in material usage when the element widths increase. Then when the connection design is made the amount of timber used per element width becomes more similar. For the final timber usage determined by the SLS design it differs what width uses the least timber. The effect of the element width on the material usage in the final design will be discussed further in paragraph 8.2.4.

The graphs shown in figure 6.1 only show the results for one stability system, one floor span and one plot size. When studying the same graphs for designs with other parameters it is seen that the relationship between the increase in timber for the ULS checks is similar. This means that the choice for the stability system, floor span and plot size has no to little influence on the material increase between the ULS checks.However, there is a big difference in material increase from the ULS checks to the SLS checks for the other parameters. This material increase from the ULS to the SLS design was shown in table 6.2 in the column (Timber). For this reason no additional graphs will be shown for the other designs.

6.4. Final design

After the elements are sized based on the SLS checks all designs are finalized. In this paragraph two designs are chosen to show how a final design looks and to gain insight into the element sizes. The options that will be shown have a single brace, a plot size of 27.2 x 27.2 m, a floor span of 6.8 meters and an element width of 500 mm. One option will have permanent floor load 1 and the other load 3.

The figures will show the timber element sizes in centimeters and portray the elements scaled as their real size. First, a figure will be shown of the total head facade, then only the bottom part of the head facade will be shown with timber element sizes and a blue circle. Lastly, the connection design located in the blue circle will be shown. When the connection design is portrayed the number of bolts will be indicated with r_{row} x n_{row}.

Permanent floor load 1

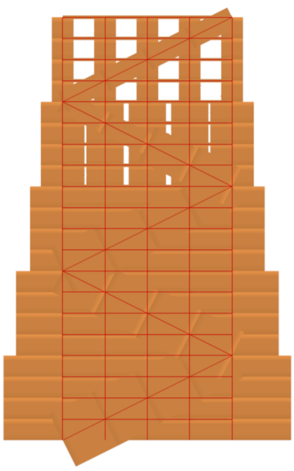


Figure 6.2: Head facade

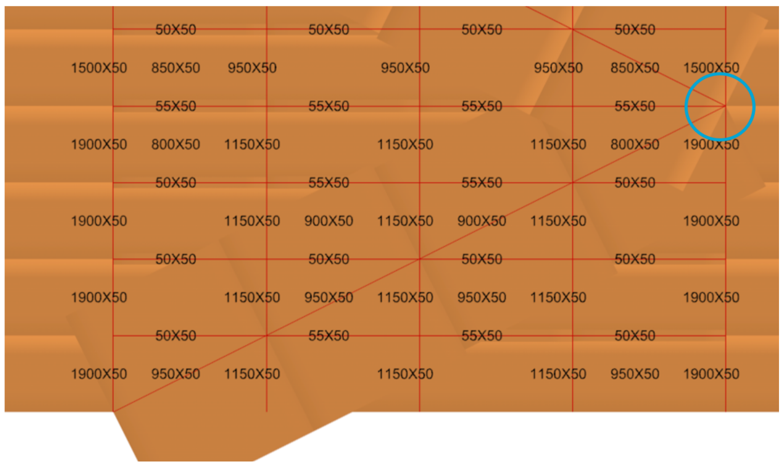


Figure 6.3: Bottom part of the head facade with element sizes (cm)

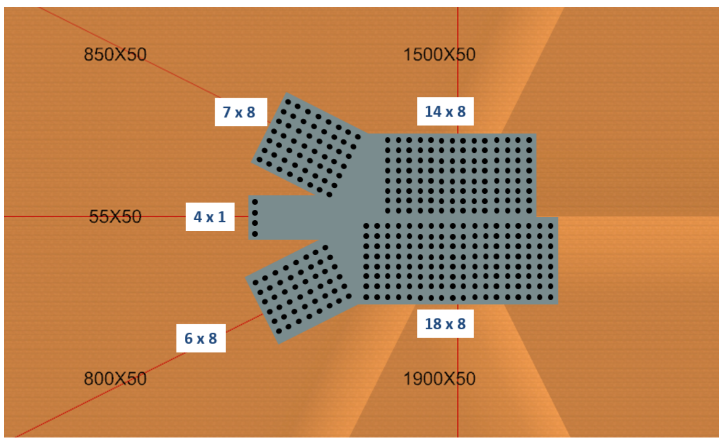


Figure 6.4: Connection design with timber element sizes (cm) and number of bolts

From figures 6.2 to 6.4 it is visible that the timber element sizes are so large that they overlap. This means that the design is not feasible. The largest columns have a size of 19000 mm, the largest beam of 550 mm and the largest brace of 9500 mm. The connection design was not altered during the SLS design so the connection size does not increase. The connection has four bolts for the beam, 56 & 48 bolts for the braces and 112 & 144 bolts for the columns.

<u>Permanent floor load 3</u>

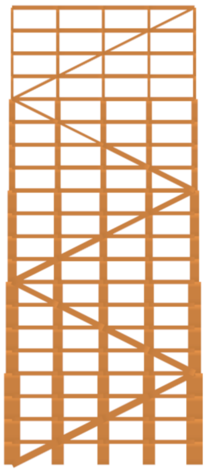


Figure 6.5: Head facade

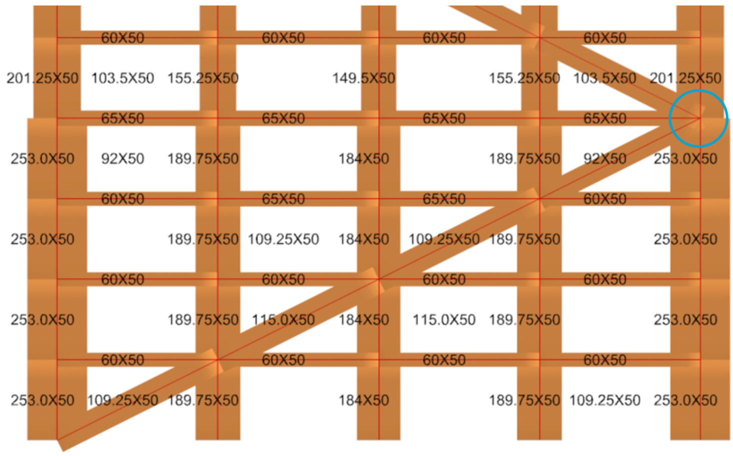


Figure 6.6: Bottom part of the head facade with element sizes (cm)

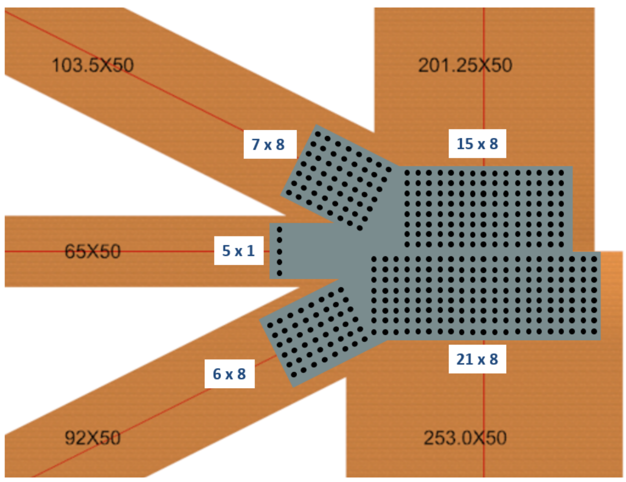


Figure 6.7: Connection design with timber element sizes (cm) and number of bolts

From figures 6.5 to 6.7 it can be seen that the timber elements do not overlap. This means that the design is feasible. The largest columns have a size of 2530 mm, the largest beam of 650 mm and the largest brace of 1150 mm. The connection design has five bolts for the beam, 56 & 48 bolts for the braces and 120 & 168 bolts for the columns.

When looking at the designs for permanent floor load 1 and permanent floor load 3 it can be seen that there is a significant decrease in timber sizes when the load increases. The large sizes for load 1 make the design unfeasible. In the discussion in paragraph 8.1.2 the largest element sizes for all designs with element width 500 mm will be shown. Here, all the unfeasible designs made by the parametric model will be discussed. For the connection design it can be seen that the connection sizes for the braces are the same and the beam and column connections increase when the permanent floor load increases.

7

Results per parameter

In this chapter the results of the influence of the different parameters will be discussed. Because the type of stability system has a large influence on the results, the results will be split up for the three stability systems. For each system the results will be discussed per parameter. The results will show the amount of timber and steel used per parameter. The amount of material used in the stability system and in the total building will be reported.

When one parameter is discussed the average value of the other parameters is taken. With the exception of the plot size. The results will always be split-up per plot size as the results are considerably different. For example, if the floor span parameter is discussed for the single braced frame design there are a total of 72 designs. These designs are split up per plot size giving 36 designs. There are two floor spans giving 18 designs per span. These 18 designs are divided into six element widths and three permanent floor loads. The results shown in the table for the floor span will be the average value of the 18 designs for all element widths and permanent floor loads.

7.1. Single brace

The results shown in the tables are the amount of timber and steel in the stability system in the facade and in the total building in cubic meters. This means that the steel connecting the internal beams and columns, and the timber of the internal floors, beams and columns are added to the material in the stability system in the facade. The values in the tables are the average values of the other parameters as explained in the introduction of the chapter.

7.1.1. Plot size

The timber usage for plot size 27.2 x 27.2 m is much larger than that of plot size 27.2 x 40.8 m as can be seen in table 7.1. The amount of steel used is smaller for the plot size of 27.2 x 27.2 m. For the total amount of timber there is an increase of 42 % and for the total amount of steel 39 %. In the facade the timber increase is 224% and the steel increase is 31%.

Plot size (m)	27.2 x 27.2 m	27.2 x 40.8 m
Timber (m^3)	8251	5817
Timber facade (m^3)	5583	1723
Steel (m^3)	5.64	7.83
Steel facade (m^3)	4.24	5.57

Table 7.1: Material used

7.1.2. Floor span

Plot size 27.2 x 27.2 m

From table 7.2 it can be seen that the total amount of timber used is larger with a floor span of 3.4 m with an increase of 27%. The timber in the facade is also larger for the span of 3.4 m with an increase of 34%. The total amount of steel used is smaller with a floor span of 6.8 m. The increase in steel in the facade is 9% and in the total amount of steel is 15%.

Span (m)	3.4	6.8
Timber (m^3)	9221	7281
Timber facade (m^3)	6399	4768
Steel (m^3)	6.03	5.26
Steel facade (m^3)	4.43	4.05

Table 7.2: Material used for plot size 27.2 x 27.2 m

Plot size 27.2 x 40.8 m

Table 7.3 shows that the total amount of timber is fairly more for a span of 3.4 m with an increase of 6.1%. In the facade the timber used is larger for the span of 6.8 with an increase of 5.3%. The total amount of steel used is smaller with a floor span of 6.8 m. The increase in steel in the facade is 6.1% and in the total amount of steel is 11%.

Span (m)	3.4	6.8
Timber (m^3)	5988	5646
Timber facade (m^3)	1678	1767
Steel (m^3)	8.25	7.41
Steel facade (m^3)	5.74	5.41

Table 7.3: Material used for plot size 27.2 x 40.8 m

7.1.3. Permanent floor load

Load 1 is the permanent floor load of 3.5 kN/m^2, load 2 is 5.3 kN/m^2 and load 3 is 6.7 kN/m^2.

Plot size 27.2 x 27.2 m

In table 7.4 it is shown that the amount of timber used decreases when the permanent floor load increases, for both the total amount of timber and timber in the facade. The increase between the options with the least timber and the most is 216% for the facade and 84% for the total amount of timber. The steel usage increases when the load increases. The increase in the facade is 29% and in the total building it is 39%.

Load	1	2	3
Timber (m^3)	11115	7592	6046
Timber facade (m^3)	9001	4903	2847
Steel (m^3)	4.78	5.51	6.64
Steel facade(m^3)	3.69	4.28	4.76

Table 7.4: Material used for plot size 27.2 x 27.2 m

Plot size 27.2 x 40.8 m

As seen in table 7.5 the total amount of timber used increases when the permanent floor load increases. The timber in the facade is the least for load 2. The increase between the options with the least timber and the most is 8.9% for the facade and 31.5% for the total amount of timber. The steel usage increases when the load increases. The increase in the facade is 31% and in the total building it is 42%.

Load	1	2	3
Timber (m^3)	5043	5777	6632
Timber facade (m^3)	1792	1646	1730
Steel (m^3)	6.56	7.61	9.33
Steel facade (m^3)	4.79	5.64	6.29

Table 7.5: Material used for plot size 27.2 x 40.8 m

7.1.4. Element width

The element width has no influence on material usage of the internal structure, therefore only the material used in the stability system in the facade will be shown.

Plot size 27.2 x 27.2 m

The amount of timber used from low to high per element width is 650, 600, 550 & 400, 500 and 450, this can be seen in table 7.6. The difference between the option with the least and the most timber is 5.8%. The steel usage is the largest for an element width of 400 mm. From a width of 400 to 550 mm there is a significant decrease in steel usage, where the decrease in these steps becomes smaller. From a width of 550 mm to 650 mm there is a very small decrease in steel. The maximum difference in steel usage is 46%.

Width (mm)	400	450	500	550	600	650
Timber facade (m^3)	5598	5737	5687	5595	5458	5425
Steel facade (m^3)	5.52	4.53	4.04	3.81	3.79	3.77

Table 7.6: Material used in facade for plot size 27.2 x 27.2 m

Plot size 27.2 x 40.8 m

The results for the large plot size are shown in table 7.7. The amount of timber used increases when the element width increases. The difference between the option with the least and the most timber is 9.2%. The steel usage is the largest for an element width of 400 mm. From a width of 400 to 550 mm there is a significant decrease in steel usage, where the decrease in these steps becomes smaller. From a width of 550 mm to 600 mm there is a very small decrease and 650 mm has the same usage as 600 mm. The maximum difference in steel usage is 44%.

Width (mm)	400	450	500	550	600	650
Timber facade (m^3)	1660	1667	1693	1731	1772	1814
Steel facade (m^3)	7.20	5.86	5.31	5.05	5.01	5.01

Table 7.7: Material used in facade for plot size 27.2 x 40.8 m

7.2. Double brace

The results shown in the tables are the amount of timber and steel in the total building in cubic meters. This means that the steel connecting the internal beams and columns, and the timber of the internal floors, beams and columns are added to the material in the stability system in the facade. The values in the tables are the average values of the other parameters as explained in the introduction of the chapter.

7.2.1. Plot size

The timber usage for plot size 27.2 x 27.2 m is larger than that of plot size 27.2 x 40.8 m as can be seen in table 7.8. The amount of steel used is smaller for the plot size of 27.2 x 27.2 m. For the total amount of timber there is an increase of 18% and for the total amount of steel 39%. In the facade the timber increase is 150% and the steel increase is 28%.

Plot size (mm)	27.2 x 27.2 m	27.2 x 40.8 m
Timber (m^3)	6752	5744
Timber facade m^3)	4079	1629
Steel (m^3)	5.89	8.00
Steel facade (m^3)	4.49	5.75

Table 7.8: Material used

7.2.2. Floor span

Plot size 27.2 x 27.2 m

From table 7.9 it can be seen that the total amount of timber used is larger with a floor span of 3.4 with an increase of 36%. The timber in the facade is also larger for the span of 3.4 m with an increase of 56%. The total amount of steel used is smaller with a floor span of 6.8 m. The increase in steel in the facade is 11% and in the total amount of steel is 15%.

Span (m)	3.4	6.8
Timber (m^3)	7789	5714
Timber facade (m^3)	4966	3191
Steel (m^3)	6.30	5.48
Steel facade (m^3)	4.70	4.28

Table 7.9: Material used for plot size 27.2 x 27.2 m

Plot size 27.2 x 40.8 m

Table 7.10 shows that the total amount of timber is fairly more for a span of 3.4 m with an increase of 5.5%. In the facade the timber used is larger for the span of 6.8 m with an increase of 8.5%. The total amount of steel used is smaller with a floor span of 6.8 m. The increase in steel in the facade is 6.4 % and in the total amount of steel is 11%.

Span (m)	3.4	6.8
Timber (m^3)	5897	5590
Timber facade (m^3)	1562	1695
Steel (m^3)	8.44	7.58
Steel facade (m^3)	5.93	5.57

Table 7.10: Material used for plot size 27.2 x 40.8 m

7.2.3. Permanent floor load

Load 1 is the permanent floor load of 3.5 kN/m^2, load 2 is 5.3 kN/m^2 and load 3 is 6.7 kN/m^2.

Plot size 27.2 x 27.2 m

In table 7.11 it is shown that the amount of timber used decreases when the permanent floor load increases, for both the total amount of timber and timber in the facade. The increase between the options with the least timber and the most is 192% for the facade and 60% for the total amount of timber. The steel usage increases when the load increases. The increase in the facade is 27% and in the total building it is 37%.

Load	1	2	3
Timber (m^3)	8766	5999	5490
Timber facade (m^3)	6652	3308	2276
Steel (m^3)	5.03	5.76	6.89
Steel facade (m^3)	3.94	4.54	5.00

Table 7.11: Material used for plot size 27.2 x 27.2 m

Plot size 27.2 x 40.8 m

As seen in table 7.12 the amount of timber used increases when the permanent floor load increases. The increase between the options with the least timber and the most is 14% for the facade and 39% for the total amount of timber. The steel usage increases when the load increases. The increase in the facade is 27% and in the total building it is 39%.

Load	1	2	3
Timber (m^3)	4787	5792	6653
Timber facade (m^3)	1517	1640	1730
Steel (m^3)	6.80	7.78	9.45
Steel facade (m^3)	5.04	5.80	6.41

Table 7.12: Material used for plot size 27.2 x 40.8 m

7.2.4. Element width

The element width has no influence on material usage of the internal structure, therefore only the material used in the stability system in the facade will be shown.

Plot size 27.2 x 27.2 m

The amount of timber used from low to high per element width is 400, 500, 550 & 600, 450 and 650, this can be seen in table 7.13. The difference between the option with the least and the most timber is 10%. The steel usage is the largest for an element width of 400 mm and then decreases when the element width increases. The maximum difference in steel usage is 42%.

Width (mm)	400	450	500	550	600	650
Timber facade (m^3)	3843	4165	4009	4106	4112	4237
Steel facade (m^3)	5.68	4.65	4.35	4.15	4.10	4.02

Table 7.13: Material used in facade for plot size 27.2 x 27.2 m

Plot size 27.2 x 40.8 m

The results for the large plot size are shown in table 7.14. The amount of timber used decreases from 400 mm to 450 mm and then it increases. The difference between the option with the least and the most timber is 13%. The steel usage is the largest for an element width of 400 mm. From a width of 400 to 550 mm there is a significant decrease in steel usage, where the decrease in these steps becomes smaller. From a width of 550 mm to 600 mm there is a very small decrease and from 600 mm to 650 mm there is an increase again. The maximum difference in steel usage is 42%.

Width (mm)	400	450	500	550	600	650
Timber facade (m^3)	1596	1552	1572	1626	1678	1748
Steel facade (m^3)	7.37	6.02	5.46	5.24	5.19	5.23

Table 7.14: Material used in facade for plot size 27.2 x 40.8 m

7.3. Diagrid

The results shown in the tables are the amount of timber and steel in the total building in cubic meters. This means that the steel connecting the internal beams and columns, and the timber of the internal floors, beams and columns are added to the material in the stability system in the facade. The values in the tables are the average values of the other parameters as explained in the introduction of the chapter.

7.3.1. Plot size

The timber usage in the facade for plot size 27.2 x 27.2 m is larger than that of plot size 27.2 x 40.8 m as can be seen in table 7.15. However, when the internal structure is included the large plot size uses more timber. The amount of steel used is smaller for the plot size of 27.2 x 27.2 m. For the total amount of timber there is an increase of 4% and for the total amount of steel 29%. In the facade the timber increase is 49% and the steel increase is 26%.

Plot size (mm)	27.2 x 27.2 m	27.2 x 40.8 m
Timber (m^3)	6213	6480
Timber facade (m^3)	3544	2386
Steel (m^3)	19.93	25.65
Steel facade (m^3)	18.53	23.39

Table 7.15: Material used

7.3.2. Floor span

Plot size 27.2 x 27.2 m

From table 7.16 it can be seen that the total amount of timber used is larger with a floor span of 3.4 m with an increase of 7.3%. The timber in the facade is also larger for the span of 3.4 with a difference increase of 3.7%. The total amount of steel used is smaller with a floor span of 3.4 m. The increase in steel in the facade is 20% and in the total amount of steel is 16%.

Span (m)	3.4	6.8
Timber (m^3)	6432	5993
Timber facade (m^3)	3609	3480
Steel (m^3)	18.47	21.39
Steel facade (m^3)	16.87	20.18

Table 7.16: Material used for plot size 27.2 x 27.2 m

Plot size 27.2 x 40.8 m

Table 7.17 shows that the total amount of timber used is more for a span of 3.4 m with an increase of 3.9%. In the facade the timber used is larger for the span of 6.8 m with an increase of 7.8%. The total amount of steel used is smaller with a floor span of 3.4 m. The increase in steel in the facade is 15.2% and in the total amount of steel it is 12%.

Span (m)	3.4	6.8
Timber (m^3)	6606	6355
Timber facade (m^3)	2296	2476
Steel (m^3)	24.24	27.05
Steel facade (m^3)	21.73	25.05

Table 7.17: Material used for plot size 27.2 x 40.8 m

7.3.3. Permanent floor load

Load 1 is the permanent floor load of 3.5 kN/m^2, load 2 is 5.3 kN/m^2 and load 3 is 6.7 kN/m^2.

Plot size 27.2 x 27.2 m

In table 7.18 it is shown that the amount of timber used is the smallest for permanent floor load 2. The increase between the options with the least timber and the most is 230% for the facade and 86% for the total amount of timber. The steel usage increases when the load increases. The increase in the facade is 36% and in the total building it is 38%.

Load	1	2	3
Timber (m^3)	8699	4685	5254
Timber facade (m^3)	6585	1994	2054
Steel (m^3)	16.72	19.96	23.10
Steel facade (m^3)	15.62	18.74	21.22

Table 7.18: Material used for plot size 27.2 x 27.2 m

Plot size 27.2 x 40.8 m

As seen in table 7.19 the amount of timber used increases when the permanent floor load increases. The increase between the options with the least timber and the most is 20% for the facade and 38% for the total amount of timber. The steel usage increases when the load increases. The increase in the facade is 37% and in the total building it is 40%.

Load	1	2	3
Timber (m^3)	5408	6535	7498
Timber facade (m^3)	2158	2404	2596
Steel (m^3)	21.37	25.65	29.92
Steel facade (m^3)	19.61	23.67	26.88

Table 7.19: Material used for plot size 27.2 x 40.8 m

7.3.4. Element width

The element width has no influence on material usage of the internal structure, therefore only the material used in the stability system in the facade will be shown.

Plot size 27.2 x 27.2 m

The amount of timber used increases when the element width increases, this can be seen in table 7.20. The difference between the option with the least and the most timber is 14%. The steel usage is the largest for an element width of 400 mm. From a width of 400 to 600 mm there is a significant decrease in steel usage, where the decrease in these steps becomes smaller. From a width of 600 mm to 650 mm there is a small increase. The maximum difference in steel usage is 43%.

Width (mm)	400	450	500	550	600	650
Timber facade (m^3)	3238	3402	3561	3674	3685	3706
Steel facade (m^3)	23.94	19.37	17.20	16.78	16.72	17.16

Table 7.20: Material used in facade for plot size 27.2 x 27.2 m

Plot size 27.2 x 40.8 m

The results for the large plot size are shown in table 7.21. The amount of timber used first decreases from 400 mm to 500 mm and then it increases. The difference between the option with the least and the most timber is 6.3%. Again, the steel usage is the largest for an element width of 400 mm. From a width of 400 to 600 mm there is a significant decrease in steel usage, where the decrease in these steps becomes smaller. Then for a width of 650 mm the steel increases. The maximum difference in steel usage is 43%.

Width (mm)	400	450	500	550	600	650
Timber facade (m^3)	2472	2361	2326	2339	2380	2439
Steel facade (m^3)	30.28	24.45	21.74	21.16	21.11	21.58

Table 7.21: Material used in facade for plot size 27.2 x 40.8 m

8 Discussion

In this chapter the results will be explained and discussed. At first, the general results will be discussed and secondly the results per parameter.

8.1. General results

Firstly, the change in the material usage for the stability system design from the ULS to the SLS design will be discussed. Secondly, the final designs of the models will be discussed where the biggest element sizes will be shown and the feasibility of the designs will be discussed.

8.1.1. From ULS to SLS design

The first results after the ULS design in table 6.1 showed that some designs did not have sufficient global stiffness or weight to comply with the SLS requirements. All designs with plot size 27.2 x 27.2 m did not satisfy the acceleration requirements. Whereas, almost all designs with plot size 27.2 x 40.8 m satisfied the acceleration requirement except the single brace with permanent floor load 1.

Then, the design for SLS results were shown. In table 8.1 the trends described in paragraph 6.2 are summarized. It was seen that if the plot size increases the unity check for the displacement increases, the percentage of timber added decreases and the efficiency of the added timber increases. An increase in permanent floor load gives the same behaviour. There is an exception for this behaviour indicated in the table with an asterisk. This exception will be explained in paragraph 8.2.2. With an increase in span the displacement unity check decreases, the percentage of timber added decreases and the efficiency of the added timber increases. The global stiffness of the single brace is the lowest, the double brace has a higher stiffness and the diagrid has the highest stiffness. It was seen that the stability systems with a higher global stiffness had a smaller displacement unity check, less timber added and a higher efficiency. From this table it can be seen that an increase in the displacement unity check sometimes causes a decrease in added timber and other times a decrease in displacement unity check causes a decrease in added timber. To explore why this is, first it is important to understand the model response for acceleration. Then afterwards this phenomena is explained by exploring the relationship between the global displacement and the building mass.

	UC d	Timber (+%)	Δ a/ timber
Plot size ↑	↑	↓	↑
Floor load ↑	↑	↓	↑*
Span ↑	↓	↓	↑
Global stiffness stability system ↑	↓	↓	↑

Table 8.1: Trends found for the design for SLS
*Not true for braced frame designs with floor span 3.4 m and plot size 27.2 x 27.2 m

Model response for acceleration

The options that did not meet the SLS requirements needed to have their element heights increased. This was done to increase the global stiffness of the stability systems but it also increased the weight of the building which can decrease the acceleration. The response of the parametric model to a decrease in displacement and increase in weight is shown in figures 8.1 and 8.2. To calculate the natural frequency of the model the weight and the global lateral displacement of the building are used as can be seen in formula 4.2. To calculate the acceleration the weight and the calculated natural frequency are used as is shown in formula 8.1. In this formula μ_{ref} represents the mass of the building and $R(n)$ and $k_p(n)$ are functions of the natural frequency.

$$a_{\max}(n) = c_{\mathrm{f}} \cdot \rho \cdot I_{\mathrm{v}} \cdot v_{\mathrm{m}}^2 \cdot \mathbf{R}(\mathbf{n}) \cdot \frac{K_{\mathrm{y}} \cdot K_{\mathrm{z}} \cdot \Phi}{\boldsymbol{\mu}_{\mathrm{ref}} \cdot \Phi_{\max}} \cdot \mathbf{k_p}(\mathbf{n}) \tag{8.1}$$

A smaller frequency will increase the acceleration if the weight remains the same. A larger weight will decrease the acceleration if the natural frequency is the same. However, as can be seen in figure 8.1 the increase in weight in the parametric model will decrease the frequency but still decrease the acceleration. Figure 8.2 shows that a smaller displacement will increase the frequency and therefore decrease the acceleration. A smaller natural frequency will also increase the allowed acceleration.

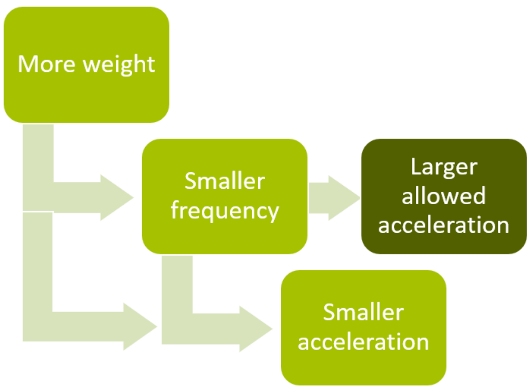


Figure 8.1: Effect of weight on the along-wind acceleration

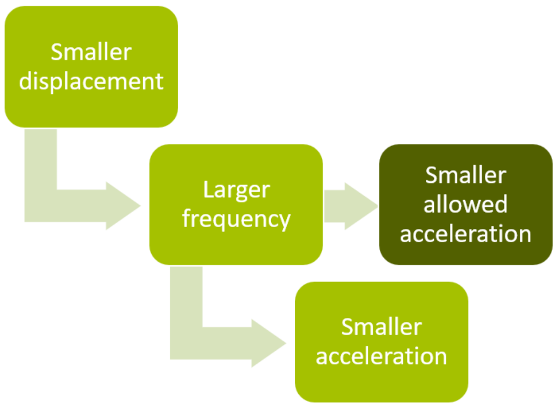


Figure 8.2: Effect displacement on along-wind acceleration

Relationship between global displacement and building mass

As just discussed, when the acceleration needs to be decreased you can increase the weight and decrease the displacement. In table 8.1 it was seen that global displacement unity check was sometimes higher when the amount of added material was lower. And, sometimes the global displacement unity check was lower when the added timber was lower. In figures 8.3 and 8.4 the relationship between the weight of the building and the displacement is shown. When the displacement is smaller, less weight is required to meet the acceleration requirement. Figure 8.3 shows a situation where the unity check for the displacement is 1, so the global displacement is 136 mm. In figure 8.4 the unity check is 0.5 and the global displacement is 68 mm. In these figures the average weight of the building per running meter in height after the ULS design from each plot size and permanent floor load is shown with vertical lines. With a blue line the acceleration of the building is plotted. The acceleration decreases with increased mass of the building. In orange the acceleration requirement is shown. The acceleration requirement increases with increased weight. A building satisfies when the requirement is larger then the acceleration. With a displacement of 136 mm in figure 8.3 it can be seen that the building should have a weight of approximately 225000 kg/m to meet the acceleration requirement this is indicated with a black dotted line. When the displacement is 68 mm the required weight becomes less and is approximately 190000 kg/m this is also indicated with a black dotted line.

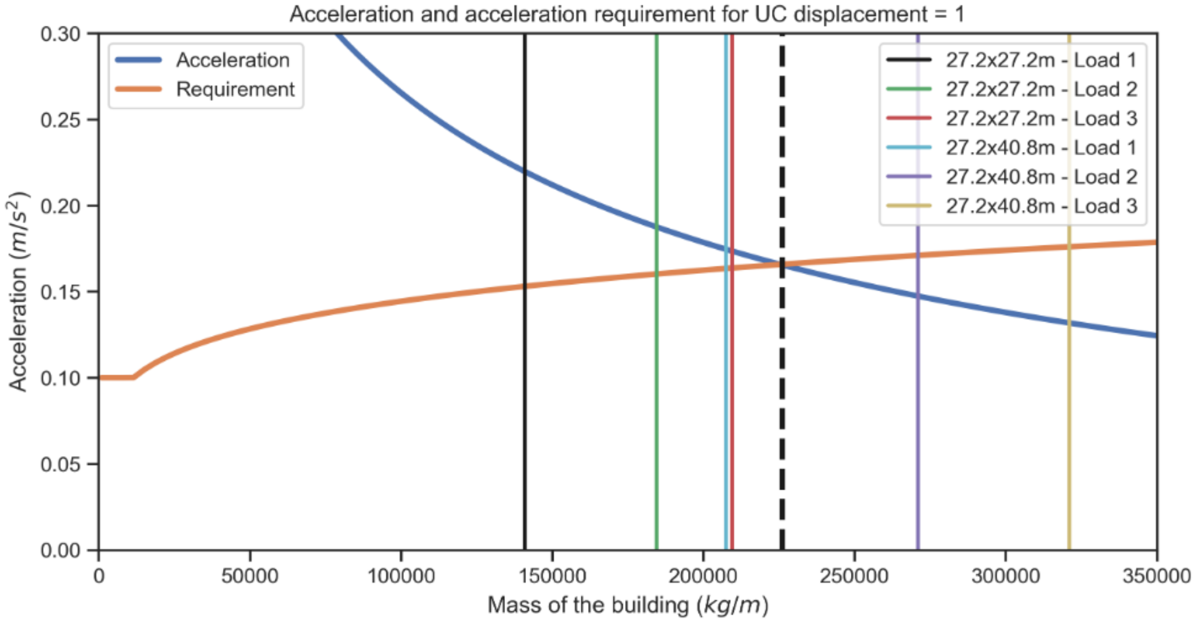


Figure 8.3: Acceleration and requirement for displacement UC = 1

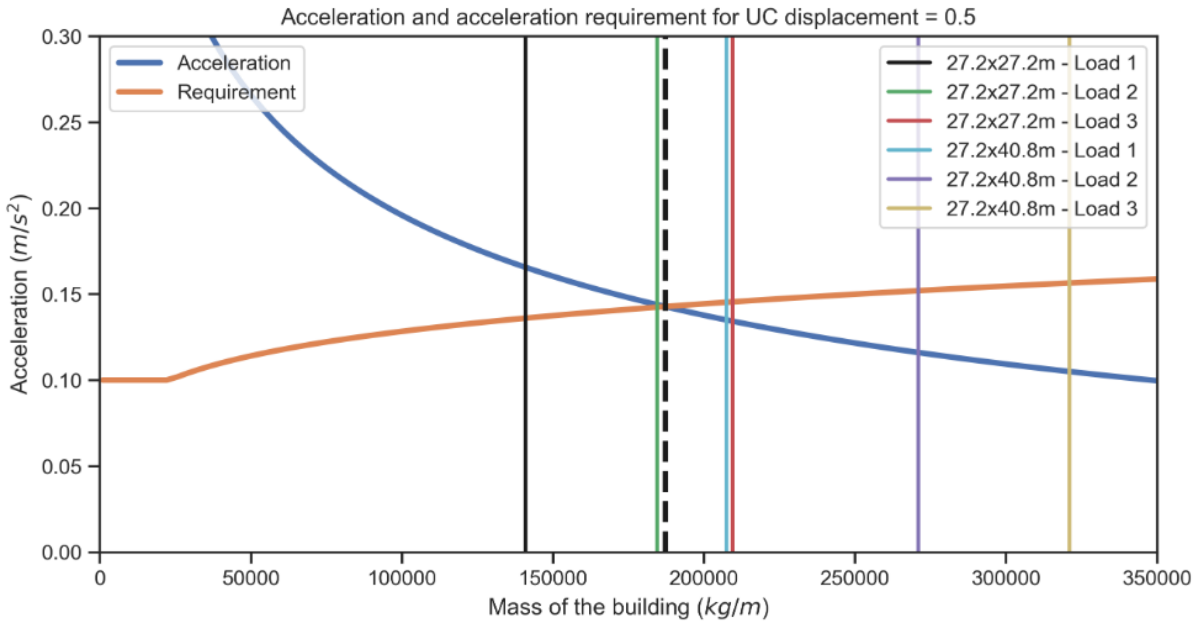


Figure 8.4: Acceleration and requirement for displacement UC = 0.5

Looking at table 8.1 once more it was seen that when the plot size is larger and the permanent floor load is larger the displacement unity check is larger, and the required added timber is smaller. When the plot size and permanent floor load are larger the total mass of the building is also larger. Therefore, the acceleration requirement is met with a larger global displacement. Meaning that the stability system requires less global stiffness. That is why almost all designs with a large plot size met the SLS requirements after sizing for ULS, as was seen in table 6.1.

The other relationship that could be seen in table 8.1 is that a larger floor span and higher global stiffness require less added timber. A larger floor span also increases the global stiffness on average, this will be explained in paragraph 8.2.2. A higher global stiffness will result in a smaller global displacement. consequently, when comparing figure 8.3 to 8.4 it can be sees that with a smaller global displacement less building mass is required. This is why the diagrid needed the least added timber.

8.1.2. Final design

Not all final designs were feasible as was seen in paragraph 6.4. For all stability system designs with an element width of 500 mm the largest element heights in millimeter per element type are given in the matrix in figure 8.5. The values for the permanent floor loads were load 1: 3.5 kN/m^2, load 2: 5.3 kN/m^2 and load 3: 6.7 kN/m^2.In the figure the braces are also called diagonals. In paragraph 6.4 the design for a single brace with a small plot size, floor span of 6.8 m with permanent floor load 1 and load 3 was shown. The design for load 1 was unfeasible as the columns had the large element height of 19000 mm and the braces of 9500 mm. These element heights caused the elements to overlap. In figure 8.5 this option is therefore indicated in red. The other designs that are unfeasible are all the braced frame designs with a plot size of 27.2 x 27.2 m and a floor span 3.4 m. The single brace with a small plot size, a floor span of 6.8 m and permanent floor load 2. The double brace with the small plot size, a floor span of 6.8 m and permanent floor load 1. And, the diagrid designs with a small plot size and load 1. These results are shown for an element width of 500 mm. The element heights will decrease with a larger element width, however this decrease still gives designs with element heights that overlap or fill almost the complete facade. For this reason it is concluded that the designs marked red in figure 8.5 are not feasible for every element width.

Stability system	Plot size	Floor load	Floor span 3.4 m			Floor span 6.8 m		
			Column	Beam	Diagonal	Column	Beam	Diagonal
Single brace	27.2 x 27.2	1	18900	450	11475	19000	550	9500
		2	8680	500	5320	7483	600	3468
		3	5610	550	3230	2530	650	1150
	27.2 x 40.8	1	3510	450	2535	3938	550	2100
		2	2438	500	1625	2818	600	1495
		3	2400	550	1560	2805	650	1430
Double brace	27.2 x 27.2	1	124800	450	6760	9075	550	3570
		2	6745	500	4260	2960	600	1120
		3	3680	550	2080	1950	650	700
	27.2 x 40.8	1	1450	450	850	1850	550	850
		2	1550	500	850	2050	600	850
		3	1600	550	850	2200	650	850
Diagrid	27.2 x 27.2	1	250	2960	5625	250	3330	5735
		2	250	990	1815	250	1000	1850
		3	250	1000	1650	250	1150	2000
	27.2 x 40.8	1	250	800	1450	250	900	1650
		2	250	900	1700	250	950	1900
		3	250	950	1900	250	1050	2150

Figure 8.5: Largest element heights (mm) for design with an element width of 500 mm

Enhancing the final design

In this thesis the global stiffness of the designs was only enhanced by increasing certain timber element heights. In the braced frame designs only the columns and braces were increased and in the diagrids only the diagonals and beams. In the exploratory study performed on the design for SLS in paragraph 4.5 it was explored how the global stiffness could be increased. This study was performed on designs with a floor span of 6.8 m and plot size 27.2 x 27.2 m. In this study it was seen that increasing the column height was less material efficient than increasing the brace height in the braced frame systems. To enhance the braced frame designs and make them feasible a solution could be to increase the element height of the brace more than the element height of the column. In this parametric model the element heights of all elements are always increased with the same percentage so the design is not fully optimized. For the diagrid designs increasing the beam height was also less efficient than increasing the diagonal height. Therefore increasing the diagonal height more than the beam height could give better results.

From the exploratory study it was also seen that increasing the connection stiffness could enhance the global stiffness. Increasing the connection stiffness would decrease the required timber element heights. For the single braced frame design it was seen that increasing the beam connections would increase the global stiffness. For the double brace increasing the brace and beam connections had a positive influence. Increasing the brace connections is 3.5x more effective than increasing the beam connections. In the diagrid designs increasing the connections in the diagonals could improve the stiffness.

From figure 8.5 it could also be seen that the braced frame designs with a span of 3.4 m give more unfeasible designs than those with a span of 6.8 m. Why this happens will be explained in paragraph 8.2.2.

8.2. Results per parameter

Now the parameters will be discussed to explore what design choices increase the material efficiency. There were five parameters in the model. These parameters were the stability system, the plot size, the floor span, the permanent floor load and the element size. This discussion will first start with a comparison of the stability systems split-up per plot size. Afterwards, the parameters of the floor span, permanent floor load and element width will be discussed. The parameters of the stability system and plot size have such a large influence on the other parameters that they will be discussed within the paragraphs of the other parameters. The element width will be discussed lastly as it has the least influence on the other parameters.

8.2.1. Comparison of the systems

In figure 8.6 a matrix is given where the total amount of steel and timber in the complete building used per design is shown for the designs with plot size 27.2 x 27.2 m. The total amounts in the building including internal structure are used to make a fairer comparison between the systems. The results in the tables are the average values of all element widths. This is done because the element widths do not influence the other parameters and therefore the average value gives clear results. In grey the unfeasible options are marked and in green the options with the least amount of material are shown. When an option is unfeasible it will not be chosen as an option that uses the least amount of material. From figure 8.6 it can be seen that the least amount of steel is always used for the smallest load. In the external braced frame systems the span of 6.8 m uses the least steel and in the diagrid a span of 3.4 m. For the external braced frame the floor span of 6.8 m and permanent floor load 3 uses the least timber. For the diagrid the least amount of timber is used with a permanent floor load of 2. The option with the least timber is the double brace, the diagrid is second and the single brace is third. The least amount of timber used per system does not differ much, the difference between the single brace and double brace is only an increase of 2.2 %. For the steel the difference is much larger. From the double brace to the single brace the increase is 15% and from the double brace to the diagrid the increase is 241%.

Floor span 3.4

Single brace

Floor load	1	2	3
Timber (m^3)	11398	8893	7538
Steel (m^3)	5.1	5.8	7.1

Double brace

Floor load	1	2	3
Timber (m^3)	9407	7438	6524
Steel (m^3)	5.4	6.1	7.4

Diagrid

Floor load	1	2	3
Timber (m^3)	9049	4890	5358
Steel (m^3)	15.6	18.4	21.5

Floor span 6.8

Single brace

Floor load	1	2	3
Timber (m^3)	10832	6458	4554
Steel (m^3)	4.4	5.2	6.2

Double brace

Floor load	1	2	3
Timber (m^3)	8126	4560	4456
Steel (m^3)	4.7	5.4	6.4

Diagrid

Floor load	1	2	3
Timber (m^3)	8349	4482	5150
Steel (m^3)	17.9	21.5	24.8

Figure 8.6: Timber and steel for plot size 27.2 x 27.2 m

In figure 8.7 the results are shown for the plot size of 27.2 x 40.8 m. Here as well, the amount of steel is smallest for the lowest load. In the external braced frame systems the span of 6.8 m uses the least steel and in the diagrid a span of 3.4 m, this is the same as for the small plot size. For all options the lowest permanent floor load gives the least timber. The option with the least amount of timber is the double brace again. Now, the single brace is second and the diagrid is last. The difference in timber between the double brace and the diagrid is an increase of 12.2%. For the steel the difference from the single brace to the double brace is an increase of 3.2% and from the single brace to the diagrid the increase is 225%.

Floor span 3.4

Single brace

Floor load	1	2	3
Timber (m^3)	5037	6040	6888
Steel (m^3)	6.9	7.9	9.9

Double brace

Floor load	1	2	3
Timber (m^3)	4776	6029	6887
Steel (m^3)	7.2	8.1	10.0

Diagrid

Floor load	1	2	3
Timber (m^3)	5378	6745	7695
Steel (m^3)	20.2	24.1	28.4

Floor span 6.8

Single brace

Floor load	1	2	3
Timber (m^3)	5049	5514	6376
Steel (m^3)	6.2	7.3	8.7

Double brace

Floor load	1	2	3
Timber (m^3)	4798	5556	6418
Steel (m^3)	6.4	7.4	8.8

Diagrid

Floor load	1	2	3
Timber (m^3)	5439	6325	7300
Steel (m^3)	22.5	27.2	31.5

Figure 8.7: Timber and steel for plot size 27.2 x 40.8 m

8.2.2. Floor span

Steel

The amount of steel used in not optimised for the SLS design it is only calculated based on the ULS requirements. For the floor span there is a difference in steel usage between the braced frame designs and the diagrid. For the braced frame the options that use the least amount of steel have a floor span of 6.8 m, this can be explained by there being less connections. Because with a floor span of 3.4 m there are twice as many columns in the building. For the diagrid it is opposite where the floor span of 3.4 m uses less steel. This is caused by the difference in transferring the internal beam loads in the facade shown in figure 4.17. The amount of steel mostly decreases in the diagonals, this is because the floor span of 6.8 meters causes smaller normal forces in the diagonals on average. These relationships are true for both plot sizes.

Timber

To discuss the timber usage the discussion is split up in external braced frame and diagrid stability systems.

External braced frame

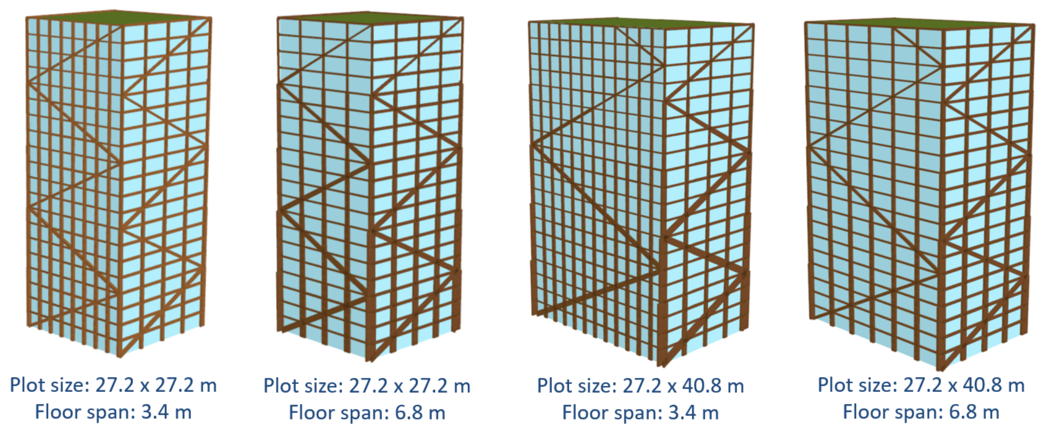


Figure 8.8: Floor span and plot size designs for single brace

In figure 8.8 the four different designs for a single brace are shown for the two plot sizes and floor spans. As can be seen from the figure the options with plot size 27.2 x 40.8 m will always have their short facade, which is normative for the global displacement, with a floor span of 6.8 m. Therefore a change in floor span will not effect the displacement much. The amount of timber used in the facade is larger for the floor span of 6.8 m but the total amount of timber used in the building is largest for the floor span of 3.4 m.

In plot size 27.2 x 27.2 m with a span of 6.8 m you will have the same head and side facade but with a span of 3.4 m the side facade will become normative for the displacement. The facade has more connections and therefore the displacement of the facade is also larger and the global stiffness is lower. This was clear from the results in table 6.1 and was also indicated in table 8.1. This is also why more timber in the facade was required with a floor span of 3.4 for the plot size of 27.2 x 27.2 m. The efficiency of increasing the timber is also lower for a span of 3.4 as was seen in table 6.2, this is because the timber in both facades is increased while the facade with a span of 3.4 m is normative. The material efficiency could therefore be enhanced for these designs by increasing the stiffness of the facade with the short span first. Also, the designs for a span of 3.4 m were not investigated in the exploratory study. For the designs with a span of 6.8 m it was already seen that increasing the beam connections would have a positive effect on the global stiffness. With a smaller span the connection stiffness in the beams is even less because of the smaller load transferred to the connection. Consequently, the designs with plot size 27.2 x 27.2 m and floor span 3.4 could probably benefit even more from increasing the beam connection stiffness than the options with floor span 6.8.

Diagrid

For the diagrid the change in floor span does not change the facade design, the only difference in the load transfer is to the beams of the stability system. For the plot size of 27.2 x 40.8 m the amount of timber used in the facade is larger for a floor span of 6.8 m, this is caused by the span giving larger normal forces in the stability system. However, the total timber usage is larger for a span of 3.4 m due to the higher timber usage of the internal structure.

For plot size 27.2 x 27.2 m the amount of timber used in the facade is larger for the span of 3.4 m. These designs use less steel and have less bolts resulting in lower connection stiffnesses. With lower connection stiffnesses more timber is required to provide the same global stiffness.

8.2.3. Permanent floor load

The three different permanent floor loads are load 1: 3.5 kN/m^2, load 2: 5.3 kN/m^2 and load 3: 6.7 kN/m^2.

Steel

When looking at the steel usage for the different plot sizes the large plot size of 27.2 x 40.8 m uses the most steel as there are more connections present in the system. With an increased permanent floor load the amount of steel also always increases since the joints will get larger vertical forces and therefore also need larger connections.

Timber

For the plot size of 27.2 x 40.8 m permanent floor load 1 always has the least amount of timber. This is because of the weight of the building. From figure 8.3 it could be seen that with a unity check of 1 the building weight was close to meeting the acceleration requirement and load 2 and 3 were very safe. For the braced frame designs with plot size 27.2 x 27.2 m, load 3 used the least timber. However, for the double brace load 2 and 3 did not differ much. The diagrid designs with this plot size used the least timber with load 2. This is also because of the relationship between the global displacement and the mass of the building.

The diagrid designs with a small plot size had a displacement unity check of roughly 0.5 after the ULS design, so they can be compared to figure 8.4. From this figure it can be seen that with this displacement the permanent floor load of 2, shown in the green vertical line is almost at the intersection of the acceleration and the acceleration requirement. The single brace designs had a displacement unity check of roughly 1 after the ULS design, so they can be compared to figure 8.3. The weights closest to the intersection are the small plot size with permanent floor load 3 shown in red, and the large plot size with load 1. These loads are also the same as the most material efficient designs. This means that these kinds of graphs can help predict what kind of building mass is the most material efficient when the range of the displacement is known. The graphs can also be plotted with acceleration on the y-axis and displacement on the x-axis and then the type of stability system can be chosen based on the required displacement unity check.

8.2.4. Element width

Steel

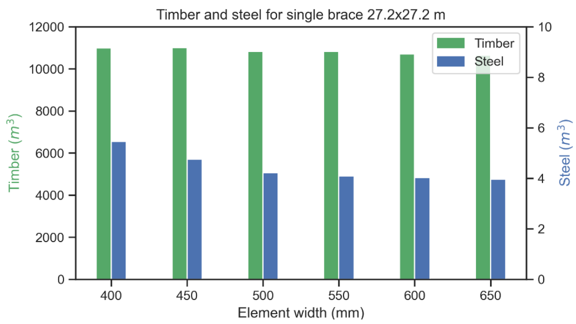


Figure 8.9: Steel usage per element width for single brace

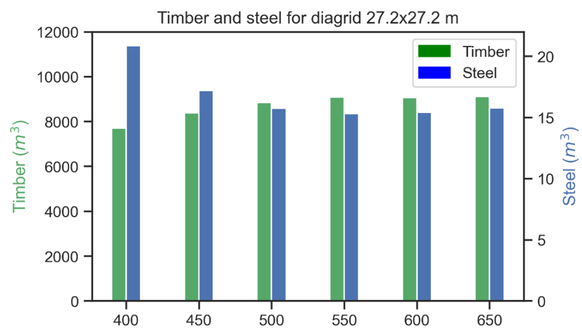


Figure 8.10: Steel usage per element width for diagrid

The most interesting result for the steel usage comes from the element width. In figures 8.9 and 8.10 the amount of steel and timber is shown per element width. In figure 8.9 the result is shown for a single brace with plot size 27.2 x 27.2 m with permanent floor load 1 and in figure 8.10 the result is shown for the diagrid with plot size 27.2 x 27.2 m and floor load 1. The difference between the steel usage of some of the designs was that either the amount of steel used decreased slightly from a width of 600 to 650 like in figure 8.9 or it would increase slightly like in figure 8.10. However, the difference between the steel usage was always relatively small. Therefore, it can be concluded that the elements widths of 600 and 650 on average use the same amount steel. For all designs the element width of 400 mm used the most steel and it then decreased up until 600 mm. The options with an element width of 400 mm, 450 mm and 500 mm also use more bolts than the other widths giving the connections a higher stiffness. This is caused by the capacity of the connection. In a thinner timber element the capacity per bolt can be less because the bolt length is shorter. Shorter bolts do not decrease the connection stiffness however. Also, the block shear capacity is reached earlier with thinner elements making adding rows of bolts (n_{row}) ineffective. To get a higher connection capacity the timber element height will have to be increased.

Timber

When looking at the results for the timber increase per check in paragraph 6.3 it was shown that the increase in timber between the fire design and connection design was the largest for smaller element widths. This is caused by the block shear capacity becoming normative and the elements requiring a higher element height. Nonetheless, the timber usage after the connection design is very similar for all element widths. For designs that are sized by the SLS requirements the element width with the least timber usage differs quite a lot. The only real noticeable trend is that for the small plot size with a double brace or diagrid the amount of timber required is lower for smaller element widths when permanent floor load 1 or 2 are applied. This is because of these widths having more bolts and a higher connection stiffness. In figure 8.10 this can be seen clearly where the amount of timber used increases up until an element width of 550 mm. This is because the increased connection stiffness of the brace and diagonals decreases the displacement and therefore less timber is needed to decrease the displacement. For the single brace this effect is less because it was seen in the exploratory study for the SLS design in paragraph 4.5.1. that the increase of connection stiffness of the brace had no influence on the displacement. That is why this increase in steel is less efficient than for the other designs as can be seen in the timber usage of figure 8.9.

For the diagrid designs that are sized based on the ULS design the least amount of timber is used for the element widths in the middle of the range, so least for 500 mm and 550 mm and increasing a little when moving outwards. For the braced frame designs that are sized based on the ULS design the element widths with the least amount of timber are from 400-500 mm and increases slightly when the width increases.

8.3. Conclusions

From ULS to SLS design

The results and discussion on the sizing of the elements for ULS to SLS design showed that heavier buildings were more often sized on the ULS checks. That is why the buildings with the larger plot size met the acceleration requirement already when the displacement unity check was close to 1. For the smaller plot size the designs with a higher load also needed less material increase. The same went for buildings with a higher global stiffness requiring less added material. It was also seen that some of the designs were unfeasible due to their large element sizes. These options were all of the small plot size. No option with load 1 nor braced options with span 3.4 were feasible. To improve the designs the exploratory study for the SLS design could be used. The external braced frames can be improved by increasing the column heights less than the brace height and the diagrid designs can be improved by increasing the diagonal heights more than the beam height. For braced frame designs where the facade with a floor span of 3.4 m can become normative for the global displacement the material efficiency can also be increased. This can be done by first only improving the facade with floor span 3.4 m until it has the same stiffness as the facade with floor span 6.8 m. Also, the connection stiffness can be increased to decrease the element heights. This was out of the scope of this research but was studied in the exploratory study for the SLS design. It was found that increasing the connection stiffness of the beam would increase the global stiffness in the single braced frame designs. In the double braced frame designs increasing the brace had the most effect and increasing the beam connection would also improve the global stiffness. For the diagrid designs increasing the diagonal connections would enhance the global stiffness.

Parameters

When the difference between parameters is smaller than 10% the influence is seen as insignificant.

- **Plot size:** The amount of steel used increases when the plot size increases. This increase in the facade is around 30% for the braced frames and 26% for the diagrid. The total amount of steel is increased with 39% for the braced frames and 29% for the diagrid. From this it can be concluded that a large plot size uses more steel, where the increase is larger for the internal structure than the facade.

  The amount of timber used in the braced frames is more for the small plot size, In the facade the increase is 150% and 224% and in the total building 42% and 18%. So in the external braced frame designs the small plot sizes uses more timber especially in the facade. For the diagrid the amount of timber used in the facade is 49% more for the small plot size and the total amount of timber used is 4% more. This means the big plot size is still more timber efficient as the floor area of the 27.2 x 40.8 m plot size is 1.5x larger than the 27.2 x 27.2 m plot size.

- **Floor span:** For the braced frame designs with a small plot size the amount of steel used is 10% more in the facade and 15% in total with a floor span of 3.4 m than with a span of 6.8 m. For the large plot size the amount of steel used in the braced frames is also more with a span of 3.4 m, in the facade the increase is roughly 6% and the total building it is 11%. The external braced frame designs are therefore more efficient with a span of 6.8 m. Although, the influence on the facade in the large plot size is small. For all the diagrid designs the amount of steel used is higher for a span of 6.8 m with roughly 18% for the facade and 14% in total. This means that the load transfer through the facade with a span of 3.4 m is more steel efficient than a span of 6.8 m.

  The timber usage in the braced frame systems with a small plot size is larger for floor span 3.4 m. The timber in the facade is 34% and 56% higher and the total timber in the building 27% and 36% higher. Thus, the span of 6.8 m is more timber efficient for a braced frame with plot size 27.2 x 27.2 m. Braced frames with a large plot size use less timber in the facade with a span of 3.4 m and use less timber in total with a span of 6.8 m. The increase is only around 7% in the facade and roughly 5% in total. Therefore, the floor span is not seen as a significant influence on the large plot size for the braced frame designs. For the diagrid designs with a small plot size the timber usage is smaller for a span of 6.8 m with 3.7% in the facade and 7.3% in total. For the large plot size the diagrid designs use less timber in the facade with a span of 3.4 m and less timber in the total building with 6.8 m. The difference is 3.9% for the facade and 7.8% in total. All increases for timber in the diagrid designs are below 10% so the influence of the span on the timber usage is seen as insignificant.

- **Permanent floor load:** The three different permanent floor loads are load 1: 3.5 kN/m^2, load 2: 5.3 kN/m^2 and load 3: 6.7 kN/m^2. The steel usage in the braced frame designs is higher when the permanent floor load increases. The increase in steel usage from permanent floor load 1 to load 3 is around 30% in the facade, and 40% in the total building for all braced frame designs. The steel usage in the diagrid is also more when the load increases. From load 1 to load 3 the increase is around 38% for the facade and the total building. The steel usage will therefore always increase when the permanent floor load increases for all designs.

  In the braced frame designs with a small plot size the highest permanent floor load uses the least timber. The increase from load 1 to load 3 is 192% and 216% for the facade and 84% or 60% in total. The diagrid designs with a small plot size use the least timber with permanent floor load 2 and the most with load 1 the difference is 230 % in the facade and 86% in total. The second best option was permanent floor load 3 that used 12% more timber in total. For the large plot size all designs use the least timber with the least load. The increase of timber for the braced frames in the facades was 8.9% and 14% and in the total building 32% and 39%. For the diagrid designs with large plot size the increase in the facade is 38% and in the total building 20%.

To compare if it is more material efficient to increase the permanent floor load or to increase the timber in the facade, options with a load that is 'too high' are compared to options with a load that is 'too low'. The load is too high when the design with a lower load uses less material. Thus, all options with a large plot size with a load higher than load 1 are seen as too high. In the designs with the large plot size the total increase of timber in the buildings when using permanent floor load 3 instead of 1, ranges from 20% to 39%. And, the increase for the diagrid with a small plot size from load 2 to 3 was only 12% as mentioned before. The small plot sizes use the least amount of timber with permanent floor load 2 or 3, depending on the global stiffness of the system. The increase from load 1 to the option with the least amount of timber for designs with a small plot size ranges from 60% to 86%. These percentages are significantly higher than the percentages for the increase when the permanent floor load is 'too high'. Also, the options for the small plot size with load 1 gave unfeasible results. Therefore, it seems to be more timber efficient to use a higher permanent floor load then to increase the timber in the facade.

- **Element width:** The steel usage in the facade per element width decreases from width 400 mm up until width 600 mm. Then from 600 mm to 650 mm the steel usage either is the same, increases or decreases. The options with the least amount of steel are either 600 mm or 650 mm. The difference between the element width with the most steel usage and the least is for every design between 42% and 46%. Consequently, it can be said that the steel efficiency is higher with a larger element width. It was also discussed that from an element width of 400 mm to an element width of 550 mm the amount of bolts decreased in the connections increasing the connection stiffness.

  The most timber efficient element width differs for all different designs. Therefore no conclusion can be drawn on what width is the most timber efficient.

- **Stability system:** When using the same parameters for both braced frame designs the double braced frame design uses between 6.8% to 1.1% more steel in the entire building than the single brace. This is a relatively small increase. The diagrid designs use between 150% and 300% more steel than both braced frames. This an average steel usage of 3x more for the diagrid designs. The diagrid design might have an even larger steel usage as the connections for the corner columns might require a steel box as mentioned in paragraph 4.2.1. It can therefore be concluded that the steel efficiency for the braced frame designs is similar and the diagrid designs have a much lower steel efficiency.

  The stability system with the least timber usage differs as the global stiffness of the systems is highly related to the required permanent floor load. When looking at the matrices in figure 8.6 and 8.7 it could be seen that the double braced frame designs use the least timber in the total building. However, for the small plot size the best options of the three stability systems only differ 2.2%, and for the large plot size it is 12.6%. Therefore it can be concluded that for the small plot size the three systems are similarly timber efficient. For the large plot size the timber efficiency is highest for the double brace, however the difference is not large.

9

Conclusion and Recommendations

9.1. Conclusions

The research performed in this study can be used to answer the main question:

> *How can different design choices influence the material efficiency of an externally braced timber stability system for high-rise, based on an integral comparison considering both connections and timber elements?*

The following conclusions apply to the researched ranges of the parameters and buildings dimensions. The studied designs have a height of 68 meters with 20 floors. The researched stability systems are a braced frame with a single brace, a braced frame with a double brace and a diagrid. The plot sizes are the small plot size of 27.2 x 27.2 m and larger plot size of 27.2 x 40.8 m. Floor spans of 3.4 m and 6.8 m. Three permanent floor loads with self weights of 3.5 kN/m^2, 5.3 kN/m^2 and 6.7 kN/m^2. And six element widths of 400 mm, 450 mm, 500 mm, 550 mm, 600 mm and 650 mm. When a result has an influence of 10% or less it is seen as insignificant.

Stability system design

- An external braced frame system has a very different load transfer than the diagrid system. This results in a more uniform load distribution in the diagrid but causes more elements to have larger normal forces resulting in a steel usage that is 3x higher than that of the external braced frame. Therefore, the external braced frame is more material efficient when considering the steel usage.
- The diagrid stability system is more often sized based on the connection design as it has a higher global stiffness than the external braced frame designs. The external braced frame designs are more often sized based on the along-wind acceleration. For the buildings with a small plot size this resulted in the diagrid requiring a lighter permanent floor load to get the most material efficient design. The most timber efficient designs for the small plot size all used roughly the same amount of timber, there was only a difference in timber usage of 2.2%. For the large plot size the external braced frame options were slightly more timber efficient with a difference of 12.6%.
- The double brace designs uses less than 7% more steel than the single brace designs and the single brace designs use slightly more timber. The most timber efficient designs of the single brace use less than 6% more timber than the double brace designs. The material efficiency of both braced frame designs is therefore similar.

Design parameters

- **Plot size:** The plot size of 27.2 x 27.2 m requires heavier floors to meet the acceleration requirement than the plot size of 27.2 x 40.8 m. Therefore, the amount of material used per floor area of the building is larger and therefore the material efficiency is less for a smaller plot size.

- **Floor span, external braced frame:** Having a smaller floor span in a facade that can be normative for the global displacement is not material efficient. This is seen in the braced frame designs with a small plot size. The added steel connections between the elements add 10% more steel in the facade with a floor span 3.4 m, and decreases the global stiffness of the building. This results in bigger displacements which in its turn requires 34% or more timber in the facade. The steel and timber usage of the internal structure is also larger for a floor span of 3.4 m than that of 6.8 m. This results in the total material usage in the building being 15% higher for steel and more than 27% higher for timber when using floor span 3.4 m.

  Having a smaller floor span in a facade that is not normative for the global displacement will also decrease the steel efficiency in the facade with 6% and in the total building with 10%. This is due to an increase in number of connections. However, a smaller floor span can cause a higher timber efficiency of roughly 7% in the stability system. This was seen in the braced frame designs with a large plot size. Nevertheless, the timber efficiency of the total building will still go down when using the smaller floor span with 5% due to added timber in the internal structure.

- **Floor span, diagrid:** The larger floor span in the diagrid causes a larger average normal force in the members of the diagrid. This causes roughly 18% more steel usage in the facade of the diagrid with a floor span of 6.8 m compared to the diagrid with a span of 3.4 m. But, the total steel usage is roughly 14% higher with the smaller span. Using a smaller span can also decrease the timber usage in the facade which was seen in the large plot size, where the large span used 3.9% more timber in the facade. Nonetheless, the timber efficiency of the total building is roughly 7.5% higher for all diagrid designs when using a floor span of 6.8 m, due to the internal structure.

- **Permanent floor load:** The most material efficient permanent floor load depends on the displacement of the building. When the wind force, building dimensions and building weight or global displacement are known the acceleration requirement can be plotted against the acceleration. Using these types of graphs can give a good estimation as to what floor load or displacement will give material efficient designs without having to model the whole building. For the small plot size a higher permanent floor load of 5.3 kN/m^2 or 6.7 kN/m^2 was more timber efficient. While, for the large plot size the smallest load of 3.5 kN/m^2 was the most timber efficient. It was also found that increasing the permanent floor load is more timber efficient than increasing the timber in the stability system for the three researched permanent floor loads. The steel usage of the designs increase when the load increases since the steel connections are only sized on the ULS checks and no extra steel is added to improve the global stiffness of the stability system. Consequently, under these conditions the steel efficiency is highest with the lowest loads.

- **Element width:** The amount of steel used in the stability system is the largest for the smallest researched element width of 400 mm. The steel usage then decreases up until an element width of 600 mm, this decrease is more than 40%. For the widths 600 mm and 650 mm the amount of steel used is similar and gives the lowest steel usage. Also, the number of bolts used decreases from a width of 400 mm to a width of 550 mm. From 550 mm to 650 mm the number of bolts is similar. Using more bolts gives a higher connection stiffness. This can result in some of the designs requiring less timber. In this research this was the case for some of the diagrid and double braced designs with a small plot size. Because of the small plot size more global stiffness was required in the designs and therefore the added connection stiffness had a positive effect. Nonetheless, for most designs researched the element width with the least amount of timber used differs. This means no definitive conclusion can be drawn as to what the most timber efficient element width is.

9.2. Recommendations

This paragraph will describe recommendations for further research on the topic of timber high-rise buildings based on the findings from this thesis.

- **Dynamic behaviour:** The along-wind acceleration was determining for many of the studied designs. To determine the acceleration the natural frequency of the building is used. Determining the acceleration and frequency of a timber high-rise building is challenging as there are not many buildings in practice that can be used to verify if the behaviour is correct. In this thesis the analytical method was used to determine the frequency due to difficulties with the modelling software. It is recommended to perform further research to prove if the analytical method to determine the natural frequency for timber high-rise made by Oosterhout (1996) [78] is correct. When calculating the acceleration with the Eurocode the exponent mode shape ξ and the structural logarithmic decrement of damping δ_s are used. The commonly used value for the mode shape is based on research done for the building Treet. Later research on the building Mjøstårnet showed that this mode shape might not be correct for all timber high-rise designs. The value used for the structural logarithmic decrement of damping was a value from the Eurocode for timber bridges. That is why it is also recommended to further investigate the mode shape and structural logarithmic decrement of damping for timber high-rise. During this investigation not only the axial connection stiffness should be regarded but also the rotational connection stiffness. Since the rotational connection stiffness was found to have a large impact on the dynamic behaviour [73].
- **Improving dynamic behaviour:** In this study the dynamic behaviour was improved by either adding weight or increasing the global stiffness. The global stiffness was enhanced by increasing some of the timber element heights. From an exploratory study on the design for SLS it was seen that increasing some of the timber elements was more material efficient than others. It was also seen that increasing the connection stiffness of certain elements would also increase the global stiffness. It would be interesting to study the relationship between increasing the connection stiffness and timber element stiffness further to find what elements are the most material efficient to increase. Another interesting way to decrease the dynamic behaviour that could be studied would be to add dampers in the building. The dampers could decrease the floor load which would decrease the material usage in the building.
- **Stability system design:** In this study only three stability system designs were researched for only two building dimensions. One diagrid design with a top angle of 90 degrees and a ring beam every floor. And, two external braced frame designs with one or two braces and a slope of 1:2. For the external braced frame designs it is interesting to study the material efficiency of short to slender buildings. However, for the studied diagrid design exploring buildings with a slenderness higher than 2.5 is interesting as it was seen in this thesis that the diagrid is not more material efficient than the external braced frame for this slenderness. The angle of the stability systems can also influence the material efficiency of the stability system. For instance a diagrid design with a smaller top angle and ring beam every two floors might have resulted in a more material efficient system. That is why further research towards the most material efficient stability system design is recommended.
- **Connection design:** In this research it was concluded that the diagrid models use roughly 3x more steel than the external braced frame designs. That is why it is recommended to study other element types that might be more material efficient for diagrid stability systems. The slotted-in steel plate connection designs used in this research can also be improved. In the parametric model the connection design was mostly fixed to decrease the number of parameters. Nevertheless, from the exploratory study on the connection design it was seen that creating the most material efficient slotted-in steel plate connection design is difficult. For this thesis the slotted-in steel plate connection study was only a part and it is believed the connection design can be enhanced further. Also, in this study two steel plates were used for all connection designs while three plates were also considered. Using three plates would have increased the connection stiffness which could have possibly decreased the amount of timber required. That is why it is recommended to study the slotted in-steel plate connections more.

- **Internal structure design:** The internal structure of the building would at times dictate what design parameter was more material efficient. As this research focused mainly on the stability system design the internal structure design was simplified and not optimized. For this reason, studying the internal structure for timber high-rise with different floor spans and floor weights might be interesting.

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A

Reference projects

A.1. Treet, Bergen Norway

Treet is a housing building with 62 apartments on 14 floors and has a height of 45 meters. This building was finished in 2015, at the time it was the highest building with a timber structural system. The location of the building is in urban and central Bergen close to the ”Puddefiorden”. Treet is owned by housing association BOB, Bergen og Omegn Boligbyggelag [8]. BOB wanted to make a 20 story high building but this was not permitted since Bergen mostly has buildings of 6 floors high. So, BOB proposed to make the highest tree tower in the world that could show in a modern way that Bergen is a timber city. The project goals were sustainability, cost efficiency and prefabrication. Sweco was chosen as the structural engineer and Moelven as the timber supplier. Both companies were selected because they had the most experience with making large timber structures for bridges [42].

Figure A.1: Treet, Bergen [48]

A.1.1. Structural System

In figure A.2 the assembly of Treet is shown. First a concrete plinth is made and four floors of prefabricated modules are stacked upon the plinth. The modules in the building are made from timber framework. After this a glulam framework is placed on the outside and a power story is created on the fifth floor. This power storey is also made of glulam elements and has a prefabricated concrete slab on it that carries the load of the following four floors of modules. On the tenth floor there is another power storey that supports the remaining three floors and the roof. The vertical forces are transferred through the groups of four modules and then transported to the main frame by the power storeys [70].

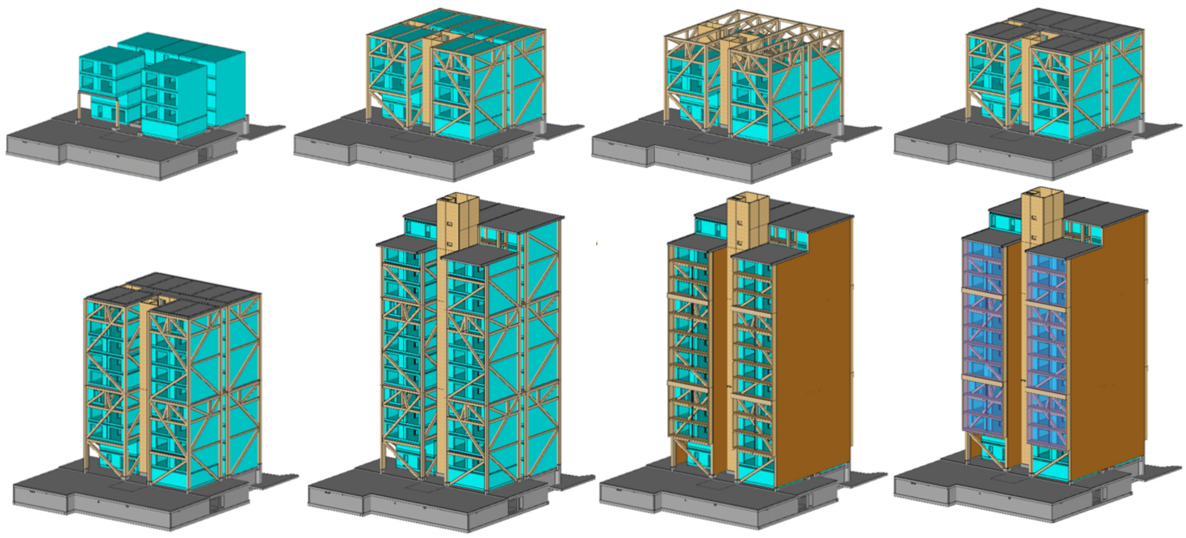


Figure A.2: Assembly structure Treet, [70]

The normal floors have a height of 3 meters and the power storeys have a height of 3.3 meters. The frame work is designed to resemble the modern designs of timber bridges. Another reason why a lot of smaller timber elements are used is because the design of this building started with the modules. These modules could not be stacked higher than four storeys so the idea of making three smaller buildings stacked on top of each other was born [3]. The elevator shaft is made from CLT walls, these walls are separate from the main structural system and do not provide extra stability. Wind loading causes tension in the foundation. These forces are transported to the bedrock by steel core piles. The building was made to be robust. Meaning that when a structural member were to fail the other members will be able to take the force [48]. This was verified using the accidental limit state.

The materials used are GL30c and GL30h for the glulam in the frame work. Almost all timber elements were protected from the outside weather by metal or glass sheeting. Climate class 1 was used for inside members and class 2 for members next to the external walls on the cold side. The steel used for plates is S355 and the dowels are made of A4-80 [48]. The stability in the building is provided by the glulam framework made of K-shaped parts. This framework is mostly in the facade but also has a frame between the modules. To connect the frames and transfer the horizontal loads, CLT floors are used in the corridors on each level. As well as the power storeys with their framework and concrete floors [77]. All the connections in the framework and power storeys are hinged joints. Most element sizes are based on the ultimate limit state with wind and dynamic behaviour, this will be elaborated in paragraph A.1.4. The remaining element sizes were governed by the fire design [8]. This could refer to the design of the connection or that of the beam. The elements affected most by the connections are beams that bear vertical loads [11]. The global deformation requirement caused by static effect of wind is:

$$H/500 = 45000/500 = 90mm$$

The deformation for Treet is 71 mm and this equals H/634 [8].

A.1.2. Floor plan and loads

Treet has a plan of 20.7 x 22.3 meters. The floor plan for Treet is dictated by the modules as can be seen in figure A.3. The vertical load transfer for this building goes from four modules stacked on one another to its respective concrete floor on the power storey. The corridors in the middle of the building are attached to the main frame on every floor. In figure A.3 in red the framework that carries the vertical loads down to the foundation is shown. In blue the span direction of the concrete floors is shown. The concrete floors rest on the power storey which transfers the loads to the framework in the facade and columns in the middle of the building. The length between grid B and D and E and G is 9.3 meters so

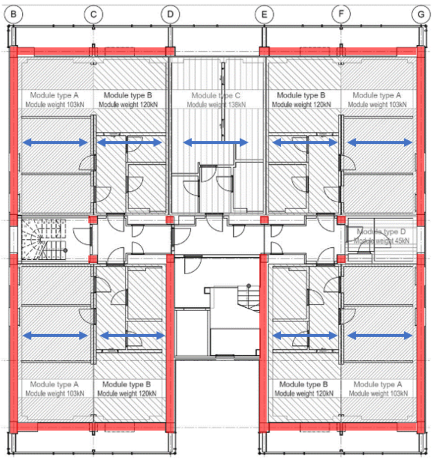


Figure A.3: Typical plan of Treet, [48]

each side gets 4.65 meters of floor loads [40].

The loads are determined using the Eurocode and the Norwegian national annexes. The maximum wind speed was determined to be $V = 44.8\ m/s$ and this gives the equivalent wind pressure of $q = 1.26\ kN/m^2$. The wind load was checked from all four sides as well as diagonal directions of 45° and 135°. No wind tunnel was used because of the traditional shape of the building. The wind load was determined to be the dominant load so according to the Norwegian method loads from earthquakes can then be excluded. The self weight for the timber parts was $4.5\ kN/m^3$ and the concrete to $25\ kN/m^3$. The live loads were set to $2.0\ kN/m^2$ for the apartments, $3.0\ kN/m^2$ for the common areas, $4.0\ kN/m^2$ for the balconies and $5.0\ kN/m^2$ for the gym [48]. The highest compression force in a column is 4287 kN, the highest tension in a column was 296 kN and the highest tension in a diagonal was 930 kN. The cross-sections for columns are either 405x650 mm or 495x495 mm. The typical cross-section for a diagonal is 405x405 mm [8].

Cross-section	Force [kN]	$\sigma_{\mathrm{N,Ed}}\ [\mathrm{N/mm^2}]$
405x650	-4287	16.3
405x650	296	1.12
405x405	930	5.7

A.1.3. Dynamic response

Since Treet was an innovative design not much was known about how this building would behave when loaded horizontally by wind. Treet has a low structural weight and is proportionally high meaning it falls within a range where natural frequency can cause discomfort. The natural frequency affects the acceleration of the building. The higher the acceleration the more discomfort users experience. According to the standard ISO 10137 the perception acceleration limit for 50% of the population is 0.049 m/s^2 and the limit for nausea is 0.098 m/s^2 [36]. Figure A.4 shows the graph of permitted acceleration where line 1 is for offices and line 2 is for residences. To study the dynamic behaviour of Treet multiple studies were done beforehand. And afterwards the real wind-induced accelerations of the building were measured to see if the predictions were correct [12].

The first research was performed on the structural concept in 2012. During this research a 3D-model was created in a finite element program. In the model the external timber frame for storeys 0 to 4, 6 to 9, 11 to 14 and the power-storeys in storey 5 and 10 were modelled. Also the massive wooden slabs in the corridors and the concrete slabs in storey 6 and 11 were included. The modules and the rest of the building were modelled as added mass. Based on Eurocode 1-4 and matlab scripts from

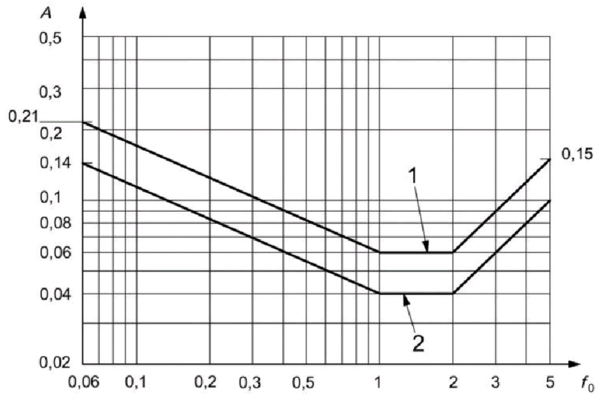


Figure A.4: Evaluation curves for wind-induced vibrations from ISO 10137:2007 [29]

the professor of structural dynamics at NTNU the structural damping of the frame structure is ξ= 1.5%. Findings from the research were [77]:

- The eigenfrequency of the model was much higher than the common approximation for eigenfrequency $f_1 = 46/h$.
- The model had a much higher eigenfrequency when the mass of the modules was not included.
- Using a pinned or rigid support almost had no effect on the eigenfrequency.
- Increasing the damping ratio with 500% to ξ= 9% reduced the eigenfrequency with a maximum of 0.3%. So the effect is negligible.
- The maximum acceleration calculated with Eurocode 1-4 is 70% to high for discomfort levels.
- Increasing the damping ratio will decrease the acceleration. If the damping ratio is ξ= 4% or higher the criteria for comfort are met. It could be argued that the building has such a high damping ratio because of the complexity of the structure and the modules.
- Increasing the mass will give a lower eigenfrequency and a lower acceleration.
- Reducing the height will give a higher eigenfrequency and a lower total mass. This reduces the acceleration.
- Using the dynamic wind load instead of the Eurocode gives a 10x higher acceleration but is deemed to be incorrect due to faulty load calculations.
- Since the top part of the building had a low stiffness Sweco proposed to add concrete slabs as the roof, this reduced the acceleration and decreased the eigenfrequency. Which now are 1.09 Hz for the z-direction and 1.18 Hz in x-direction.
- Stiffness in the connections has low impact on the global stiffness, although this should probably be investigated further.
- The eigenfrequency was reduced when simplified models for the modules were added. This is because the modules have high damping that increase the global damping ratio.
- Estimating the damping ratio was difficult and resulted in fluctuating results.

From this research the final design of Treet was altered to have concrete roofs to decrease the acceleration. Now the accelerations at the top of the building become 0.059 m/s^2 for the z-direction and 0.058 m/s^2 for the x-direction.

Another study was done for the wind-induced motions of Treet [12]. First the modules were built to scale and tested. Individual modules and stacked modules were used. The test was performed using an impact hammer and accelerometers that measured the acceleration time history. From these results the natural frequency and damping were established. A simplified model with the same properties was made for the modules. Than, a FEM-model was created for the whole structure including modules, power storeys, the framework and concrete slabs on the power storeys as well as on the roof. Another addition was made where steel braces were included under the power storey to avoid local deflections and vibrations. The modules under the first power storey were not modelled since they are supported solely by the foundation and do not connect to the truss work. In the model the modules are connected in all adjacent joints because they cannot move independently. The weight of these modules was added to the model. The foundation of the building with the basement and steel core piles were modelled true to their geometry. However, in the dynamic analysis the basement was fixed in horizontal direction and the weight was set to zero. Also, the piles only had axial stiffness. This was done to avoid local effects. The damping ratio used in this model is 1.9% based on the ratio for timber bridges from the Eurocode [53]. The natural frequency of four modules is a lot higher than the global frequency. This results in the modules acting like a rigid body and following the vibration of the concrete slab they rest upon. The results for the global design in the z-direction are a natural frequency of 0.75 Hz and a peak acceleration of 0.048 m/s^2. For the x-direction it is 0.89 Hz and 0.051 m/s^2.

When the building was finished data was collected with accelerometers on the vibrations and dynamic properties. This data was processed in a thesis research from NTNU [30]. In this study the damping ratio and accelerations were determined for different modes. These modes are the bending mode in Z-direction, the bending mode in X-direction and the torsional mode. The damping ratio for these modes respectively are 1.84%, 1.61% and 1.98%. Meaning that the first research was to conservative with a damping of 1.5% and the second research was higher with a ratio of 1.9%. The natural frequencies for the building are 0.97 Hz, 1.12 Hz and 1.79 Hz. This is higher than the other studies because non-structural elements are not modelled. These elements increase the stiffness of the building and have a positive effect on the acceleration of the building. The maximum acceleration measured in the day of the experiment was only 0.0082 m/s^2 [30]. This is so low because the wind load on this specific day was not the maximum design wind load.

A.1.4. Fire design

The structural system of Treet needs to resist 90 minutes of fire without collapsing. For the secondary load bearing systems this is 60 minutes. One protection measure is fire stops on the facade for every two floors that are connected to the horizontal beams in the framework [48]. Other measures taken are fire resistant paint on all exposed timber, sprinklers and escape routes that avoid combustible surfaces and have an elevated pressure. The modules were also designed to resist a fire of 90 minutes so if a fire catches in an apartment it will burn out in the module [2]. However, later tests showed that the full course of fire for one apartment was 74 minutes and so the design of the modules was reduced to withstand a fire of only 74 minutes [70].

The structural fire design is based on the reduced cross-section method from Eurocode 5 [58] and does not take these extra measures into account. Meaning that the load bearing framework also needs to be safe when exposed to fire for 90 minutes. The reduced cross-section method is used with the accidental design situation. To calculate the reduced cross-section the following formulas are used:

$$d_{\text{char},n} = \beta_n t \tag{A.1}$$

$$d_{\text{ef}} = d_{\text{char,n}} + k_0 d_0 \tag{A.2}$$

For this situation with glulam beams and unprotected surfaces exposed to fire for 90 minutes we get the following calculation.

$$\begin{aligned} d_0 &= 7 \text{ mm} \\ \beta_n &= 0.7 \text{ mm/min} \\ t &= 90 \text{ min} \\ k_0 &= 1 \\ d_{ef} &= 0,7 \cdot 90 + 7 \cdot 1 = 70 \text{ mm} \end{aligned}$$

This gives a reduction of the cross-section of 70 mm [8].

The timber elements encapsulate the steel connections. Because the charring for 90 minutes reaches a depth of 63 mm, all steel dowels are placed 65 mm from the edge of the timber. The dowels are therefore assumed not to increase the heat flux towards the steel plates. The steel plates are placed 108 mm from the outside of the timber. Some cross-sections had to be increased for fire safety. All gaps and slots are protected with intumescent fire seals [48]. These seals expand when they are exposed to heat. The seals fill up all the holes to stop the fire from spreading [34]. Figure A.5 shows a detail of a connection with a fire seal as well as how this seal would expand.

Fire safety for the load-bearing system is done with reduced cross-section method, fire resistant paint in all escape routes [48], sprinklers [70] and fire is supposed to burn out already in the modules. No extra gypsum added and gaps are filled with fire proof joint filler. Except for the power story the modules do not have glulam elements running through it [42]. In the power storey the glulam is exposed and will have sprinklers and reduced cross-sections.

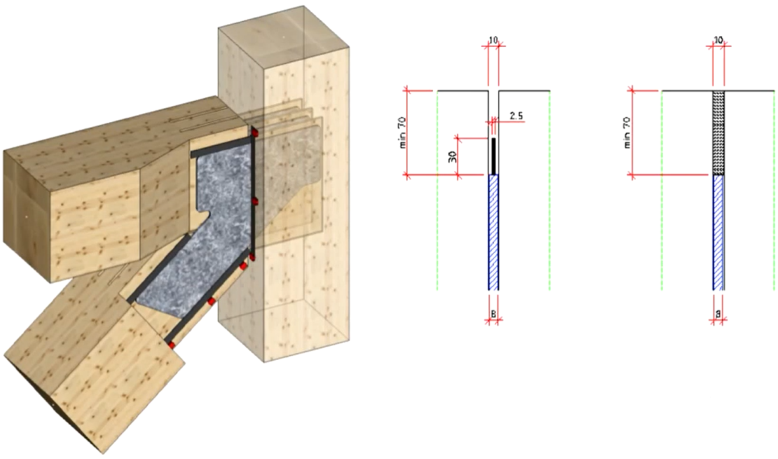


Figure A.5: Connection detail with intumescent fire seals [42]

A.1.5. Connections

The connection design of Treet is highly affected by fire safety. To deal with this all dowels are placed within 65 mm of timber and the plates 108 mm from the sides. The connections of Treet are all slotted-in steel plates. These types of connections are used because the companies involved have a lot of experience with this type of connection from previous projects with timber bridges. The connections are known to perform well and can take up a lot of tension [11] [3]. Figure A.6 shows a detail of a connection. However, the other view of the connection is not known. A possible design for the connection is derived from other details. Very important to note is that this is highly speculative.

We know that the column has a size of 650x405mm. From figure A.6 it can be derived that the connection has 3 steel plates with a thickness of 10 mm and that there are 7 rows of dowels parallel to the grain. The steel plate has a maximum width of:

$$650 - 110 - 110 = 430mm$$

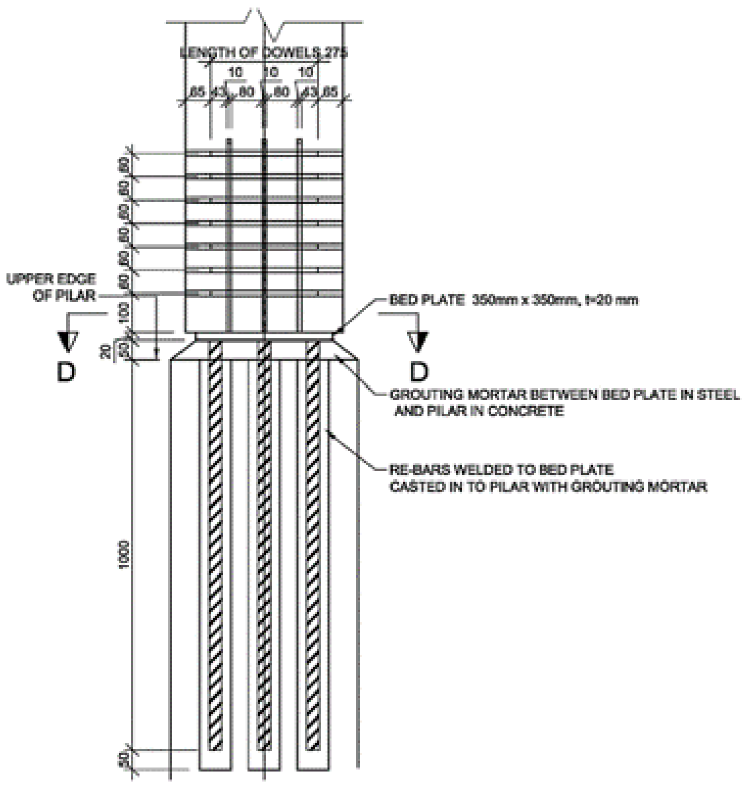


Figure A.6: Joint detail column to foundation, [8]

From other details we can derive that the space between the outer-bolts and the end of the plate is 25mm. This leaves 380 mm to place the bolts. In other details a space of 45 mm is left between the bolts. So this means that 9 bolts would fit with a heart-to-heart distance of 47.5 mm. The bolts are presumed to have a diameter of 12 mm. To calculate the capacity of the joint Eurocode 5 for timber is used [52]. To calculate the capacity of steel to timber connections the Johanson model is used, this model is explained in paragraph 3.1 and appendix B. The joint is also tested on block and plug shear failure as described in the Eurocode. The calculations are done with formulas from appendix B. The calculation determines that mode g and m from the Johanson model give the lowest capacity. The shear capacity is decided by the back of the connection and not the sides. According to Magne Aanstad Bjaertnes [11] a slotted steel timber connection is good, when the connections fails first at the back. If it fails on the sides the connection is too long.

When checking the compressive strength of the column with the compression force of 4287 kN the unity check for the effective cross-section is 0.85 which is good. However, the capacity of the connection is calculated to be 1050 kN with a unity check of 4. This means that this connection is not designed to withstand the compression force. The force is probably assumed to transfer directly to the end plate supporting the column.

A point of interest for the construction of Treet is the connections in the corners of the facade like in figure A.7. These connections do not meet at the same point because constructing a 3D-joint is very difficult and also it would make assembly a lot harder. To avoid moments it is very important that elements all join in one point and there is no eccentricity within the joint. The moments are also avoided because you can make a closing force polygon with the horizontal beam and the column [3].

Slip in the connections can also have an influence on the dynamic behaviour of a building. Since Treet has so many dowels it was presumed that no slip would occur based on knowledge from timber bridge design [8]. Nonetheless, it was determined that slip in joints would have a minimal effect on the force distribution and fundamental frequencies if it were to occur [77].

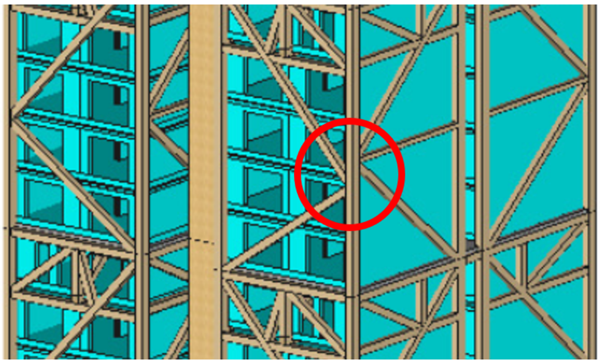


Figure A.7: Connection corner facades [8]

A.2. Mjøstårnet, Brummunddal Norway

Mjøstårnet is a mixed-use building with apartments, a hotel, offices and a restaurant. It is 18 storeys with an architectural height of 81 meters. This makes it the highest timber building in the world. The building is located next to the biggest lake in Norway, lake Mjøsa and was completed in March 2019. The building was initiated by investor Arthur Buchardt a Brummunddal native. He believed that the building could be a representation of the green shift in the construction industry. As well as proof that high-rise could be made using local suppliers, local resources and sustainable materials [5]. Moelven Limtre was responsible for supplying all the structural timber and Sweco was the structural engineer for the project [4]. These companies were chosen because they had also worked on Treet. The building is made with 2600 m^3 cross- and glue-laminated timber from which most is locally sourced, reducing the embodied carbon created by transportation.

Figure A.8: Mjøstårnet, Brummunddal [4]

A.2.1. Structural system

The load bearing system is made up of glulam trusses in the facade and has glulam internal beams and columns. CLT walls are used to carry the vertical loads of the elevators and stairs. The global stiffness is provided by the trusses in the facade. All the structural timber is located inside the facade which increases the durability of the wood and decreases the maintenance. The floor systems of floor 2 to 11 are prefabricated wooden Trä8 systems. The upper floors 12 till 18 have a prefabricated bottom where concrete is cast on top. This was done to increase the weight in the top of the building [5]. Increased weight, especially in the top, can decrease swaying of the building and increase comfort.

The structural system is shown in figure A.9. Since the structure is light there will be big tension forces at the foundation. Therefore, the foundation has considerable anchorage [6]. The glulam elements are connected with dowels and slotted steel plates. Around 128 tonnes of steel is used in the super-structure of Mjøstårnet [47]. The floor heights are on average 4 meters.

Figure A.9: Structural model Mjøstårnet, [4]

The stability in Mjøstårnet is provided by diagonals in the facade. On the short side of the facade these span all the way across the facade to ensure enough global stability. In the long facade they did not need to span all the way across the facade. There is quite a big difference between the structures of Treet and Mjøstårnet. This is because from Treet they learned that using less but bigger elements worked more efficient, especially for higher buildings. It is even thought that this stability system could make buildings up to 140 meters high. The diagonals cross the beams and columns in the facade. This does not cause moments because all the connections are presumed to be hinges. The columns in the building span four floors. The columns continue when a diagrid crosses and the diagrid is split-up. When a diagrid encounters a beam, the beam is split-up. The sizing of the elements in the stability system was mostly determined by the dynamic behaviour caused by wind. Some of the elements had to be enlarged because of the fire design and connections [3]. The global deformation is 140 mm.

$$H/500 = 81000/500 = 162mm$$

So the deflection of the building is $L/579$ [5]. The structure was made to be robust. This means that when one element fails not the whole building will collapse. The most dangerous element to lose would be one of the diagonals [11].

A.2.2. Floor plan and loads

The floor plan of the building is 17 x 37 m [23]. Figure A.10 shows a typical floor plan of the building. In red the columns are shown. The CLT core is placed at the top in the middle. The grid of the columns was determined by the sizes of the hotel rooms in the building [3]. To accomplish a floor span of 7.7 meters a Trä8 floor system was used. This floor has a top plate of LVL and then glulam flanges underneath it. This system uses less timber than using a CLT deck. The vertical forces are mostly taken up by the columns and also a bit by the diagonals where they are present [5]. The loads that were used on the building are[11]:

- Own weight of timber floors $2.5\ kN/m^2$

- Own weight of floors with concrete deck $8.5\ kN/m^2$
- Own weight facade $1.0\ kN/m^2$
- Variable floor load for hotel and homes $2.0\ kN/m^2$
- Variable floor load for offices $3.0\ kN/m^2$
- Variable floor load for balconies $4.0\ kN/m^2$
- Wind load $1.3\ kN/m^2$

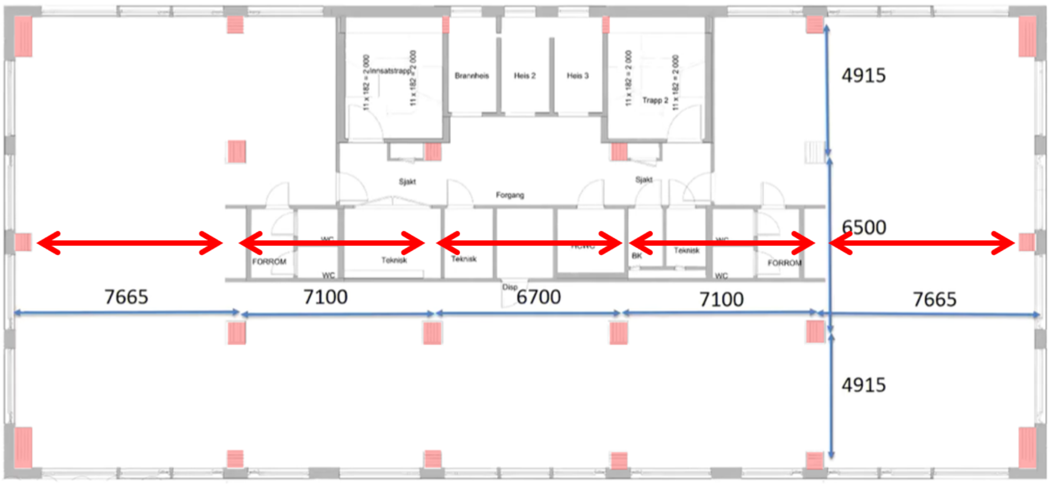


Figure A.10: Typical floor plan Mjøstårnet [6]

Floors two through seven are offices and eight through 16 are homes and hotel rooms. Floor 17 has a variable load of $3.0\ kN/m^2$ and 18 of $4.0\ kN/m^2$ because it is an outdoor space. The vertical loads are transferred to the columns in the facade from half the floor span, so for 3.85 meters. The columns in the corner are larger than the other columns because they are part of the stability system [11]. These columns have dimensions of 1485x625 mm and have the largest forces in them.

As can be seen in figure A.10 the floor spans in the long direction of the building. In this way the floor loads rest on the short facade. The short facade also gets more wind force because it has to take up all the wind force that hits the long facade. Loading the short facade with the floor loads can cause the corner columns to have less tension force and a lower overturning moment.

Cross-section	Force $[kN]$	$\sigma_{\mathrm{N,Ed}}\ [\mathrm{N/mm^2}]$
1485x625	-11500	12.4
1485x625	5500	5.9

The forces given in the table are that of the corner columns. Sadly, no forces in the diagonals are known. The sizes of the diagonals on the short side are 625x990 mm and on the long side they are smaller 625x495 mm.

A.2.3. Dynamic response

The design of the elements started with an analysis based on the dynamic response [3]. The knowledge that was gained from Treet and the three researches mentioned in paragraph A.1.3. were used to determine the dynamic response of Mjøstårnet. To calculate the peak accelerations a damping ratio of

1.9% and a wind speed of 22 m/s were used. They found that the assumption of 1.9% corresponded enough with the damping ratios found in Treet of 1.84%, 1.61% and 1.98%. The peak accelerations found for the top floor are approximately 0.066 m/s^2 and 0.045 m/s^2. This crosses the comfort criteria but only for the top floor. This floor was sold with this information [5]. To achieve an acceptable acceleration the 6 top floors of the building have a concrete floor instead of a timber floor. It was actually found that applying a 1.5 m layer of concrete on the roof of the building would be the most effective use of concrete but distributing it over 6 floors seemed more reasonable [3].

A program was set up to quantify the structural damping of timber high-rise and Mjøstårnet is included in this study. The program is called DynaTTB standing for The Dynamic Response of Tall Timber Buildings under Service Load. The program will try to create a model that can accurately predict the dynamic behaviour of timber buildings by comparing it to measurements from the building [7]. For Mjøstårnet the first measurements were done on-site but sadly the main document reporting these has not been published yet [72]. However, some results were shared with thesis students who were a part of the DynaTTB project. Figure A.11 and A.12 show the fundamental frequencies that were calculated by Sweco for the design of the building and what was measured after the building was in use.

(a) Numerical model by SWECO

Mode	Frequency [Hz]	Mode directionality
1	0.33	Longitudinal
2	0.37	Transverse
3	0.59	Torsional

(b) Measured (DD-SSI)

Mode	Frequency [Hz]	Mode directionality
1	0.50	Transverse
2	0.54	Longitudinal
3	0.82	Torsional

Figure A.11: Fundamental frequencies of Mjøstårnet [33]

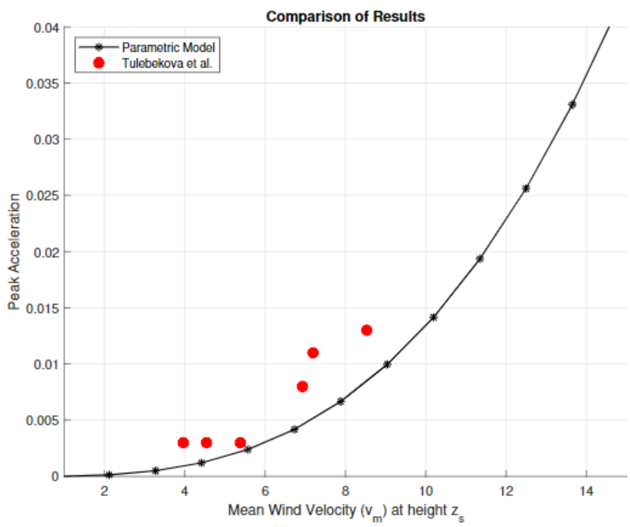


Figure A.12: Comparison of peak acceleration from on-site measurements by Tulebekova and Parametric model for Mjøstårnet [33]

The differences can be explained by an underestimation of the stiffness of the foundation in the model. The damping ratios were also determined from measurements for the first 3 modes being 1.685%, 2.458% and 1.863%. In the parametric model a damping of 1.5% is used. When comparing the peak accelerations from the model and measurements we see that the measurements give higher values. In the thesis they state that this difference is partially caused by the measurement instruments being placed higher than the calculated points in the model. The peak acceleration based on the design wind load from the parametric model is 0.114 m/s^2 in transversal direction, which is twice as high as the recommended threshold, and 0.038 m/s^2 for the longitudinal direction. No specific explanation is provided as to why they think these accelerations differ so much from the ones calculated by Sweco. Important to note is that their model gives a higher acceleration on the top floor for a higher damping ratio.

The final conclusion of the research was that the frequencies were underestimated and that the acceleration was highly dependent on the damping. Nevertheless, the damping used to recreate the most accurate model was not the same as the damping found on-site. This can be explained by the underestimation of the stiffness of the foundation and not including the stiffness of the exterior walls. The parameters that had the greatest influence on the dynamic behaviour were: vertical stiffness of foundations, material stiffness of timber frame, axial stiffness of connections in the diagonal bracing system, rotational stiffness of connections in the timber frame and stiffness of exterior walls. Parameters that had almost no influence were: horizontal foundation stiffness, rotational stiffness of foundations, timber floor material stiffness, timber floor connection stiffness, material stiffness of shaft walls, rotational stiffness of diagonal connections and axial stiffness of beam connections [33].

In 2022 Tulebekova et al. published a paper on the modeling stiffness of the connections and non-structural elements. The connections are modelled as 'connection-zones' with a modified stiffness. They propose that the stiffness of the 'connection-zone' is proportioned to the timber element sizes. In this manner the influence of the glulam connection stiffness and stiffness of the non-structural elements on the dynamic behaviour can be taken into account. It is concluded that the axial stiffness of the connections from the diagonal are the governing parameter for the dynamic behaviour. Whereas the rotational stiffness of the connections from the beam not having much effect. It is also found that the connections behave as semi-rigid joints. The new found damping ratio for the short side is 1.5%, for the long side it was 2.3% [73].

A.2.4. Fire design

For the fire design of Mjøstårnet it was determined that the main load bearing system must be able to endure a 120 minute fire and must withstand a burnout scenario. Meaning the building must stop burning before it collapses [35].

To ensure the safety during the cooling phase of the burnout tests were performed by SP Fire Research for Sweco. The test pieces were three glulam columns of GL30C with a profile of 405x460 mm. One of the columns featured a slotted steel plate connection similar to the ones used in the building. During the test the columns were exposed to a fire that corresponds with the ISO 834 curve for a 90 minute fire [10]. A 90 minute fire was used because the Norwegian building regulations state that the pre-accepted performance for the load-bearing main system is 90 minutes if a building is in the fire class for buildings with more than 5 floors [18]. After the columns were burned in the oven for 90 minutes the decay period of the fire starts where no extra heat is added. But the specimen was exposed to the remaining heat in the oven. It was found that the fire will eventually burn out without using external extinguishing. After applying the fire for 90 minutes the charring depth was about 50 mm. During the cooling phase a maximum extra charring depth of 24 mm was reached before the fire went out. The column with the connection had steel dowels that were placed 65 mm from the edge of the wood and the temperatures were measured in these dowels [10]. The dowels reached no more than 250 degrees at any point during the test and were therefore determined safe. It was also determined that using wooden plugs instead of leaving the dowels holes open did not have a positive effect because the charred wood expands and covers the hole [35]. In figure A.13 the charring process for the columns without the connection can be seen plotted in blue and red. In green the design charring rate for one-dimensional charring from the Eurocode is plotted.

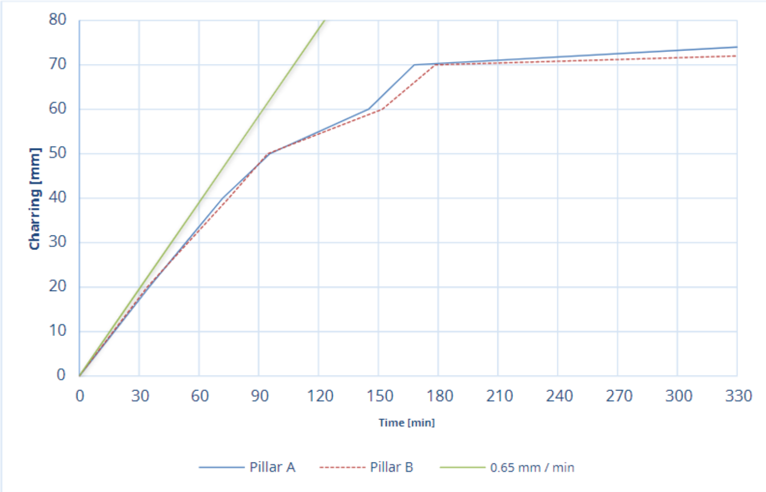


Figure A.13: Charring process of glulam column test [10]

The parametric fire curve was also used to calculate the charring depth for Mjøstårnet. With the parametric fire curve the fire energy from the wooden structures that burn is added in the input for the curve. This calculation is repeated 5 times until the calculations converge indicating that the charring will stop at a certain depth. For Mjøstårnet this depth is 30 mm after 40 minutes. However, these results were not used in the design. The reduced cross-section method from the Eurocode was used to be on the safe side [35]. This gives us a one dimensional charring depth of 85 mm as can be seen in the calculation below.

$$d_{\text{char},0} = \beta_0 t = 0.65 \cdot 120 = 78mm$$

$$d_{\text{ef}} = d_{\text{char},0} + k_0 d_0 = 78 + 7 = 85mm$$

This is why all steel parts in every connection in Mjøstårnet are embedded in 85+ mm timber.

The protection measures taken for the structural system are the reduced cross-section method, 120 minutes fire resistance, sprinklers and every storey is sealed. Measures are also taken for the secondary systems. The timber facade is covered with fire retardant and has non combustible insulation materials. Also, the spread of fire is stopped between each floor. The same counts for the elevator shaft. Sprinklers are installed in the complete building and non combustible materials are used in hidden and technical rooms. All hotel rooms and apartments are their own fire cell as well as the office floors. The floors, shafts and walls have a fire resistance of 60 minutes. As an extra safety measure the fire brigade has a control room in the building, a fire elevator and direct connection with the alarm.

A.2.5. Connections

The connections in Mjøstårnet are slotted-in steel plates as well because of the expertise of the companies involved in the project. The connection of the column with size 1485x625 mm to the foundation has 4 steel plates. The column short side of the column is made from five parts of 125 mm with the steel plates between them. The ends of the steel plates are left 340 mm from each side of the end of the timber. The dowels are embedded 85 mm within the timber for the fire safety. In the direction parallel to the grain there are 10 dowels. In the direction perpendicular to the grain there are also 10 dowels[11]. The thickness of the plates and dowels is measured to be roughly 15 mm. The distance from the end plate to the bolt is 25 mm just like in Treet. Estimating the distances between the bolts give a distance of 120 mm from the last bolt to the end of the column. The distance between the bolts parallel to the grain is approximately 100 mm [11]. The distances between the bolts in the direction perpendicular to the plane is roughly 85 mm.

In appendix B a connection calculation is shown for this connection. The calculated tensile and compression capacity is 5039 kN. The unity check for the connection under a tensile load of 5500

kN is 1.1 and for a compression of 11500 kN it is 2.3. Meaning the compression is again expected to be transferred end-grain to end-grain. The capacity of the effective cross-sections are 0.47 for tensile force and 0.63 for compressive force.

The connection design for the columns in the middle of the building is also interesting. These columns are loaded purely under compression so no connection is needed. The columns are stacked on top of each other end-grain to end-grain [3].

A.3. Monarch IV, the Hague Netherlands

The plot for Monarch IV was bought in 2018 by the government of the Netherlands. The main goal of the project is to create rapid availability of extra square meters of high-quality, flexibly deployable and sustainable office space for the government in The Hague. A building with 19000 m^2 gross floor area should provide 900 work spaces should be created. The tower is designed to be 72 meters high with 20 floors. This building should have flexible floor plans without having to change the stability system and installations. Because the government needs to have extra office spaces before 2024 it is paramount that the project can be realised in a short period of time. Since the building is made for the government they see it as their social responsibility to create a green future proof building. This entails using the least amount of materials with the least amount of CO2 emission and a possibility for circularity. To accomplish these goals a stability design in timber was made. This design was made parametric to increase the speed of the design process, and building in timber also increases the speed of the construction. As well as having to do less with nitrogen requirements that can elongate the permit process [81]. The principle design was created by Royal HaskoningDHV and the project tender closed at 31 December 2021 [1].

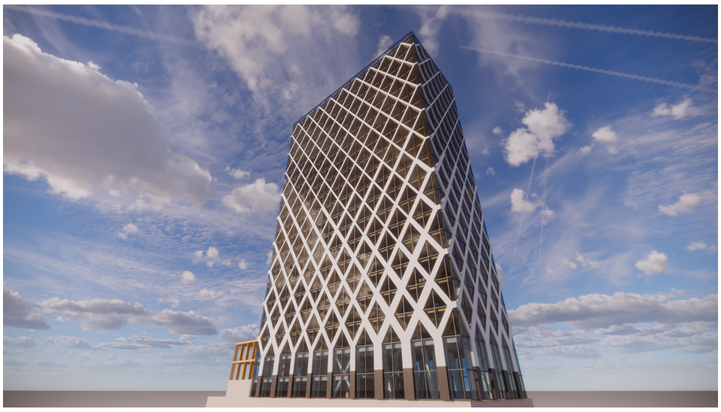


Figure A.14: Render Monarch IV, the Hague [9]

A.3.1. Structural System

The stability system of Monarch is a diagrid in the facade of the building from the second floor up, this can be seen in figure A.15. The system is made with laminated wood GL28c and steel slotted plate connections. The floor heights are 3.6 meters. The diagrid has diagonal components as well as horizontal beams that occur every floor. The rhythm of the diagrid is that there is a full cross on every two floors. In this way the loads of every floor can be easily transferred to the diagrid. The width of the diagrid is determined by the width of the facade panels. The diagrid has no columns so all vertical forces go through the diagonals. The whole facade has diagonals so every part of the facade provides stability to the building. This type of facade and stability system was chosen based on aesthetic considerations from the architect. As well as this providing freedom in the floor plan which was required by the client [9]. There is a CLT core present but this does not carry any horizontal loads, only vertical. Because the core does not have a function in the stability system it could be placed anywhere in the floor plan again giving extra freedom. The elevators and stairs are placed within the core. To take up the vertical forces between the diagrid and the core, columns are used. The core stops above the basement in order to avoid obstructing the layout of the parking area. Below the second floor there is a concrete plinth. Above ground there are concrete piers with steel wind bracing in the facade. These piers have rigid supports to contribute to the stiffness and dynamic behaviour of the building. The floors from the second floor down are all made in concrete. The CLT core also changes into a concrete core for the bottom two floors and basement. Some columns in the basement need a larger spacing than the columns on top so steel V-shaped constructions are used to transfer the load from two columns to one [69].

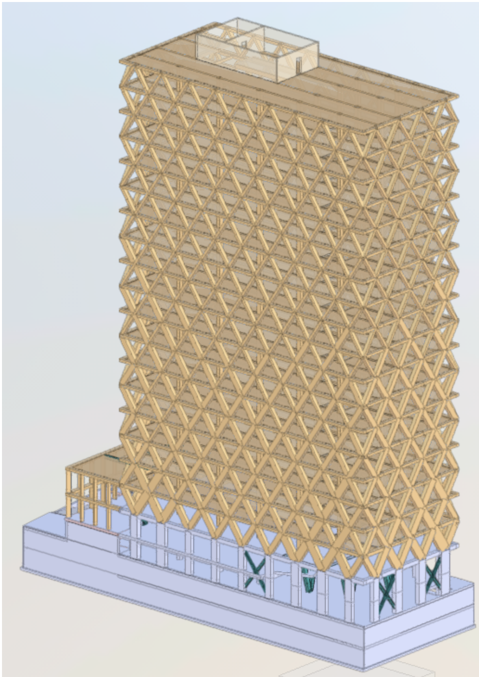


Figure A.15: Structural model Monarch IV [69]

The stability system is modelled and calculated with a parametric script in Grasshopper. To optimize the structure, Karamba was used. The only parameter in the script was the element size of the diagonals and beams in the stability system. The sizes of the elements all had the same depth of 600 mm to make it easy to connect the facade and floors. The system was tested with own weight, live loads and wind loads from all 4 directions. The criteria to which the model was tested was a maximum stress of 5.5 N/mm^2 in the timber elements. After this the model was exported to SCIA Engineering to do a more elaborate check. The diagonals are checked on stress in the joint and buckling under compression. This is because no moment occurs in the diagonals due to it being a diagrid with hinged connections. The horizontal beams are checked on stress in the joint, buckling under compression, beam cross-sections subjected to either bending or combined bending and compression. These extra checks are added because of the floor loads working on the beams. The deformations of the global design are calculated in SCIA Engineering and are 41 mm in the slender direction and 8 mm in the other direction. The requirement is:

$$H/500 = 72000/500 = 144mm$$

So this is easily satisfied. The second order effect is also calculated and this causes no problems because the building is very stiff compared to the weight. In the final design we have four sizes of beams for the diagrid. The smallest size is 400x600 mm, this is the most occurring size and can be found mostly in the top of the building. The other sizes are 800x600 mm, 1200x600 mm and 1600x600 mm. The long side of the facade has relatively smaller elements than the short side of the facade. This is logical since it has to take up more wind from the long facade and also has a smaller moment of inertia than the long facade. The beams have 9 different sizes ranging from 400x600 mm to 820x600 mm.

For both beams and diagonals the buckling and bending are not the normative checks and neither is the dynamic behaviour or global deflection. The normative check is those of the joints for every profile [69].

A.3.2. Floor plan and loads

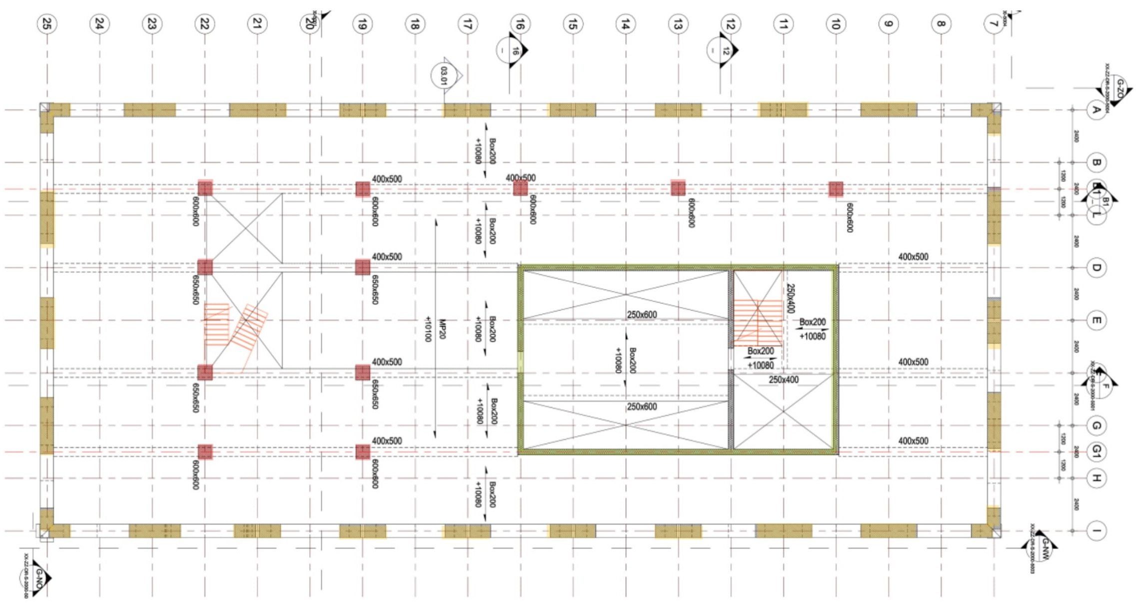


Figure A.16: Floor plan Monarch

The floor plan of the building is 44.7 X 20.7 m. The long side of the building is divided in 7.2 meters between each column. The diagrid in the facade however, occurs every 4.8 meters. The short side of the building is divided in 5 parts where the middle part is 4.8 meters in width and the other four are 3.6 meters wide. The diagrid is again 4.8 meters apart. The floor system is made of a box floor spanning in one direction which is supported by beams spanning in the other direction. The beams are parallel to the long facade and have a length of 7.2 meters. The floors have a shorter span and span in the other direction. Figure A.16 shows the floor plan of Monarch with the internal columns in red, the core in green and the diagrid in yellow.

The floor system was chosen based on a study of variants to see what option would give the least floor height. The box floor is used instead of a CLT floor so that extra weight can be added in the floors. This weight can improve the acoustic behaviour of the floor. [9]. This extra weight can also have a positive effect on the dynamic properties of the building.

The loads working on the upper timber part are:

- Own weight of floors $3.0\ kN/m^2$
- Own weight facade $1.0\ kN/m^2$
- Roof $3.75 kN/m^2$
- Variable floor load for offices $4.0\ kN/m^2$
- Variable floor load for the roof $1.0\ kN/m^2$
- Load caused by snow $0.56\ kN/m^2$
- Wind load $1.75\ kN/m^2$

From the vertical loads of the floors $3.6\ m/2 = 1.8\ m$ will be transferred to the diagrid in the facade. All the own weight from the facade will be carried by the diagrid. The maximum forces in the diagrid are caused by the load combination where wind is the predominant load. The load combination for the beams that give the highest forces is the one where the variable floor load for the offices is predominant. The approximate highest forces we get for the diagonals are [69]:

Cross-section	Force $[kN]$	$\sigma_{\mathrm{N,Ed}}\ [\mathrm{N/mm^2}]$
400x600	-1200	5
800x600	-2600	5.5
1200x600	-3550	5
1600x600	-5100	5.5

A.3.3. Dynamic response

The wind vibrations are checked using the dutch national annex. Figure A.17 shows the limit values for different types of spaces. Gebruik 2 is used for residential spaces and gebruik 1 for offices. These requirements are a lot higher than the allowed values from the ISO standard used in Treet and Mjøstårnet.

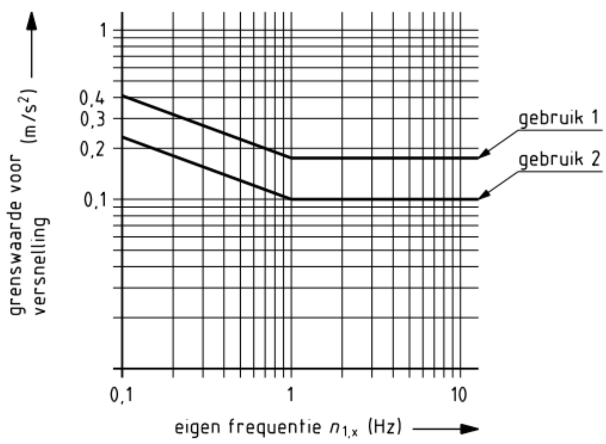


Figure A.17: Limit value for wind acceleration for occupied spaces in buildings [59]

To calculate if the dynamic response is satisfactory the required stiffness EI of the building is calculated when the deformation at the top is $1/500 * L$. The eigenfrequency is calculated with formula A.3 where the building is seen as a cantilevered beam with a rigid connection.

$$f = \frac{1}{2\pi}\sqrt{\frac{3EI}{0.24\mu l^4}} \tag{A.3}$$

An eigenfrequency of 0.47 Hz is found and this gives a peak acceleration of 0.3 m/s^2 which is to high. Hence, the deformation requirement is increased to $1/1000 * L$. The eigenfrequency now becomes 0.66 Hz with a peak acceleration of 0.2 m/s^2 and that is sufficient for offices. The peak acceleration is calculated with the following formula:

$$\sigma_{\mathrm{ax}}(y,z) = c_{\mathrm{f}} \cdot \rho \cdot I_{\mathrm{v}}\left(z_{\mathrm{s}}\right) \cdot v_{\mathrm{m}}^{2}\left(z_{\mathrm{s}}\right) \cdot R \cdot \frac{K_{\mathrm{y}} \cdot K_{\mathrm{z}} \cdot \Phi(y,z)}{\mu_{\mathrm{ref}} \cdot \Phi_{\max}} \tag{A.4}$$

where:

c_f is the force coefficient
ρ is the air density
$I_v(z_s)$ is the turbulence intensity at height z_s above ground
$v_{\mathrm{m}}(z_{\mathrm{s}})$ is the characteristic mean wind velocity at height z_{s}
z_s is the reference height
R is the square root of the resonant response
K_y, K_z are constants
μ_{ref} is the reference mass per unit area
$\Phi(y,z)$ is the mode shape
$\Phi_{\max}$ is the mode shape value at the point with maximum amplitude

A.3.4. Fire design

The fire safety of Monarch is checked using the one-dimensional reduced cross-section method. With a fire resistance requirement of 90 minutes that applies to the entire main supporting structure. Using formula A.1 & A.2 we get:

$$d_{ef} = 0.65 \cdot 90 + 7 \cdot 1 = 65.5 \text{ mm}$$

With the reduced cross-section the accidental situation is checked for the floor beams and the columns and they both suffice with unity checks between 0.38 and 0.62. Regarding the wooden hollow-core slab floors, the basic principle is that the floor itself has a fire resistance of 60 minutes and it is provided an extra 30 minutes at the bottom with fire-resistant plating. The diagrid and slotted steel plates have not been checked yet in this part of the design phase. According to Eurocode 1995-1-2 table 6.2 steel plates with unprotected edges must be larger than 280 mm which is the case for the connection design [69].

A.3.5. Connections

The connections of Monarch are slotted-in steel plates with steel dowels. These types of connections are used because the steel is covered by timber. This is good for the fire safety and for the aesthetics. Since the depth of all the diagonals and beams is the same the distribution of the plates can be the same for all elements. For this design phase only one type of connection was calculated. The connections in the corner meet at the same spot so probably a 3D-connection needs to be made. This connection was made to fit a beam with an area of 600x600 mm and the steel parts of the connection were not embedded within an extra layer of timber for fire safety. The connection has three steel plates and has 11 rows of dowels parallel to the grain and 4 perpendicular. The connection is tested with the Johanson model and for block and plug shear failure. For this particular connection the normative failure mode for the Johanson model was a combination of m for the inner-parts and g for the outer-parts, these modes can be seen in figure B.6. The load capacity from the Johanson failure is roughly 3100 kN. However, the block and plug shear failure give a smaller load capacity, namely, 987 kN. The first part that fails are the sides and not the block tear of the end [69].

A.4. Comparison and conclusion

Treet is 45 meters high, 22.3 meters wide and has a depth of 20.7 meters. Giving it a width to height ratio of 1:2 and 1:2.2 Mjøstårnet is 81 meters high, 37 meters wide with a depth of 17 meters. The width to height ratios are 1:2.2 and 1:4.8. Monarch is 72 meters high, 44.7 meters wide and 20.7 meters deep. Giving it width to height ratio's of 1:1.6 and 1:3.4. This means that Mjøstårnet is the most slender. Where Treet and Monarch use more small elements, Mjøstårnet uses less but bigger elements. Treet and Mjøstårnet both have columns to transfer a lot of the vertical forces, Monarch only

	Height (m)	Floor height (m)	Floorplan (m x m)	Volume building (m^3)	Nr floors	Total floor area (m^2)	Volume glulam facade	**glulam per volume**	**glulam per square (m) *10^2**	Volume of glulam (m^3) façade with columns	**glulam per volume**	**glulam per square (m) *10^2**	Volume of glulam (m^3) facade, beams, columns	**Glulam per volume**	**Glulam per square (m) *10^2**	Global deflection
Treet	45	3-3.3	20.7 x 22.3	20772	14	6463	475*	**2,29%**	**7,35**	475*	**2,29%**	**7,35**	475	**2,29%**	**7,35**	71 mm with foundation, **UC= 0.79**
Mjostarnet	81	4.2-3.8	17 x 37	46798	18	11322	930	**1,99%**	**8,21**	1170	**2,50%**	**10,33**	1400	**2,99%**	**12,37**	140 mm with foundation **UC=0.86**
Monarch	72	3.6	20.7 x 44.7	66621	20	18506	1987	**2,98%**	**10,74**	2312	**3,47%**	**12,49**	2787	**4,18%**	**15,06**	41 mm without foundation ** **UC= 0.57**

Figure A.18: Comparison reference projects on dimensions and timber usage [69] [77] [47]
*Treet does not have an internal structure with columns and beams like the other two buildings
**UC is H/1000 instead of H/500

has diagonals. In figure A.18 the amount of timber used in the buildings is compared. From this figure it can be concluded that the diagrid of Monarch uses more timber per square meter floor than Mjøstårnet meaning it is less efficient. However, Monarch has a lot less lateral displacement. Treet seems the most efficient from figure A.18 but this is because the timber modules add a lot of timber that are not considered here.

For both Treet and Mjøstårnet the comfort criteria from the dynamic behaviour was the most important design criteria. Minority of the elements had to be enlarged due to fire safety and the connections. In Monarch the connection sizes were most important even without using extra timber to embed the material for fire safety. This could partly be explained by the fact that the requirements used in Monarch for comfort are a lot less conservative than those used in the Norwegian buildings. Monarch could also be a lot stiffer than the other buildings because of the large amount of diagonal elements. As well as the building being a lot lighter because there are only timber floors present in the top part of the building. Furthermore, the Norwegian buildings had a more extensive dynamic behaviour assessment. The dynamic behaviour of Treet was tested on-site and gave good results that were better than the expected behaviour. For Mjøstårnet the first outcome of the research based on the on-site testing was not very accurate. However, the measured accelerations were higher than the expected accelerations. This could mean that the dynamic behaviour was underestimated. It was also found that the actual damping for the short direction of 1.5% was lower than the considered damping of 1.9%. Table A.1 compares the dynamic behaviour of all three buildings.

	Damping ratio used	Frequency calculated (Hz)	Acceleration (m/s^2)	UC	Damping ratio measured (%)	Frequency measured (Hz)
Treet	1.9	0.75, 0.89	0.048, 0.051	0.051/0.049 =**1.04**	1.84, 1.61, 1.98	0.97, 1.12 1.12
Mjøstårnet	1.9	0.33, 0.37, 0.59	0.045, 0.066	0.66/0.62 =**1.06**	1.5, 2.3, 2.2	0.50, 0. 54, 0.82
Monarch*	-	0.66	0.2	0.2/0.2=**1**	-	-

Table A.1: Dynamic behaviour of the reference projects
**Monarch has only been checked to see if it falls within the acceptable range, so the real dynamic behaviour is different

The fire design of the buildings is also very important for the global design of the building and especially for the design of the connections. Monarch did not use the reduced cross-section method for the design of the connections. But the connections were still normative for the element sizes. This can be explained by the comfort criteria being less strict. The allowable compression stress in Mjøstårnet is 12.4 N/mm^2 with a force of -11500 kN and a tension stress of 5.9 N/mm^2 with a force of 5500 kN. This compression stress is much higher than the allowable stress calculated for Monarch of 5.5 N/mm^2. When re-calculating the connection for Mjøstårnet with the current Eurocode a maximum allowable compression and tension force of 5039 kN is found. These calculations can be found in appendix B. Presumably, the calculation for the capacity differs slightly and the design was not estimated correctly which can explain the tension capacity difference of 5500 kN to 5039 kN. However, the large difference between -11500 kN and -5039 kN has another explanation. Namely, in the calculation method of appendix B the tension and compression capacity is the same, as it is determined by the capacity of the connection. In Mjøstårnet and Treet they most likely presume that the column can be loaded end-grain to end-grain when it is loaded under compression. Nonetheless, they do not assume slip in the connection. That would mean that the columns would need to be installed perfectly on top of each other since the gap between the columns cannot be pushed closed, because there is no slip. These assumptions of end-grain to end-grain loading and no slip are contradictory. Being, some installation deviations always need to be taken into account and the surface of the columns is probably not perfectly flat. This would result in the columns first needing to be pushed closed before end-grain to end-grain loading can occur. Another design consideration for the connections is that joints that connect two facades to each other are important because making a 3D-connection can be complicated. Also, all elements connected to the joint must not give eccentricities.

B

Calculating a timber slotted-in steel plate connection with multiple plates

This appendix will show how to calculate the capacity and slip of a slotted-in steel plate connection. The connection used in this calculation is the same as the connection that connects the bottom of the corner column to the foundation in Mjøstårnet. It is known that this connection can take up a tension force of 5500 kN and a compression force of -11500 kN. Figure B.1 shows all the different parameters in a slotted-in steel plate connection. The parameters d, t, a_1, a_2, a_3, a_4 of the connection in Mjøstårnet are determined by measuring distances from a drawing provided by SWECO Norge therefore the real values might differ slightly.

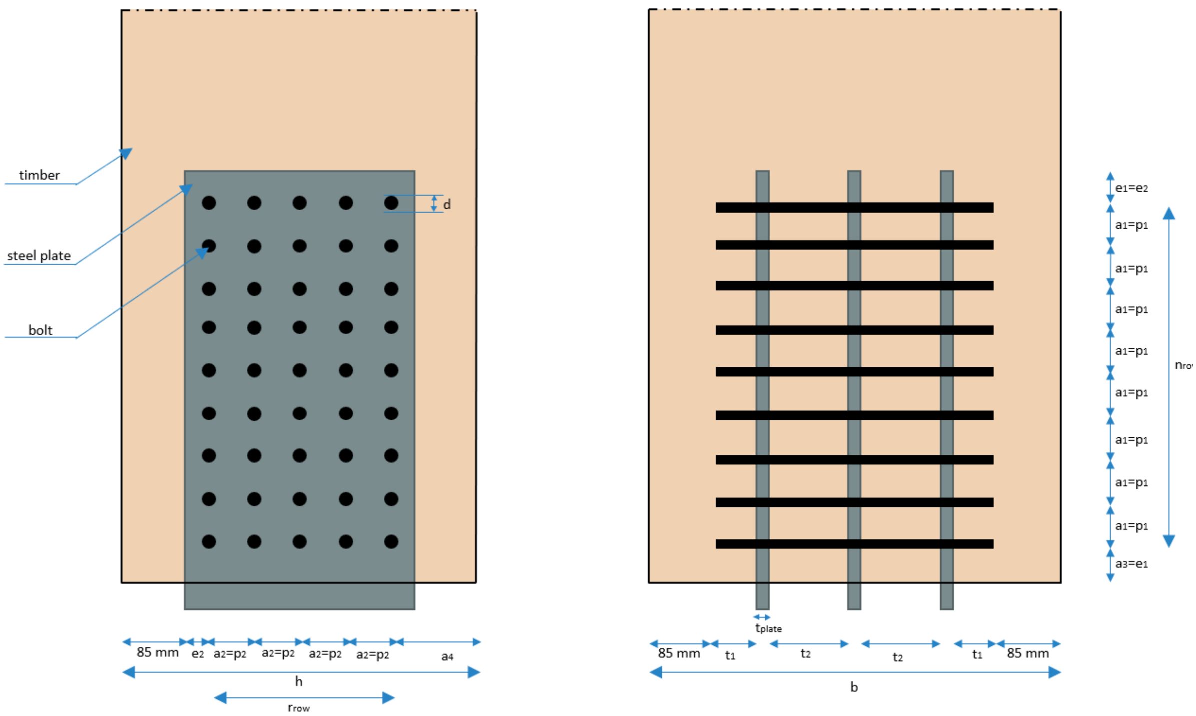


Figure B.1: Example slotted-in steel plate connection with parameters

B.0.1. Characteristics connection

Material properties timber GL30h	
ρ_k	$390 kg/m^3$
ρ_{mk}	$430 kg/m^3$
f_{t0gk}	$19.5 N/mm^2$
f_{c0gk}	$30 N/mm^2$
f_{vk}	$3.5 N/mm^2$
f_{td}	$k_{mod t0gk}/1.25 = 14\ N/mm^2$
f_{cd}	$k_{mod c0gk}/1.25 = 21.6\ N/mm^2$

Material properties plate	
f_{yd}	$355 N/mm^2$
f_{ts}	$510 N/mm^2$

Material properties bolts	
f_{ub}	$800 N/mm^2$
f_{ts}	$640 N/mm^2$
α	0
f_{uk}	f_{ub}

Connection parameters	
b	$625\ mm$
h	$1495\ mm$
n_{plate}	4
t_1	$40\ mm$
t_2	$105\ mm$
n_{row}	10
r_{row}	10
a_1	$100\ mm$
a_2	$85\ mm$

Connection parameters	
a_3	$120\ mm$
a_4	$365\ mm$
e_1	$120\ mm$
e_2	$25\ mm$
p_1	$= a_1$
p_2	$= a_2$
t_{plate}	$15\ mm$
d	$15\ mm$

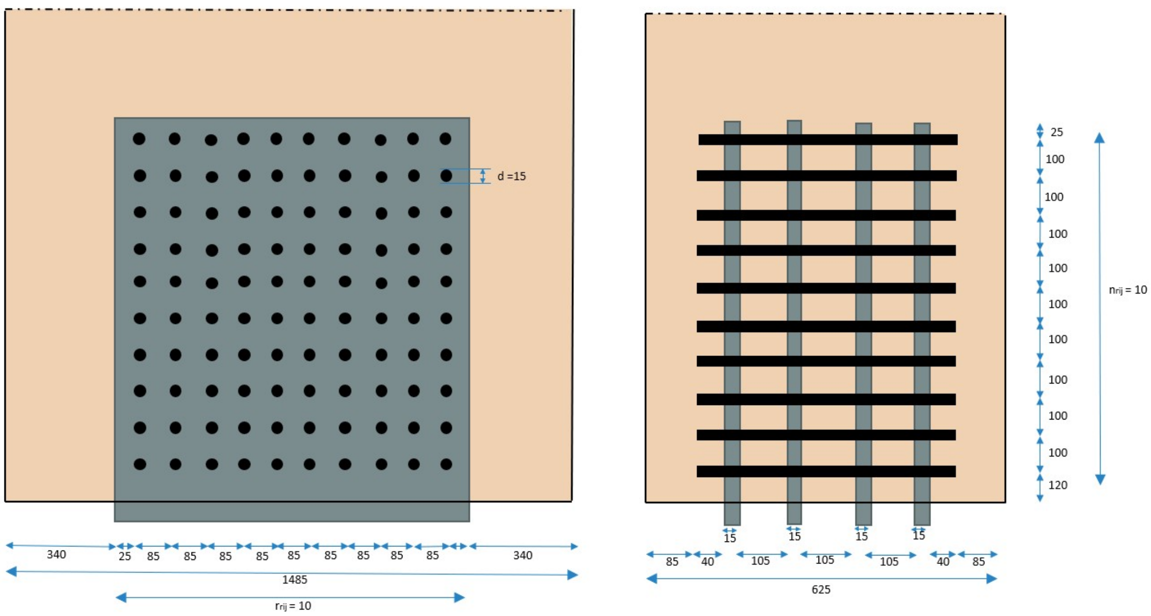


Figure B.2: Connection Mjøstårnet

B.0.2. Minimal distances

The Eurocode 5 [58] for timber and Eurocode 3 [56] for steel give some minimal distances for bolted connections. These can be seen in figure B.4 and B.5 respectively.

Tabel 8.4 — Minimale tussen-, eind- en randafstanden voor bouten

Tussen-, eind- en randafstanden (zie figuur 8.7)	Hoek	Minimale afstanden
a_1 (evenwijdig aan de vezelrichting)	$0^\circ \le \alpha \le 360^\circ$	$(4 + \lvert\cos\alpha\rvert)\, d$
a_2 (loodrecht op de vezelrichting)	$0^\circ \le \alpha \le 360^\circ$	$4d$
$a_{3,t}$ (belast eind)	$-90^\circ \le \alpha \le 90^\circ$	max ($7d$; 80 mm)
$a_{3,c}$ (onbelast eind)	$90^\circ \le \alpha < 150^\circ$	$(1 + 6 \sin\alpha)\, d$
	$150^\circ \le \alpha < 210^\circ$	$4d$
	$210^\circ \le \alpha \le 270^\circ$	$(1 + 6 \lvert\sin\alpha\rvert)\, d$
$a_{4,t}$ (belaste rand)	$0^\circ \le \alpha \le 180^\circ$	max [$(2 + 2 \sin\alpha)\, d$; $3d$)]
$a_{4,c}$ (onbelaste rand)	$180^\circ \le \alpha \le 360^\circ$	$3d$

Figure B.3: Minimal distances for timber from EC5

Afstanden en tussen-afstanden, zie figuur 3.1	Mini-male maat	Maximale maat [1) 2) 3)]	
		Constructies vervaardigd van staalsoorten overeenkomstig EN 10025 met uitzondering van staalsoorten overeenkomstig EN 10025-5	
		Staal blootgesteld aan het buitenklimaat of aan andere corrosieve invloeden	Staal niet blootgesteld aan het buitenklimaat of aan andere corrosieve invloeden
Eindafstand e_1	$1{,}2d_0$	$4t + 40$ mm	
Randafstand e_2	$1{,}2d_0$	$4t + 40$ mm	
Afstand e_3 in sleufgaten	$1{,}5d_0$ [4)]		
Afstand e_4 in sleufgaten	$1{,}5d_0$ [4)]		
Steek p_1	$2{,}2d_0$	Kleinste waarde van $14t$ of 200 mm	Kleinste waarde van $14t$ of 200 mm
Steek $p_{1,0}$		Kleinste waarde van $14t$ of 200 mm	
Steek $p_{1,i}$		Kleinste waarde van $28t$ of 400 mm	
Steek p_2 [5)]	$2{,}4d_0$	Kleinste waarde van $14t$ of 200 mm	Kleinste waarde van $14t$ of 200 mm

Figure B.4: Minimal distances for steel from EC3

The connection is checked for all the minimal distances and the results are given in figure B.5. All values are within the allowed range.

```
a1= 100 > a1min= 75.0
a2= 85 > a2min= 60
a3= 120 > a3min= 105
a4= 365 > a4min= 45
e1= 120 > e1min= 18.0
e2= 25 > e2min= 18.0
p1max= 200 > p1= 100 > p1min= 33.0
p2max= 200 > p2= 85 > p2min= 33.0
```

Figure B.5: Minimal distances for the connection

B.0.3. Strength of the timber element

Tension strength:

$$N_{rdt} = (h - nplate) \cdot b \cdot f_{td} = 565 \cdot 1485 \cdot \frac{14}{1000} = 11746kN$$

Compression strength:

$$N_{rdc} = (h - nplate) \cdot b \cdot f_{cd} = 565 \cdot 1485 \cdot \frac{21.6}{1000} = 18123kN$$

B.0.4. Johansen-Failure modes connection steel timber

To calculate the different failure modes of the timber connection the Johanson model is used. Paragraphs 8.2 & 8.5 from EC5 [58] are used for this calculation and all failure modes are shown in figure B.6. If there are multiple plates in a connection all parts are considered as having a thick plate and failure mode 4 is not possible as this would only happen with a very thin plate and very small t_1 [61]. This gives us the following possible failure modes for the inner and outer parts:

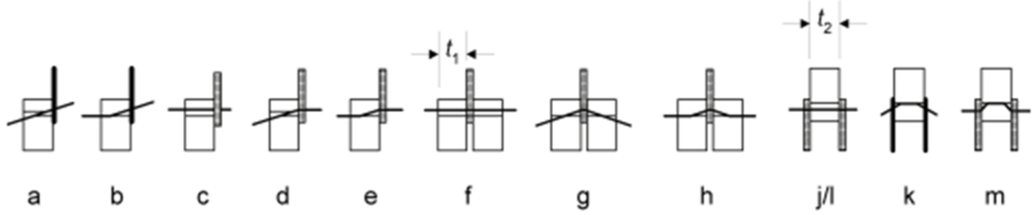


Figure B.6: Failure mechanisms for steel-to-timber connections, from EC5

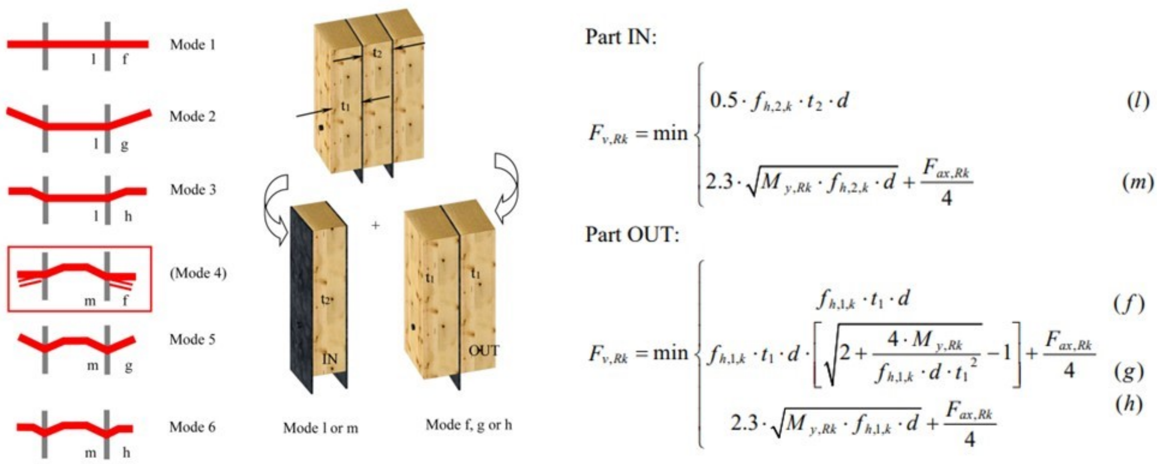


Figure B.7: Possible failure modes for steel-to-timber connections with multiple plates [61]

Where:

$$M_{y,\mathrm{Rk}} = 0,3 f_{\mathrm{uk}} d^{2,6} = 0.3 \cdot 800 \cdot 15^{2,6} = 274188 \mathrm{Nmm}$$

$$f_{h,0,k} = 0.082(1 - 0.01d)\rho_k = 0.082(1 - 0.01 \cdot 15) \cdot 390 = 27.2\ \mathrm{N/mm^2}$$

For a connection with four plates the possible failure modes are shown in figure B.8.

Using the formulas from figure B.7 for the capacity of the failure modes per shear plane we get:

$$\boldsymbol{l} = 0.5 \cdot 27.2 \cdot 105 \cdot 15/1000 = \mathbf{21.4kN}$$

$$m = 2.3 \cdot \sqrt{274188 \cdot 27.2 \cdot 15}/1000 = 24.3\mathrm{kN}$$

$$f = 27.2 \cdot 40 \cdot 15/1000 = 16.3\mathrm{kN}$$

$$\boldsymbol{g} = 27.2 \cdot 40 \cdot 15 \cdot \left[\sqrt{2 + \frac{4 \cdot 274188}{27.2 \cdot 15 \cdot 40^2}} - 1\right]/1000 = \mathbf{15.0kN}$$

$$h = 2.3 \cdot \sqrt{274188 \cdot 27.2 \cdot 15}/1000 = 24.3\mathrm{kN}$$

For parts 2,3 and 4 the failure mechanisms l & m are present. These failure mechanisms are calculated per shear plane but parts 2,3 and 4 all have two shear planes. Therefore, the capacities should be multiplied by two for these parts. The total capacities of the connection can be calculated with the following summations:

Mode 1 : $2 \cdot f + 2 \cdot 3 \cdot l = 2 \cdot 16.3 + 2 \cdot 3 \cdot 21.4 = 161\mathrm{kN}$

Mode 2 : $2 \cdot g + 2 \cdot 3 \cdot l = 2 \cdot 15 + 2 \cdot 3 \cdot 21.4 = \mathbf{158kN}$

Mode 3 : $2 \cdot h + 2 \cdot 3 \cdot l = 2 \cdot 24.3 + 2 \cdot 3 \cdot 21.4 = 177\mathrm{kN}$

Mode 5 : $2 \cdot g + 2 \cdot 3 \cdot m = 2 \cdot 15 + 2 \cdot 3 \cdot 24.3 = 176\mathrm{kN}$

Mode 6 : $2 \cdot h + 2 \cdot 3 \cdot m = 2 \cdot 24.3 + 2 \cdot 3 \cdot 24.3 = 194\mathrm{kN}$

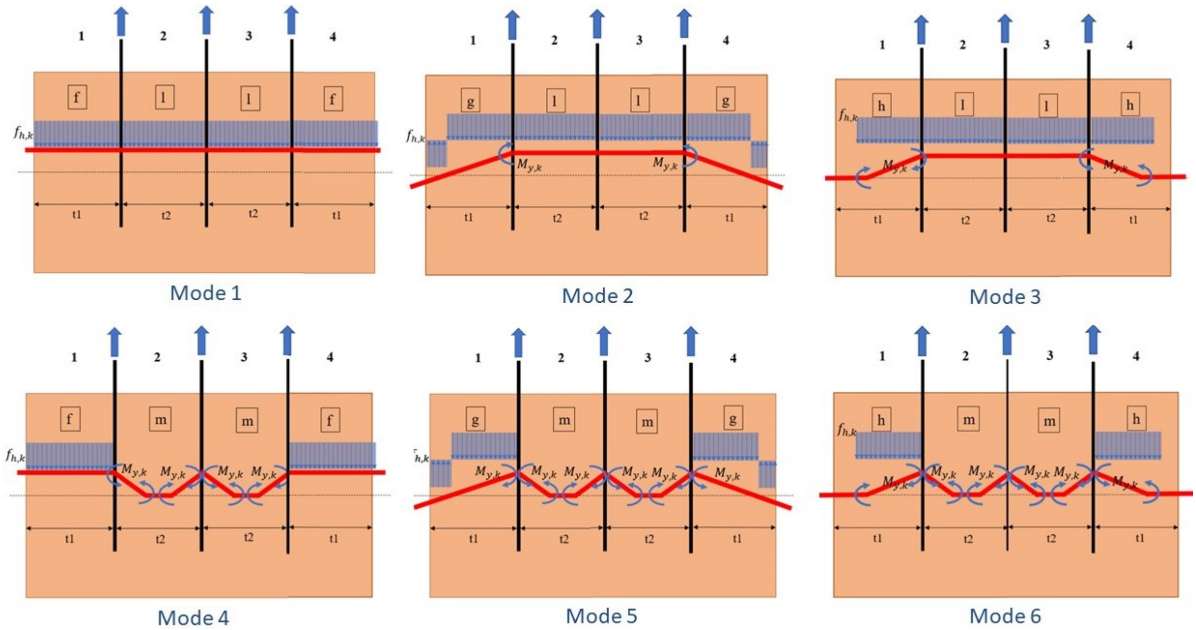


Figure B.8: Failure modes for steel-to-timber connections with four steel plates

The capacity per dowel is $F_{v,Rk,dowel} = 158kN$ from mode 2. The capacity of the total connection is:

$$F_{v,Rk} = F_{v,Rk,\text{ dowel}} \cdot n_{ef} \cdot rows$$

$$n_{\text{ef}} = \min \begin{cases} n = 10 \\ n^{0,9} \cdot \sqrt[4]{\frac{a_1}{13d}} = 10^{0,9} \cdot \sqrt[4]{\frac{100}{13 \cdot 15}} = 6.72 \end{cases}$$

$$F_{v,Rk} = 158.4 \cdot 6.72 \cdot 10 = 10644kN$$

$$\boldsymbol{F_{v,Rd} = F_{v,Rk} \cdot \frac{k_{mod}}{\gamma_m} = 10644 \cdot \frac{0.9}{1.3} = 7369\text{kN}}$$

B.0.5. Block shear failure

To calculate the block shear failure the current EC5 [58], Sandhaas et al. (2018) [61] and draft prEN 1995-1-1 (2021) are compared [16]. For connections with fully penetrating fasteners only block shear failure is considered in the draft. In the current EC5 the inner parts will also fail only on block shear but the outer parts can also fail on plug shear.

In the current EC5 & Sandhaas et al. (2018) the capacity is calculated in the following way:

The block shear capacity of the inner part becomes:

$$L_{net,t} = (a_2 \cdot (r_{row} - 1)) - ((r_{row} - 1)) = (85 \cdot 9) - (9 \cdot 15) = 630mm$$

$$L_{net,v} = ((a_3 + (n_{row} - 1)_1) - (d \cdot (n_{row} - 0.5))) \cdot 2 = ((120 + 9 \cdot 100) - (15 \cdot 9.5)) \cdot 2 = 1755mm$$

$$t_2 = 105mm, \quad f_{t,0,k} = 19.5N/mm^2, \quad f_{v,k} = 3.5N/mm^2$$

$$F_{\text{bs,Rk}} = \max\begin{cases} 1{,}5\,A_{\text{net,t}}\,f_{\text{t,0,k}} \\ 0{,}7\,A_{\text{net,v}}\,f_{\text{v,k}} \end{cases}$$

met:

$$A_{\text{net,t}} = L_{\text{net,t}}\,t_1$$

$$A_{\text{net,v}} = \begin{cases} L_{\text{net,v}}\,t_1 & \text{bezwijkmechanismen (c, f, j/l, k, m)} \\ \dfrac{L_{\text{net,v}}}{2}\left(L_{\text{net,t}} + 2t_{\text{ef}}\right) & \text{alle andere bezwijkmechanismen} \end{cases}$$

en

$$L_{\text{net,v}} = \sum_i \ell_{\text{v},i}$$

$$L_{\text{net,t}} = \sum_i \ell_{\text{t},i}$$

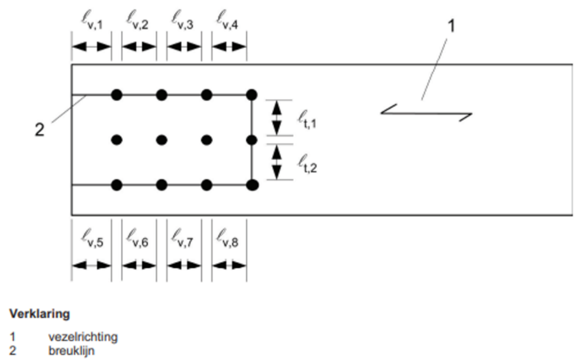


Figure B.9: Block shear capacity from EC5

$$F_{bs,Rk} = \max\begin{cases} F_{tk} = 1.5 \cdot L_{net,t} \cdot t_2 \cdot f_{t,0,k} \\ F_{vk} = 0.7 \cdot L_{net,v} \cdot t_2 \cdot f_{v,k} \end{cases} = \max\{ \begin{matrix} F_{tk} = 1.5 \cdot 630 \cdot 105 \cdot 19.5/1000 = 1935 \\ F_{vk} = 0.7 \cdot 1755 \cdot 105 \cdot 3.5/1000 = 451 \end{matrix} = \mathbf{1935}\boldsymbol{kN}$$

The capacity of the outer parts is the minimum of the plug shear failure or the block shear failure. The block shear capacity is:

$$L_{net,t} = 630mm, \quad L_{net,v} = 1755mm, \quad t_1 = 40mm$$

$$F_{bs,Rk} = \max\begin{cases} F_{tk} = 1.5 \cdot L_{net,t} \cdot t_1 \cdot f_{t,0,k} \\ F_{vk} = 0.7 \cdot L_{net,v} \cdot t_1 \cdot f_{v,k} \end{cases} = \max\{ \begin{matrix} F_{tk} = 1.5 \cdot 630 \cdot 40 \cdot 19.5 = 737 \\ F_{vk} = 0.7 \cdot 1755 \cdot 40 \cdot 3.5 = 172 \end{matrix} = 737kN$$

The plug shear capacity is:

$$L_{net,t} = 630mm, \quad L_{net,v} = 1755mm, \quad t_1 = 40mm$$

$$M_{y,\text{Rk}} = 0,3 f_{\text{uk}} d^{2,6} = 0.3 \cdot 800 \cdot 15^{2,6} = 274188\text{Nmm}$$

$$f_{h,0,k} = 0.082(1 - 0.01d)\rho_k = 0.082(1 - 0.01 \cdot 15) \cdot 390 = 27.2\ \text{N/mm}^2$$

$$t_{\text{ef}} = \begin{cases} 2\sqrt{\dfrac{M_{y,\text{Rk}}}{f_{\text{h,k}}d}} \\ t_1\left[\sqrt{2 + \dfrac{M_{y,\text{Rk}}}{f_{\text{h,k}}dt_1^2}} - 1\right] \end{cases} = \begin{cases} 2\sqrt{\dfrac{274188}{27.2 \cdot 15}} = 52 \\ 40\left[\sqrt{2 + \dfrac{274188}{27.2 \cdot 15 \cdot 40^2}} - 1\right] = 22 \end{cases} = 22mm$$

$$F_{bs,Rk} = \max\begin{cases} F_{tk} = 1.5 \cdot L_{net,t} \cdot t_1 \cdot f_{t,0,k} \\ F_{vk} = 0.7 \cdot (L_{net,v}/2) \cdot (L_{net,t} + 2_{\text{ef}}) \cdot f_{v,k} \end{cases}$$

$$F_{bs,Rk} = \max\begin{cases} F_{tk} = 1.5 \cdot 630 \cdot 40 \cdot 19.5 = 737 \\ F_{vk} = 0.7 \cdot (1755/2) \cdot (630 + 2 \cdot 22) \cdot 3.5 = 1449 \end{cases} = 1449kN$$

So for the outer part the capacity is $\boldsymbol{F_{bs,Rk} = 737kN}$

The inner parts of the connection are twice as stiff as the outer parts due to the amount of shear planes [61]. This is shown in figure B.9. This means that when the load in the outer plane reaches $737kN$, the force in the inner plane is 2x as great as can be seen in figure B.10. This force does not exceed the maximum force of $1935kN$ as it is $737 \cdot 2 = 1474kN$. The capacity of the outer parts is $\boldsymbol{737kN}$

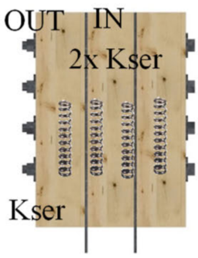


Figure B.10: Stiffness of a slotted steel plate connection with multiple plates [61]

The total capacity becomes:

$$F_{bs,Rk,tot} = 2 \cdot 737 + 3 \cdot 1935 = 7279\text{kN}$$

$$F_{bs,Rd,tot} = F_{bs,Rk,tot} \cdot \frac{k_{mod}}{\gamma_m} = 7279 \cdot \frac{0.9}{1.3} = \mathbf{5039kN}$$

In the draft for EC5 the capacity is calculated in the following way:

$$F_{\mathrm{bs,d}} = \max\left(2F_{\mathrm{v,ld}}; F_{\mathrm{t,d}}\right)$$

$$F_{\mathrm{v,l,d}} = k_{\mathrm{vef}} \cdot L_{\mathrm{con}} \cdot f_{\mathrm{v,d}}$$

$$L_{\mathrm{con}} = a_1 \cdot (n_0 - 1) + a_{3,t}$$

$$k_{\mathrm{v}} = 0,4 + 1,4\sqrt{\frac{G_{\mathrm{mean}}}{E_{\mathrm{0,\,mean}}}}$$

$$F_{\mathrm{t,Rd}} = k_{\mathrm{t}} \cdot b_{\mathrm{net\ ef}} \cdot f_{\mathrm{t,0,d}}$$

$$k_{\mathrm{t}} = 0,9 + 1,4\sqrt{\frac{G_{\mathrm{mean}}}{E_{\mathrm{0,\,mean}}}}$$

$$b_{\mathrm{net}} = (a_2 - d_n) \cdot (n_{90} - 1)$$

With the geometry of the connection being shown in figure B.11

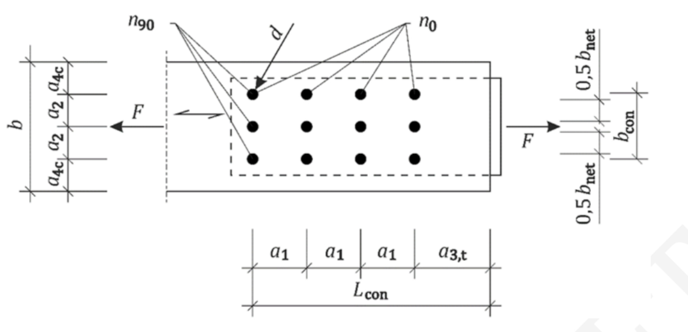


Figure B.11: Geometry of a slotted-in steel plate connection from EC5 draft

For the timber used in this connection: GL30c $G_{mean} = 650N/mm^2$ and $E_{0,mean} = 13000N/mm^2$

$$k_v = 0.4 + 1.4 \cdot \sqrt{\frac{650}{13000}} = 0.71$$

$$k_t = 0.9 + 1.4 \cdot \sqrt{\frac{650}{13000}} = 1.21$$
$$L_{\text{con}} = 0.5 \cdot \left(L_{net,v} + (n_0 - 0.5) \cdot d\right)$$
$$b_{net} = L_{net,t}$$

The design strength becomes:

$$F_{vd} = 2 \cdot F_{v,l,d} = 2 \cdot k_{\text{vef}} \cdot L_{\text{con}} \cdot f_{\text{v,d}}$$
$$F_{bs,Rd} = \max\begin{cases} F_{td} = 1.21 \cdot L_{net,t} \cdot t \cdot f_{t,0,d} \\ F_{vd} = 2 \cdot F_{v,l,d} = 2 \cdot k_{\text{vef}} \cdot L_{\text{con}} \cdot f_{\text{v,d}} \end{cases}$$

$$f_{v,d} = f_{v,k} \cdot \frac{k_{\text{mod}}}{\gamma_m} = 3.5 \cdot \frac{0.9}{1.3} = 2.42 \text{ N/mm}^2$$
$$f_{t,0,d} = k_{\text{mod}} \cdot \frac{f_{t,0,gk}}{\gamma_m} = 0.9 \cdot \frac{19.5}{1.3} = 13.5 \text{ N/mm}^2$$

The inner part can only fail on block shear and the dowels fully penetrate the elements therefore $t_{ef} = t_2$. The capacity becomes:

$$F_{bs,Rd} = \max\begin{cases} F_{td} = 1.21 \cdot 630 \cdot 105 \cdot 13.5 = 1083\text{kN} \\ F_{vd} = 2 \cdot 0.71 \cdot 1020 \cdot 105 \cdot 2.42 = 368\text{kN} \end{cases} = \mathbf{1081kN}$$

The fastener in the outer part does not fully penetrate therefore the connection can fail on block and plug shear. The block shear capacity for the outer part becomes:

(3) The effective thickness of outer timber members t_{ef} should be taken from:

$$t_{\text{ef,el}} = \begin{cases} \alpha_{\text{cl}}\, t_{\text{h,o}} & \text{if } \frac{t_{\text{h,o}}}{d} < 3 \\ \max\begin{cases} \left(1{,}17 - \frac{t_{\text{h,o}}}{18d}\right) \alpha_{\text{cl}}\, t_{\text{h,o}} \\ 0{,}35\, \alpha_{\text{cl}}\, t_{\text{h,o}} \end{cases} & \text{if } \frac{t_{\text{h,o}}}{d} > 3 \end{cases} \tag{11.46}$$

with

$\alpha_{\text{cl}} = 1$ for steel-to-timber connections where mode (f) governs;

$\alpha_{\text{cl}} = 0{,}65$ for timber-to-timber connections and all failure modes excluding mode (f) in steel-to-timber connections.

$t_{h,o} = t_1 = 40mm$

$t_{h,o}/d = 40/15 = 2.667$ Failure mode h from figure B.5 is failure mode f and this is not governing

$t_{ef,el} = 0.65 \cdot 40 = 26mm$

$$F_{bs,Rd} = \max\begin{cases} F_{td} = 1.21 \cdot 630 \cdot 26 \cdot 13.5 = 268\text{kN} \\ F_{vd} = 2 \cdot 0.71 \cdot 1020 \cdot 26 \cdot 2.42 = 92\text{kN} \end{cases} = \mathbf{268kN}$$

The plug shear failure capacity is:

$$F_{psd} = \max\begin{cases} 2 \cdot F_{v,l,d} = 2 \cdot k_{\text{vef}} \cdot L_{\text{con}} \cdot f_{\text{v,d}} \\ F_{t,d} + 2 \cdot F_{v,b,Rd} = k_{\text{t}} \cdot b_{\text{net ef}} \cdot f_{\text{t,0,d}} + 2 \cdot k_{\text{v}} \cdot L_{\text{con}} \cdot b_{\text{net}} \cdot f_{\text{v,d}} \end{cases}$$
$$F_{psd} = \max\begin{cases} 2 \cdot 0.71 \cdot 26 \cdot 1020 \cdot 2.42 = 91 \\ 1.21 \cdot 630 \cdot 26 \cdot 13.5 + 2 \cdot 0.71 \cdot 1020 \cdot 630 \cdot 2.42 = 2476 \end{cases} = \boldsymbol{2476kN}$$

Total capacity: The capacity of the outer part is the minimum value of block shear and plug shear so the capacity is $268kN$

$$F_{bs,Rd,tot} = 2 \cdot 268 + 3 \cdot 1083 = \mathbf{3786kN}$$

B.0.6. Steel parts of the connection

For the following calculations Eurocode 3 part 8 chapter 3 [56] is used to calculate the capacities.

<u>Shear resistance bolt</u>

$$F_{\mathrm{v,Rd}} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$

$$A_b = \pi \cdot \frac{d^2}{4} = \pi \cdot \frac{15^2}{4} = 176.7 \text{ mm}^2$$

$$\alpha_v = 0.6, \quad f_{ub} = 800 \text{ N/mm}^2, \quad \gamma = 1.3$$

$$F_{v,Rd} = \frac{0.6 \cdot 800 \cdot 176.7}{1.3} \cdot k_{mod} = 58.7\text{kN}$$

$$F_{v,Rd,tot} = 58.7 \cdot 10 \cdot 10 \cdot 8 = \mathbf{46975kN}$$

<u>Bearing resistance steel plate</u>

Stuikweerstand [1), 2), 3)]	$F_{\mathrm{b,Rd}} = \frac{k_1 \alpha_b f_u d t}{\gamma_{M2}}$ waarin α_b de kleinste van volgende waarden : α_d ; $\frac{f_{ub}}{f_u}$ of 1,0; — in de richting van de krachtsoverdracht: — voor eindbouten: $\alpha_d = \frac{e_1}{3d_0}$; voor binnenste bouten: $\alpha_d = \frac{p_1}{3d_0} - \frac{1}{4}$ — loodrecht op de richting van de krachtsoverdracht: — voor randbouten: k_1 is de kleinste waarde van $2{,}8\frac{e_2}{d_0} - 1{,}7$, $1{,}4\frac{p_2}{d_0} - 1{,}7$ en 2,5 — voor binnenste bouten: k_1 is de kleinste waarde van $1{,}4\frac{p_2}{d_0} - 1{,}7$ of 2,5

Figure B.12: Design resistance for individual fasteners (Part of table 3.4 from NEN-EN 1993-1-8)

$$\alpha_b = \min \begin{cases} \frac{e_1}{3 \cdot d} = \frac{120}{3 \cdot 15} = 2.67 \\ \frac{p_1}{3 \cdot d} - \frac{1}{4} = \frac{100}{3 \cdot 15} - \frac{1}{4} = 1.97 \\ \frac{fub}{fu} = \frac{800}{510} = 1.57 \\ 1 \end{cases}$$

$$k_1 = \min \begin{cases} 2.8\frac{e_2}{d} - 1.7 = 2.8\frac{25}{15} - 1.7 = 2.97 \\ 1.4\frac{p_2}{d} - 1.7 = 1.4\frac{85}{15} - 1.7 = 6.23 = 2.5 \\ 2.5 \end{cases}$$

$$F_{b,rd} = \frac{2.5 \cdot 1 \cdot 510 \cdot 15 \cdot 15}{1.3} \cdot kmod = 198.6\text{kN}$$

$$F_{b,rd,tot} = 198.6 \cdot 10 \cdot 10 \cdot 4 = \boldsymbol{79442kN}$$

Block tearing steel plate

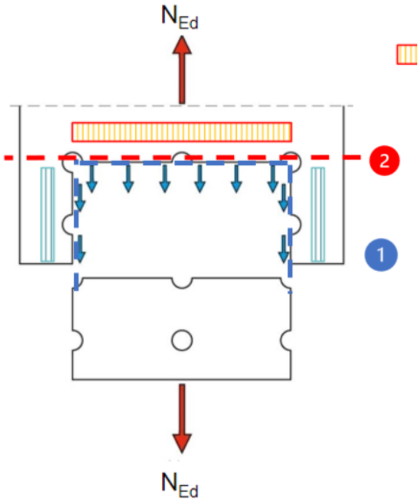


Figure B.13: Area subjected to shear or tension

$$V_{\text{eff},1,\text{Rd}} = \mathrm{f_u} \cdot \mathrm{A_{nt}}/\gamma_{\mathrm{M2}} + (1/\sqrt{3})_{\mathrm{y}} \cdot \mathrm{A_{nv}}/\gamma_{\mathrm{M0}}$$
$$A_{nv} = t_{\text{plate}} \cdot (e_1 + 9 \cdot p1) = 15 \cdot (120 + 9 \cdot 100) = 15300 \text{ mm}^2$$
$$A_{nt1} = t_{\text{plate}} \cdot (p_2 \cdot 9 - d \cdot 9) = 15 \cdot (85 \cdot 9 - 15 \cdot 9) = 9450 \text{ mm}^2$$
$$A_{nt2} = t_{\text{plate}} \cdot (p_2 \cdot 9 - d \cdot 10 + 2 \cdot e_2) = 15 \cdot (85 \cdot 9 - 15 \cdot 10 + 2 \cdot 25) = 9975 \text{ mm}^2$$
$$V_{\text{eff.rd}} = \min \begin{cases} f_u \cdot \frac{A_{nt1}}{\gamma_{M2}} + \frac{1}{\sqrt{3}} \cdot f_y \cdot \frac{A_{nv}}{\gamma_{M0}} = 510 \cdot \frac{9450}{1.3} + \frac{1}{\sqrt{3}} \cdot 355 \cdot \frac{15300}{1} = 6843.2\text{kN} \\ f_u \cdot \frac{A_{nt2}}{\gamma_{M2}} = 510 \cdot \frac{9975}{1.3} = 3913.3\text{kN} \end{cases} = 3913.3\text{kN}$$

$$V_{\text{eff. rd,tot}} = 3913.3 \cdot 4 \cdot k_{mod} = \mathbf{14088kN}$$

B.0.7. Capacity of the connection

$$F_{v,Rd} = 7369\text{kN}$$
$$\boldsymbol{F_{bs,Rd,tot,draft} = 3786\text{kN}}$$
$$\boldsymbol{F_{bs,Rd,tot,current} = 5039\text{kN}}$$
$$F_{v,Rd,tot} = 46975\text{kN}$$
$$F_{b,rd,tot} = 79442\text{kN}$$
$$V_{\text{eff},rd,tot} = 14088\text{kN}$$

B.0.8. Axial slip of the connection

Per bolt per shear plane:

$$K_{sls,v} = \frac{\rho_{\text{mean}}{}^{1.5} \cdot d}{23} \cdot 2 = \frac{430^{1.5} \cdot 15}{23} \cdot 2 = 11630.4 \text{ N/mm}$$

There are 8 shear planes and 100 bolts:

$$K_{sls,v,tot} = 11630.4 \cdot 8 \cdot 100 = 9304320 \text{ N/mm}$$

$$K_{\mathrm{u}} = \frac{2}{3} \cdot K_{\mathrm{ser}}$$
$$K_{u,tot} = 11630.4 \cdot 800 \cdot 0.6667 = 6202901 \text{ N/mm}$$

$K_{sls,v,tot}$ will be used to determine the global lateral displacement and dynamic behaviour.

B.1. Conclusion

The capacity in Mjøstårnet was most likely determined with the current Eurocode for timber [58]. The capacities calculated for the connection were 5500 kN under tension and -11500 kN under compression. When performing the calculations from the current Eurocode a capacity of 5039 kN is found for both compression and tension. The tension capacity can be 450 kN lower as the parameters d, t, a_1, a_2, a_3, a_4 were probably not assumed correctly. The compression force difference can be explained by the fact that during the design of Mjøstårnet the timber elements are expected to transfer the loads end-grain to end-grain under compression. However, in this study it is assumed that the compression force also needs to be taken up by the slotted-in steel plate connection so the compression force is not transferred end-grain to end-grain.

The capacity calculated with the current Eurocode is 5039 kN and with the draft it is 3779 kN. The capacity is reduced as the draft version uses the material dependent factors k_v and k_t as well as a reduction value for t_1. In this study the calculation method from the draft will be used to calculate the block shear capacity as it gives a lower capacity and is state of the art.

C

Exploratory study for the design of a slotted-in steel plate connection

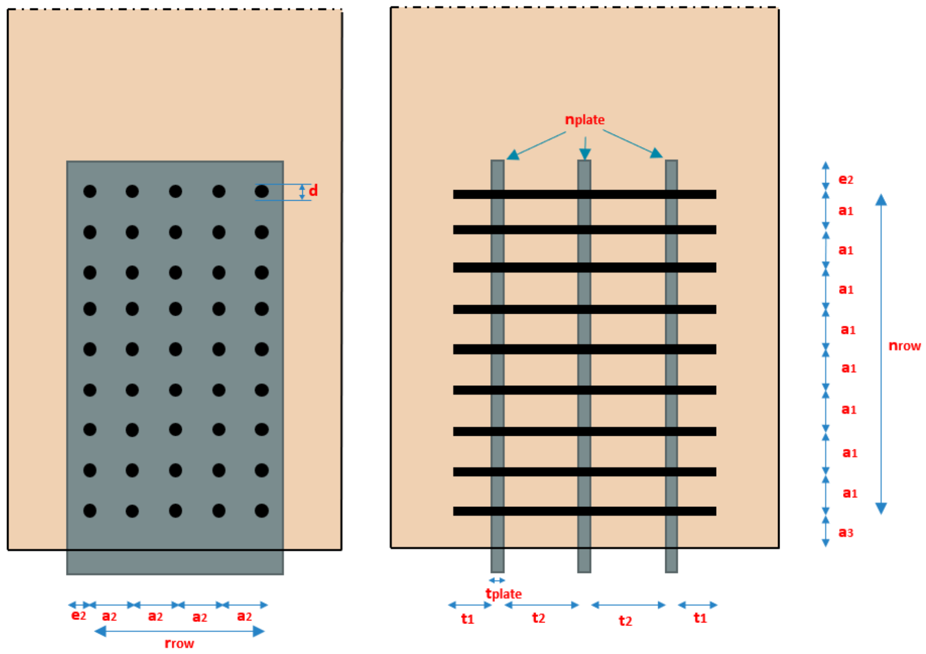


Figure C.1: Parameters that will be studied

All parameters discussed in this appendix are shown in figure C.1 in red. The parameter n_{row} will not be defined in this study but the effects will be studied. To explore the influence of different parameters on the connection two different approaches are applied. The first approach takes the connection calculated in appendix B and changes the parameter in question. In a table the capacity of the connection and the normative mechanism will be given. The first row in the table will always show the capacity for the connection design of appendix B which is 3786 kN.

The second approach is to make multiple plots per parameter where the other parameters differ as well. This is done because the most material efficient choice for a parameter depends highly on the rest of the connection design. Some parameters will be dependent on the other chosen parameters when they are not being researched. These parameters are:

- e_2: the minimum value of $1.2 * d$ is used
- a_4: the minimum value of $85\ mm$ determined by the reduced cross-section method is added to e_2
- a_2: a_4 on both sides is subtracted from the height h and then divided by the amount of bolts r_{row}
- a_3: is the minimum value of $7 * d$
- t_2: $85\ mm$ from the reduced cross-section method and t_1 will be subtracted from the width b for both sides as well as the plate thickness t_{plate} times the amount of plates. The remaining thickness of timber will be divided by the number of plates minus one to make t_2. Figure B.1 helps to clarify this explanation.

The results of both approaches will be discussed per parameter. Where first a table will be shown with the results of the first approach. Afterwards the plots of the second approach will be shown with an accompanying table where all parameters of the connection are shown. When one of the dependent parameters is defined it will also be documented in the accompanying table.

C.1. d: diameter of the bolt

It was chosen to determine d first because it gives the limits for a lot of other parameters.

Approach 1

d [mm]	Capacity [kN]	Type
15	3786	Block shear timber
10	4057	Block shear timber
8	3284	Johansen
5	1682	Johansen

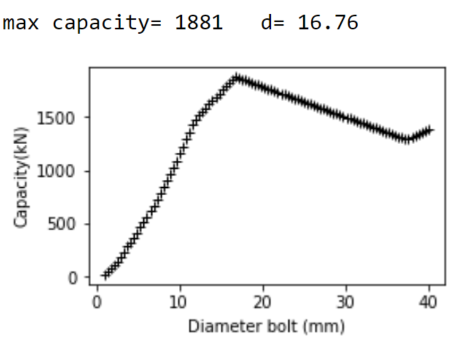
From these results we can see that if the diameter is small the connection will fail on the strength of the failures modes from the Johansen model. When the diameter is large the block shear strength of the timber becomes normative.

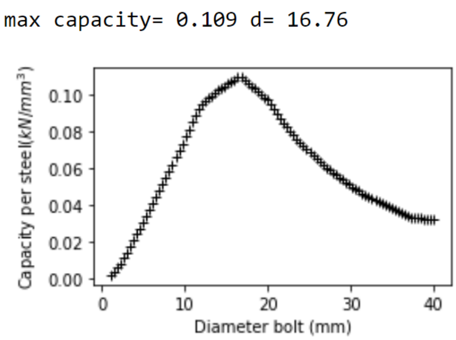
Approach 2

The minimum value of parameter a_1 is dependent on d that is why the maximum value of $100mm$ or $(4 * cos\alpha) * d$ is used for the following plots.

Plot 1:

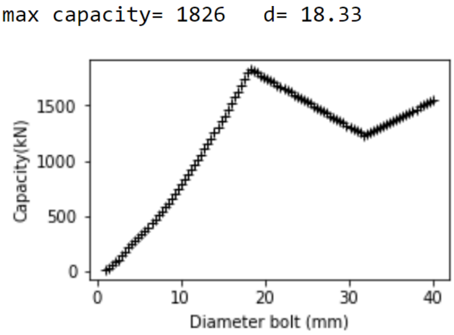
Parameters	
b	$600\ mm$
h	$600\ mm$
n_{plate}	4
t_1	$40\ mm$
n_{row}	6
r_{row}	4
t_{plate}	$15\ mm$

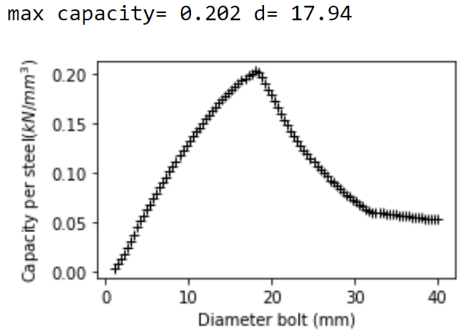




Plot 2:

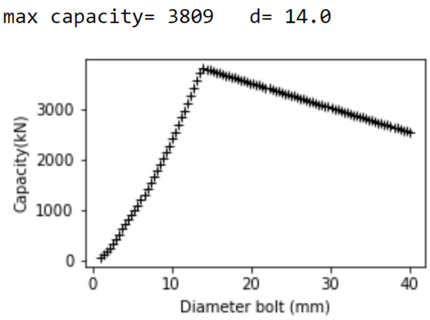
Parameters	
b	$600\ mm$
h	$600\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	6
r_{row}	6
t_{plate}	$10\ mm$

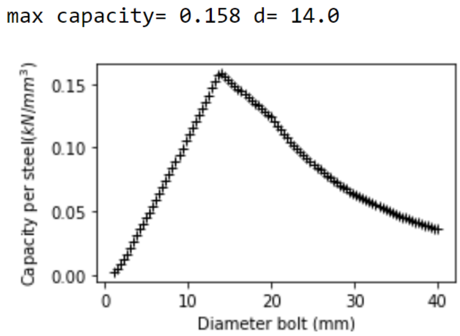




Plot 3:

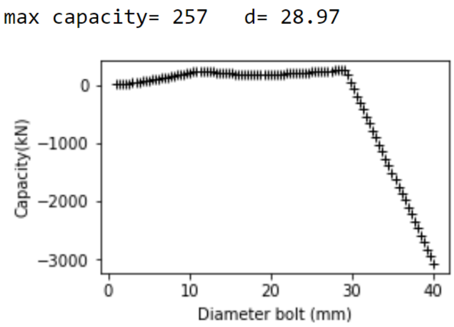
Parameters	
b	$500\ mm$
h	$1200\ mm$
n_{plate}	3
t_1	$40\ mm$
n_{row}	7
r_{row}	10
t_{plate}	$10\ mm$

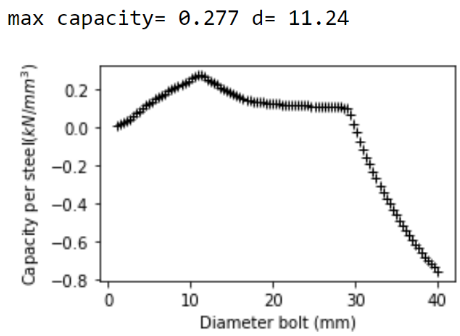




Plot 4:

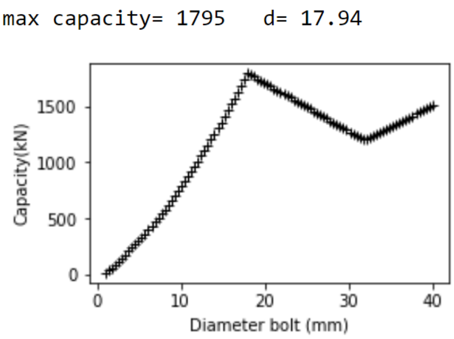
Parameters	
b	$400\ mm$
h	$300\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	3
r_{row}	3
t_{plate}	$10\ mm$

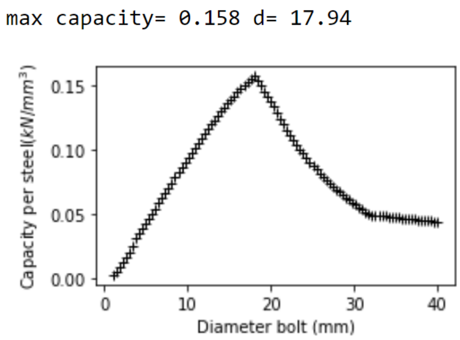




Plot 5:

Parameters	
b	$600\ mm$
h	$600\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	6
r_{row}	6
t_{plate}	$15\ mm$





The first plot shows the capacity on the y-axis and the bolt diameter on the x-axis. The second plot gives the capacity per steel usage on the y-axis and the diameter of the bolt on the x-axis. The higher the capacity per steel usage the higher the material efficiency. These plots give the most material efficient connections for values of the bolt diameter anywhere between 11 mm up until 18 mm. The average of these values is 15.5 mm with rounding up we get a bolt size of **16 *mm***.

C.2. e_2: edge distance steel plate

e_2 should have a minimal value of $1.2 * d$ and a maximum of $4t + 40mm$. These values come from EC3 and can be seen in appendix B.

Approach 1

e_2 [mm]	Capacity [kN]	Type
120	3786	Block shear timber
18.2	3786	Block shear timber

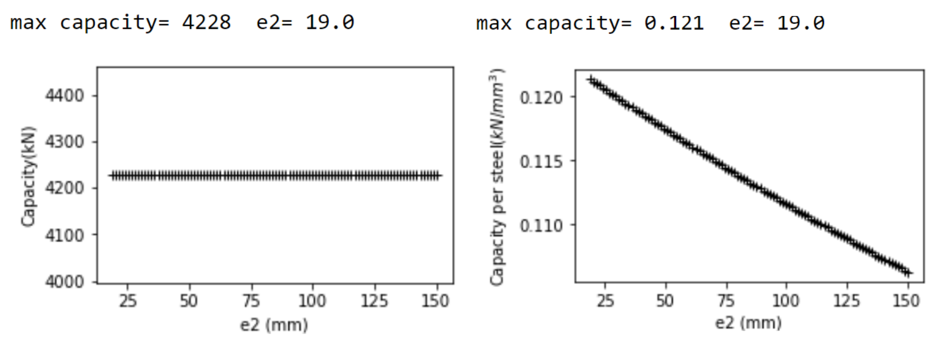
The connection from appendix B has a bolt diameter of 15 mm thus the minimum value for e_2 is 18.2 mm. e_2 seems to have no effect on the connection design.

Approach 2

The minimum value for e_2 is 19.2 mm for a connection with a bolt diameter of 16 mm thus the plots will begin at a value of 19 mm.

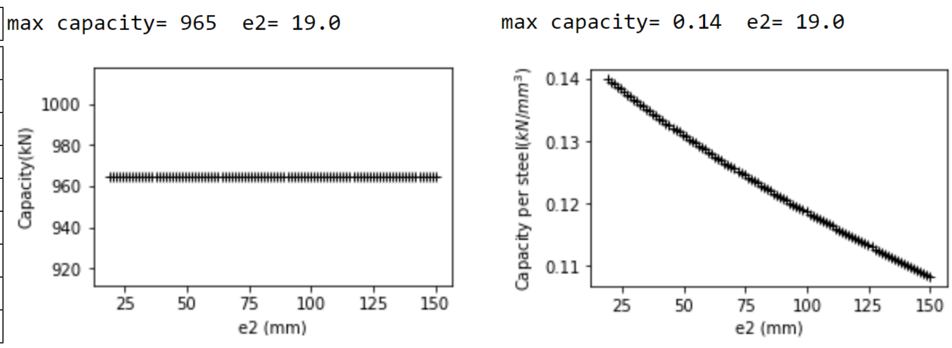
Plot 1:

Parameters	
b	$600\ mm$
h	$1000\ mm$
n_{plate}	3
t_1	$40\ mm$
n_{row}	8
r_{row}	8
t_{plate}	$15\ mm$
d	$16\ mm$
a_1	$100\ mm$



Plot 2:

Parameters	
b	$500\ mm$
h	$500\ mm$
n_{plate}	3
t_1	$40\ mm$
n_{row}	4
r_{row}	4
t_{plate}	$10\ mm$
d	$16\ mm$
a_1	$100\ mm$



From the plots it can be concluded that e_2 should be kept at a minimum value of $1.2 * d$ which is $19.2\ mm$ as this is the most material efficient. With rounding upwards a value of **25 mm** is chosen to be safe for if the steel plates and hole placements have some errors.

C.3. t_{plate}: thickness of the steel plate

Approach 1

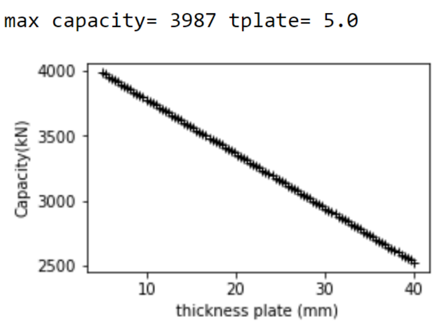
t_{plate} [mm]	Capacity [kN]	Type
15	3786	Block shear timber
10	3993	Block shear timber
8	4075	Block shear timber

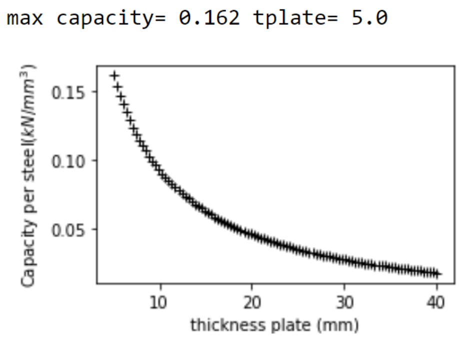
If t_{plate} increases the amount of timber in the connection will decrease slightly. If the block shear strength of the timber is normative the capacity can be increased a little by reducing the plate thickness.

Approach 2

Plot 1:

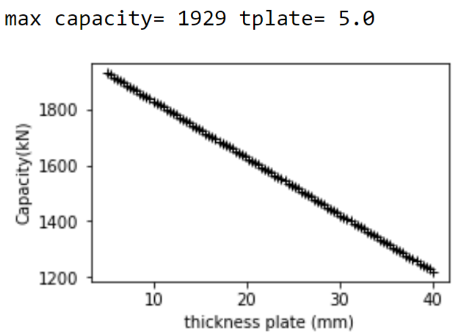
Parameters	
b	$600\ mm$
h	$1000\ mm$
n_{plate}	4
t_1	$40\ mm$
n_{row}	10
r_{row}	10
d	$16\ mm$
a_1	$100\ mm$

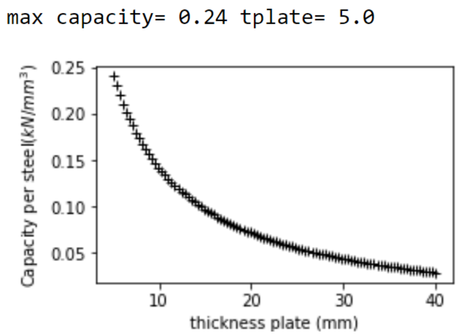




Plot 2:

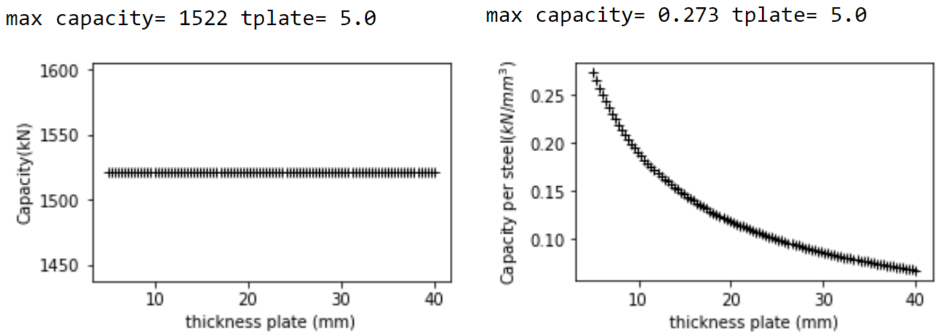
Parameters	
b	$600\ mm$
h	$600\ mm$
n_{plate}	4
t_1	$40\ mm$
n_{row}	6
r_{row}	6
d	$16\ mm$
a_1	$100\ mm$





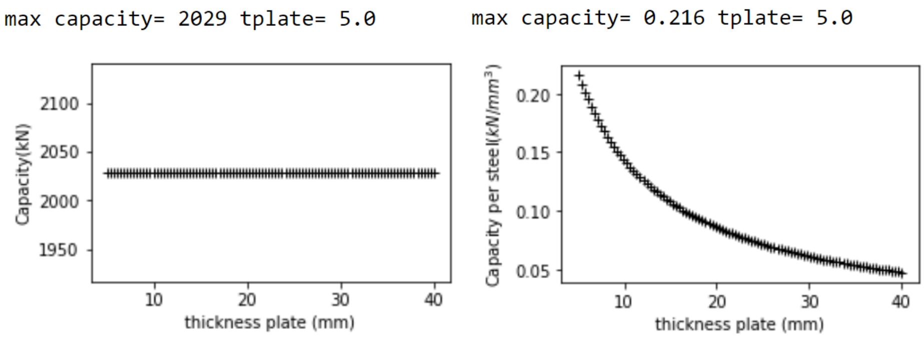
Plot 3:

Parameters	
b	$600\ mm$
h	$600\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	6
r_{row}	6
d	$16\ mm$
a_1	$100\ mm$



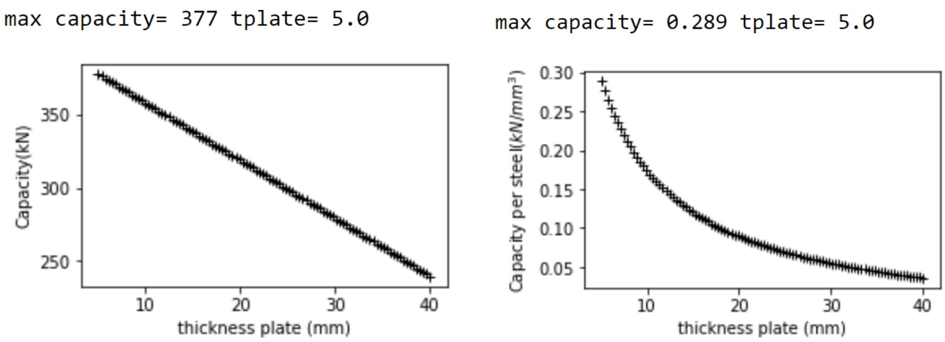
Plot 4:

Parameters	
b	$650\ mm$
h	$1000\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	6
r_{row}	8
d	$16\ mm$
a_1	$100\ mm$



Plot 5:

Parameters	
b	$400\ mm$
h	$300\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	6
r_{row}	2
d	$16\ mm$
a_1	$100\ mm$



The capacities of plot 1, 2 and 5 are all determined by the block shear strength. For plot 3 and 4 the Johansen failure modes are normative. When the thickness of timber between the plates (t_2) and on the outside of the plate(t_1) is small it is best to keep t_{plate} as small as possible so that the block shear strength of the timber becomes slightly larger. The Eurocode does not give a bottom value for the plate thickness so the connections in the reference projects are compared to find a bottom value. The plate thicknesses in the reference projects are 15 and 10 mm. It is assumed that plates smaller than 10 mm are uncommon for large connections so the bottom value of **10 mm** is chosen for the connection design.

C.4. a$_3$: edge distance of the timber

The smallest required value of parameter a_3 should be the maximum of either $80mm$, $7 * d$ and $1.2 * d$. These requirements are determined in EC3 and EC5 and are shown in appendix B.

Approach 1

a_3 [mm]	Capacity [kN]	Type
120	3786	Block shear timber
105	3786	Block shear timber

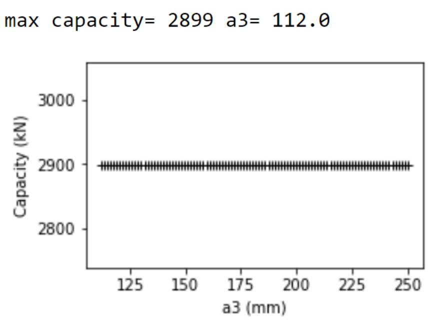
The connection of appendix B has a bolt diameter of 15 mm thus the minimum value for a_3 is 105 mm. a_3 seems to have no effect on the connection design.

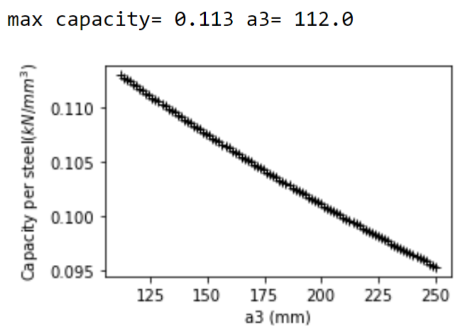
Approach 2

The plots will start at a value of 112 mm for a_3 as this is the minimum value for a connection with a bolt diameter of 16 mm.

Plot 1:

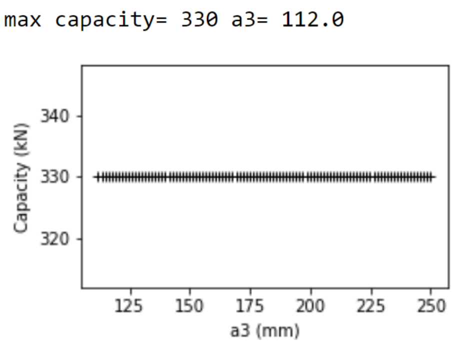
Parameters	
b	$600\ mm$
h	$1000\ mm$
n_{plate}	3
t_1	$40\ mm$
n_{row}	6
r_{row}	8
t_{plate}	$10\ mm$
d	$16\ mm$
a_1	$100\ mm$

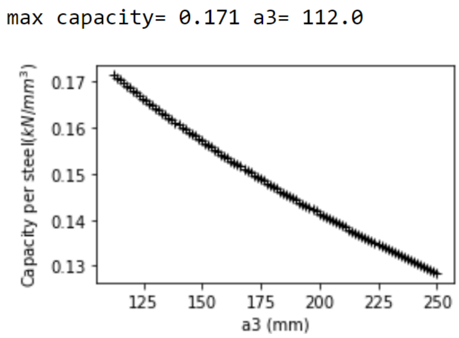




Plot 2:

Parameters	
b	$400\ mm$
h	$400\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	3
r_{row}	3
t_{plate}	$10\ mm$
d	$16\ mm$
a_1	$100\ mm$





The minimum required value of a_3 was $112mm$ for both plotted connections. The amount of steel increases when a_3 is bigger and it has no effect on the capacity of the connection. So a minimum required value of $112mm$ will be used.

C.5. n_{plate}: number of steel plates

Approach 1

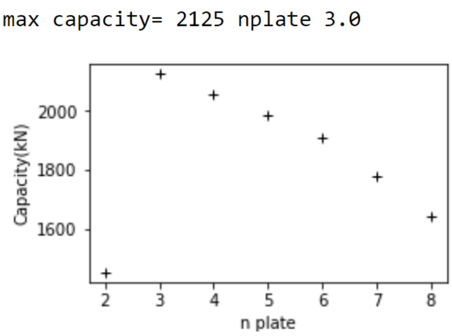
n_{plate}	Capacity [kN]	Type
4	3786	Block shear timber
3	3941	Block shear timber
2	3657	Johansen

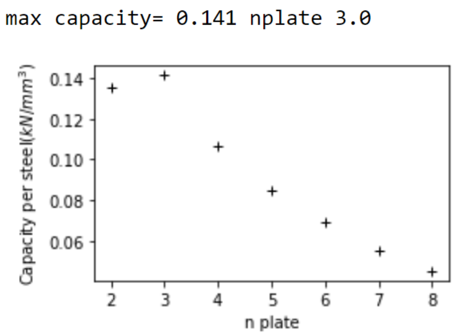
The capacity for the connection can be increased slightly by decreasing the number of plates. This is because of the block shear failure of the timber. With less plates the thickness of the timber increases a little therefore increasing the capacity.

Approach 2

Plot 1:

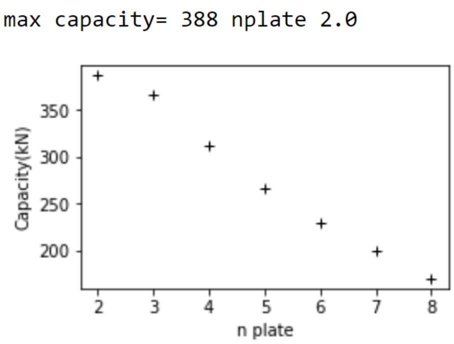
Parameters	
b	$500\ mm$
h	$800\ mm$
t_{plate}	$10\ mm$
t_1	$40\ mm$
n_{row}	7
r_{row}	5
d	$18\ mm$
a_1	$100\ mm$

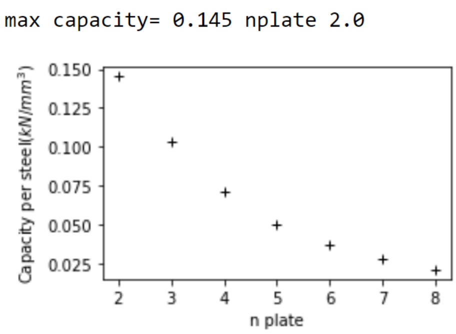




Plot 2:

Parameters	
b	$400\ mm$
h	$400\ mm$
t_{plate}	$10\ mm$
t_1	$40\ mm$
n_{row}	4
r_{row}	4
d	$18\ mm$
a_1	$100\ mm$

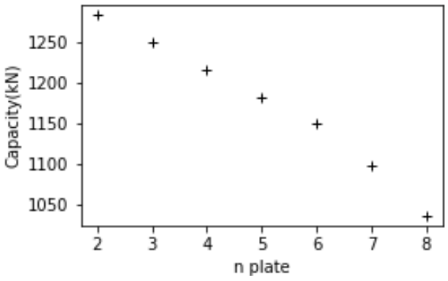




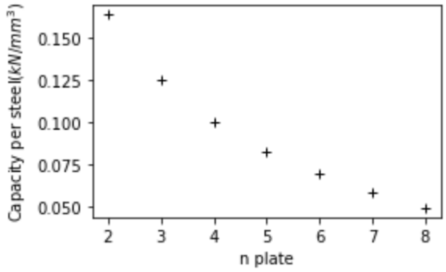
Plot 3:

Parameters	
b	$600\ mm$
h	$400\ mm$
t_{plate}	$10\ mm$
t_1	$40\ mm$
n_{row}	10
r_{row}	4
d	$16\ mm$
a_1	$100\ mm$

max capacity= 1284 nplate 2.0



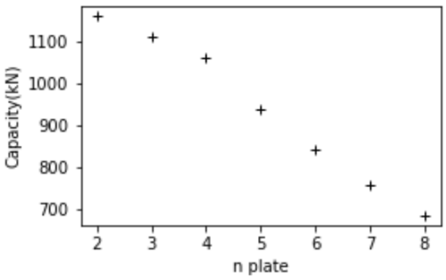
max capacity= 0.164 nplate 2.0



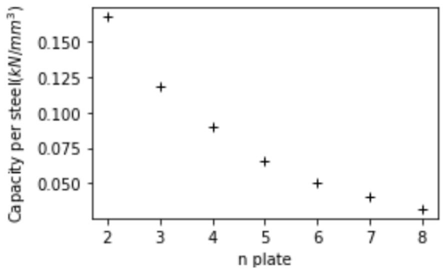
Plot 4:

Parameters	
b	$450\ mm$
h	$600\ mm$
t_{plate}	$10\ mm$
t_1	$40\ mm$
n_{row}	6
r_{row}	6
d	$16\ mm$
a_1	$100\ mm$

max capacity= 1159 nplate 2.0



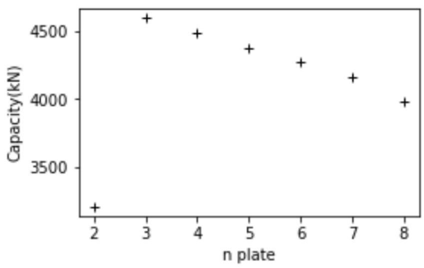
max capacity= 0.168 nplate 2.0



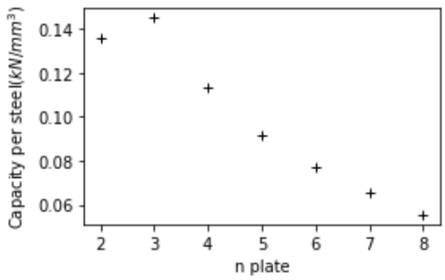
Plot 5:

Parameters	
b	$650\ mm$
h	$1000\ mm$
t_{plate}	$10\ mm$
t_1	$40\ mm$
n_{row}	10
r_{row}	8
d	$16\ mm$
a_1	$100\ mm$

max capacity= 4600 nplate 3.0



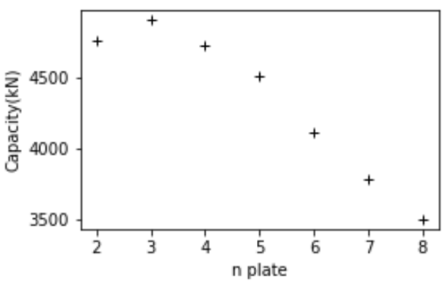
max capacity= 0.145 nplate 3.0



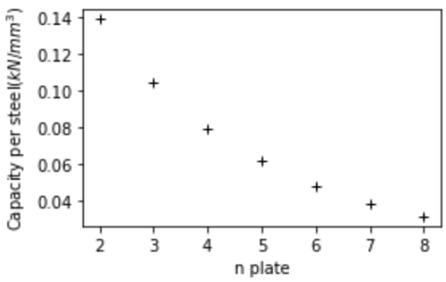
Plot 6:

Parameters	
b	$500\ mm$
h	$1500\ mm$
t_{plate}	$10\ mm$
t_1	$40\ mm$
n_{row}	10
r_{row}	10
d	$18\ mm$
a_1	$100\ mm$

max capacity= 4907 nplate 3.0



max capacity= 0.139 nplate 2.0



In plot 6 three plates gives the highest capacity but the most material efficient design is with two plates. When the connection has more than three plates the block shear strength will become normative and the capacity will decrease since the amount of timber decreases due to the added plate. In plot 2, 3 and 4 the block shear strength is already determining with two steel plates therefore the capacity will only decrease when more steel plates are added. In plot 1 and 5 again like in plot 6 the maximum capacity is for three plates but the most material efficient design is with two plates. For the researched elements with a width or height smaller than 500 mm two plates gives the highest capacity. In all three reference projects there are a lot of elements with either a width or height smaller than 500 mm. Hence, it is expected that in the researched parametric models there will also be large numbers of elements with a size smaller than 500 mm. This together with the fact that for some connections the most material efficient connection design is with two plates while the maximum capacity is with three as shown in approach 2, is the reason why **two** steel plates will be used in the parametric model.

C.6. t_1 & t_2: thickness of the timber parts

t_1 is the thickness of the timber outside of the steel plates, t_2 is the thickness of the timber between the steel plates. These parameters are both dependent on the width of the timber. If t_1 gets larger t_2 get smaller.

Approach 1

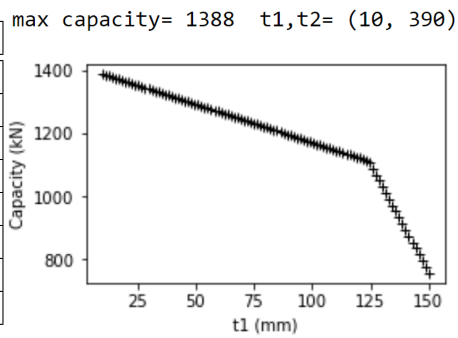
t_1 [mm]	t_2 [mm]	Capacity [kN]	Type
40	105	3779	Block shear timber
60	91.7	3642	Block shear timber
20	118	3930	Block shear timber

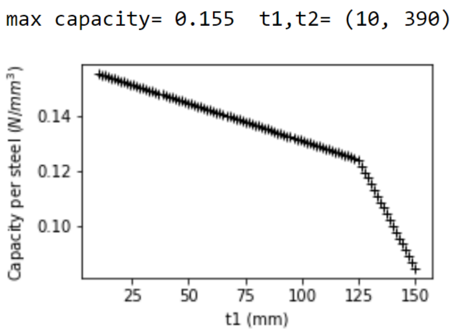
For the connection from appendix B the capacity increases when t_1 decreases and t_2 increases. This occurs when the block shear strength of the timber is normative for the capacity.

Approach 2

Plot 1:

Parameters	
b	$600\ mm$
h	$600\ mm$
n_{plate}	2
t_{plate}	$10\ mm$
n_{row}	10
r_{row}	10
d	$16\ mm$
a_1	$100\ mm$

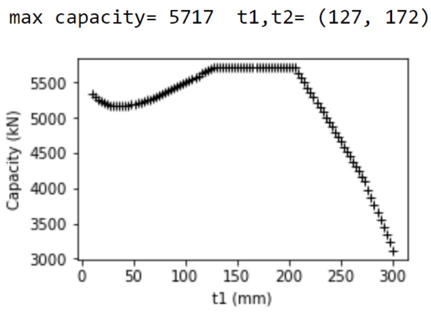


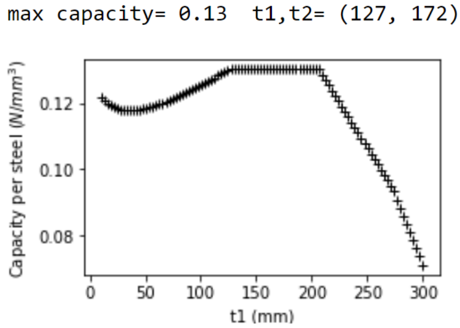


t_1 [mm]	0	50	100	150
t_2 [mm]	400	300	200	100

Plot 2:

Parameters	
b	$1000\ mm$
h	$1200\ mm$
n_{plate}	4
t_{plate}	$15\ mm$
n_{row}	6
r_{row}	10
d	$15\ mm$
a_1	$100\ mm$

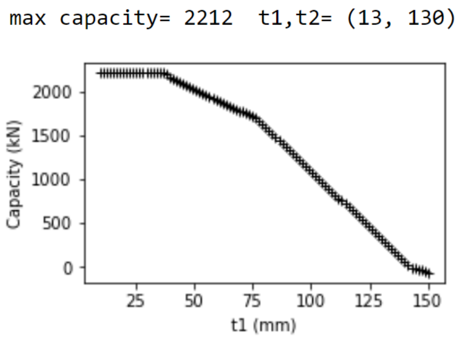


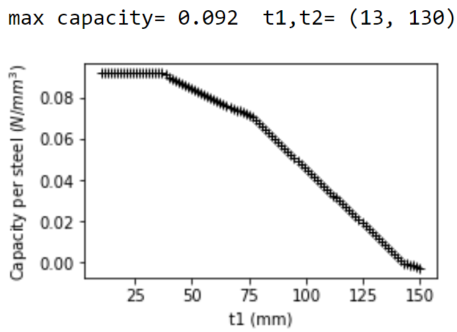


t_1 [mm]	0	50	100	150	200	250	300
t_2 [mm]	257	223	190	157	123	90	57

Plot 3:

Parameters	
b	$500\ mm$
h	$800\ mm$
n_{plate}	3
t_{plate}	$15\ mm$
n_{row}	8
r_{row}	5
d	$15\ mm$
a_1	$100\ mm$

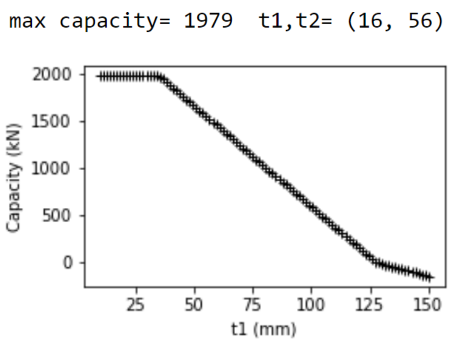


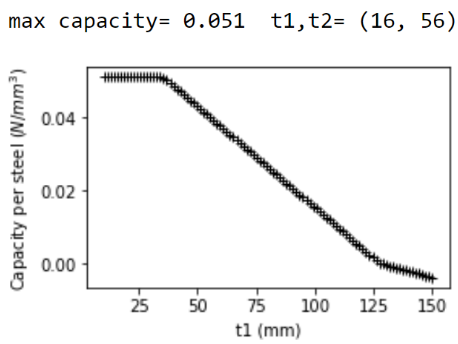


t_1 [mm]	0	50	100	150
t_2 [mm]	142.5	92.5	42.5	-7.5

Plot 4:

Parameters	
b	$500\ mm$
h	$800\ mm$
n_{plate}	5
t_{plate}	$15\ mm$
n_{row}	8
r_{row}	5
d	$15\ mm$
a_1	$100\ mm$

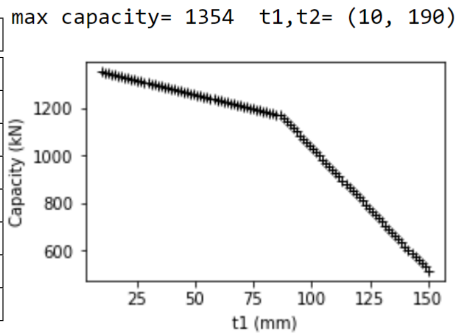


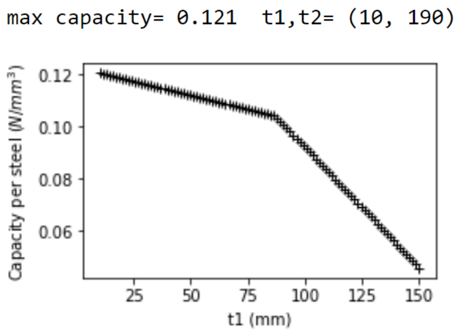


t_1 [mm]	0	50	100	150
t_2 [mm]	63.8	38.8	13.8	-11.2

Plot 5:

Parameters	
b	$600\ mm$
h	$500\ mm$
n_{plate}	3
t_{plate}	$10\ mm$
n_{row}	8
r_{row}	5
d	$18\ mm$
a_1	$90\ mm$

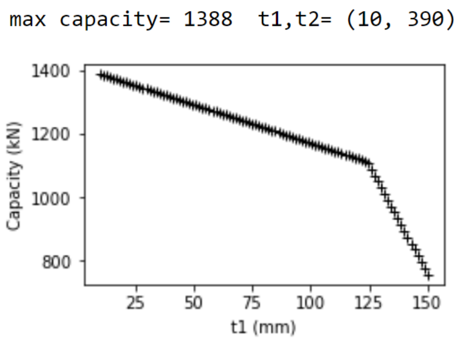


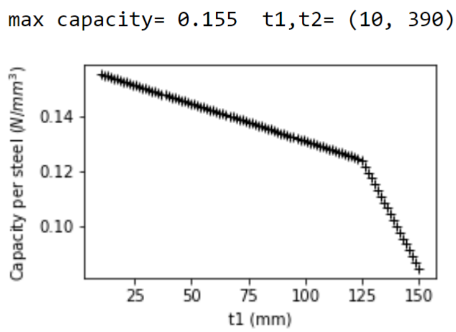


t_1 [mm]	0	50	100	150
t_2 [mm]	200	150	100	50

Plot 6:

Parameters	
b	$600\ mm$
h	$500\ mm$
n_{plate}	2
t_{plate}	$10\ mm$
n_{row}	8
r_{row}	5
d	$18\ mm$
a_1	$90\ mm$





t_1 [mm]	0	50	100	150
t_2 [mm]	410	310	210	110

Changing t_1 and t_2 adds no steel to the connection since you are only changing the location of the steel plates that is also why the graphs for capacity and capacity per steel are equal. When the block shear strength of the timber is normative for the capacity, the smallest value t_1 gives the highest capacity. This can be explained by the way of calculating the capacity. The effective thickness of the inner parts between the steel plates is always t_2. For the outer parts the effective thickness is $t_{ef} = t_1 * 0.65$ when brittle failure of the timber is normative and not the ductile failure of the bolt. For most of the connections the block shear strength is normative and this is a brittle failure. This means that the effective thickness of the entire timber element can be increased by increasing t_2 and decreased by increasing t_1 as t_1 is reduced with brittle failure. Excluding plot 2, the capacities of the connections are the highest for a t_1 of 15.6 or smaller. Only in plot 2 the capacity is higher for a larger t_1 this is because this element has a width of 1000 mm. The maximum element width in this study is 650 mm thus the connection design of plot 2 will not be used. There is no bottom value for t_1 and due to the calculation method having the smallest possible t_1 seems logical. However, t_1 must have a certain length to transfer the forces to the outer timber part. In both Treet and Mjøstårnet a t_1 of 40 mm is used. For that reason it is assumed that a value of 40 mm is an appropriate value for t_1. So a t_1 of **40** mm is used.

C.7. a_1: Distance between bolts in direction of the grain

The minimum value of a_1 the largest value of $1.2 * d$ or $5 * d$. The maximum value is the smallest of $14 * t_{plate}$ or $200mm$.

Approach 1

a_1 [mm]	Capacity [kN]	Type
100	3786	Block shear timber
75	3786	Block shear timber

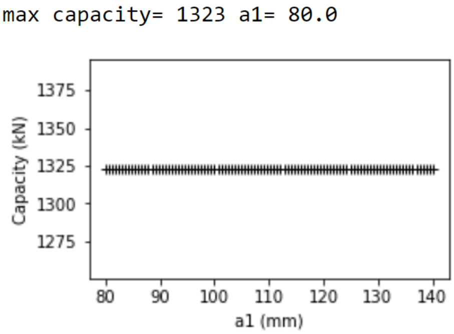
The minimum value for the connection from appendix B is 75 mm as the bolt diameter is 15 mm. The capacity does not change when changing a_1.

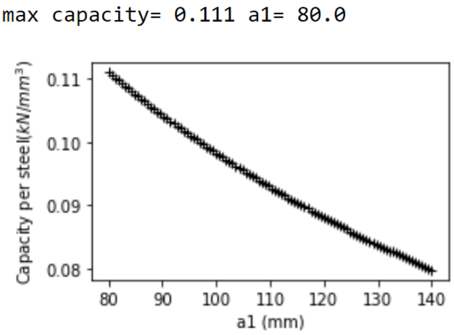
Approach 2

With a bolt diameter of 16 mm and plate thickness of 10 mm the range for a_1 is 80-140 mm.

Plot 1:

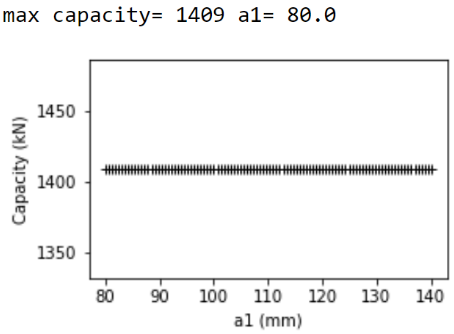
Parameters	
b	$600\ mm$
h	$600\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	10
r_{row}	6
d	$16\ mm$
t_{plate}	$10\ mm$

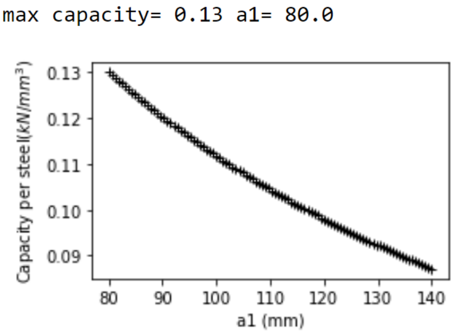




Plot 2:

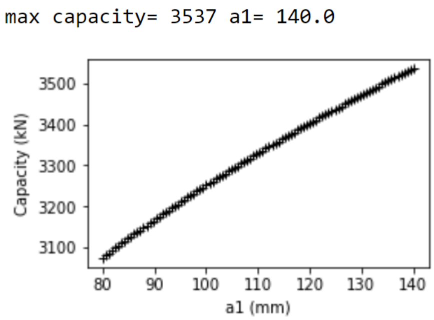
Parameters	
b	$400\ mm$
h	$800\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	8
r_{row}	8
d	$16\ mm$
t_{plate}	$10\ mm$

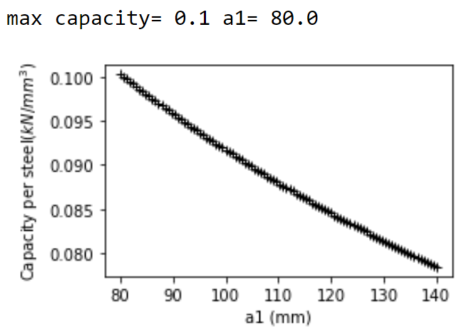




Plot 3:

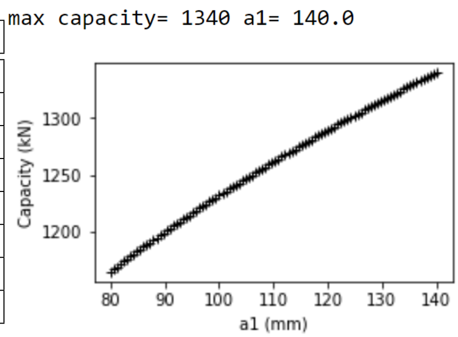
Parameters	
b	$650\ mm$
h	$1500\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	10
r_{row}	10
d	$16\ mm$
t_{plate}	$10\ mm$

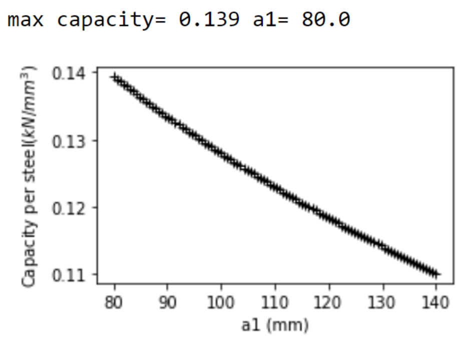




Plot 4:

Parameters	
b	$500\ mm$
h	$800\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	6
r_{row}	6
d	$16\ mm$
t_{plate}	$10\ mm$





The connections in plot 1 and 2 fail on block shear of the timber and the connections in plot 3 and 4 on the Johansen model. This means that a_1 has no effect on the capacity when block shear occurs. When the capacity of the Johansen model is normative increasing a_1 can increase the capacity slightly. However, the capacity per steel usage is always the largest with a minimum value of a_1. Therefore, a minimum value of $5 * d$ is chosen which is **80 $\boldsymbol{mm}$** for $d = 16mm$.

C.8. r_{row} & a_2: number or rows perpendicular to the grain and distance between the bolts perpendicular to the grain

The number of rows of bolts r_{row} determines the distance between the bolts a_2 since the height of the beam is not a parameter. The minimum value of a_2 is $4 * d$ or $2.4 * d$ the maximum value is $14 * t_{plate}$ or $200mm$.

Approach 1

r_row	a_2 $[mm]$	Capacity [kN]	Type
10	85	3786	Block shear timber
12	70	3636	Block shear timber
8	109	3955	Block shear timber

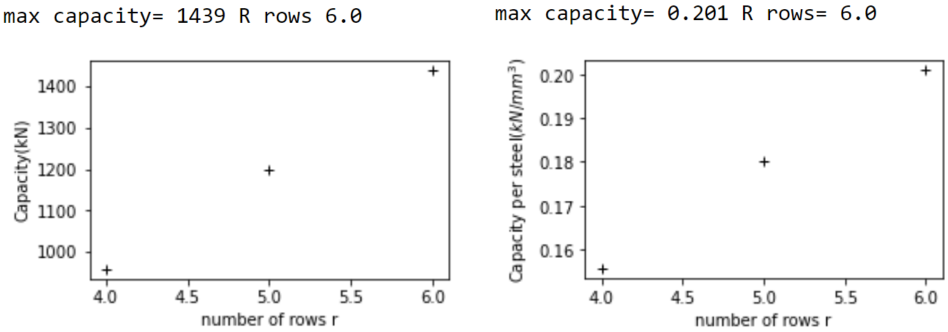
The minimum value for a_2 for the connection calculated in appendix B is 60 mm as the bolt diameter is 15 mm. Decreasing the number of rows can increase the capacity for this connection. This is caused by the block shear being normative. When calculating the block shear strength of the timber the tensile failure resistance determines the capacity for this connection. The length of the head tensile plane is b_{net} and this is calculated with the formula $b_{\text{net}} = (a_2 - d_n) * (r_{row} - 1)$. So decreasing the rows of bolts will increase the area of the timber that can resist the tensile stress.

Approach 2

For a connection with a bolt thickness of 16 mm and a plate thickness of 10 mm we get a range for a_2 of 64 and 140 mm.

Plot 1:

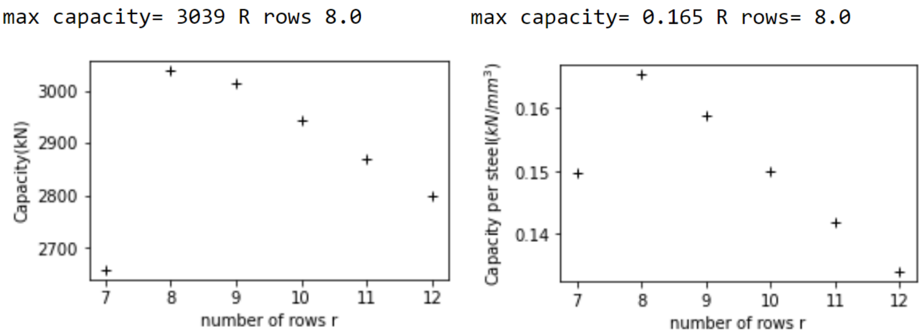
Parameters	
b	$600\ mm$
h	$600\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	6
d	$16\ mm$
t_{plate}	$10\ mm$
a_1	$80\ mm$



r_{row}	4	5	6
a_2 [mm]	130	98	78

Plot 2:

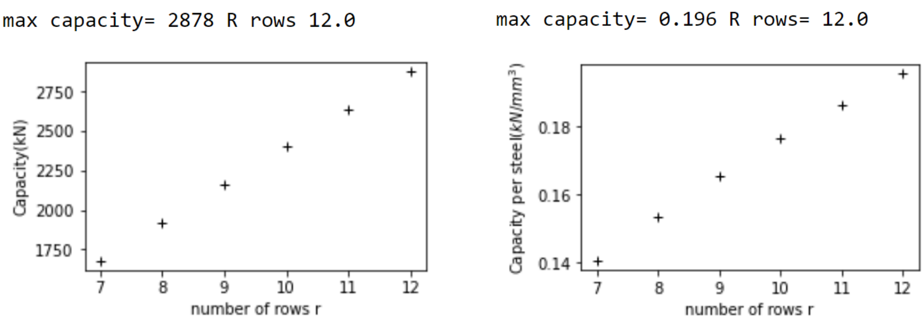
Parameters	
b	$500\ mm$
h	$1000\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	10
d	$16\ mm$
t_{plate}	$10\ mm$
a_1	$80\ mm$



r_{row}	7	8	9	10	11	12
a_2 [mm]	132	113	99	88	79	72

Plot 3:

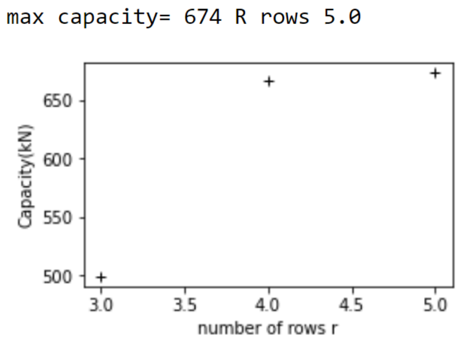
Parameters	
b	$650\ mm$
h	$1000\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	6
d	$16\ mm$
t_{plate}	$10\ mm$
a_1	$80\ mm$

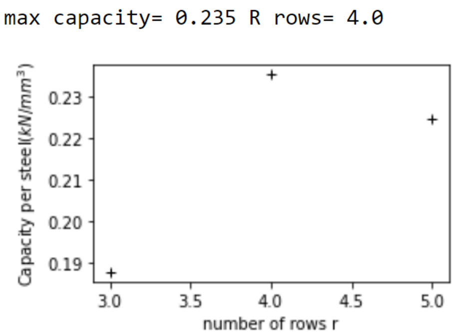


r_{row}	7	8	9	10	11	12
a_2 [mm]	132	113	99	88	79	72

Plot 4:

Parameters	
b	$400\ mm$
h	$500\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	4
d	$16\ mm$
t_{plate}	$10\ mm$
a_1	$80\ mm$

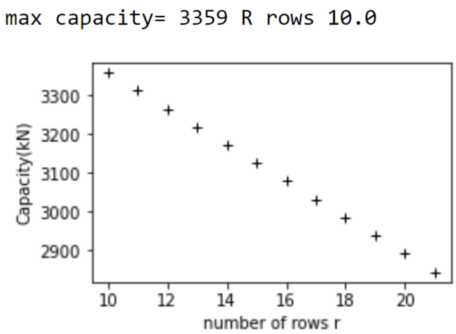


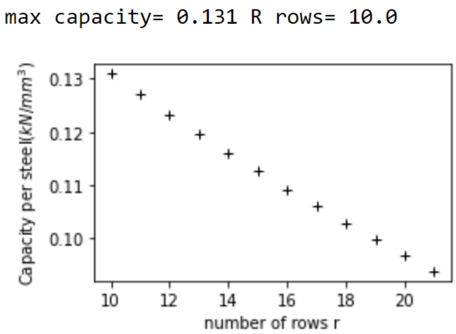


r_{row}	3	4	5
a_2 [mm]	145	97	73

Plot 5:

Parameters	
b	$400\ mm$
h	$1500\ mm$
n_{plate}	2
t_1	$40\ mm$
n_{row}	10
d	$16\ mm$
t_{plate}	$10\ mm$
a_1	$80\ mm$





r_{row}	10	11	12	13	14	15	16	17	18	19	20	21
a_2 [mm]	144	130	118	108	100	92	86	81	76	72	68	65

Plot 1, 3 and 4 show that the capacity of the connection increases when the number of rows increases. The normative failure mechanism in these connections is the Johansen model. In plot 5 the capacity decreases when rows are added. The capacities of these connections are determined by the block shear strength of the timber. Adding more rows will decrease the area of timber that can resist the tensile strength. In plot 2 the capacity first increases as the Johansen model is normative for the connection with 7 rows. Beyond this point the block shear capacity is normative. In the parametric model some of the connection designs will be determined by the block shear and others by the Johansen model. Consequently it is hard to define the best value for a_2 and r_{row}. Therefore, it is chosen to make the limits for the value of a_2 leading. The minimum value is $64mm$ with a bolt diameter of $16mm$ and the maximum is $140mm$ with a plate thickness of $10mm$. The number of rows in the connection design will be determined by dividing the available space for the bolts by $100mm$ and then round the result to give the number of rows. In this way the value for a_2 will stay within the limits but it will differ per connection design.

C.9. n_{row}: number of rows in direction of the grain

The number of bolts in the direction of the grain will not be defined in this exploratory study. This parameter will be used to be able to increase the capacity of the connection without increasing element size of the timber.

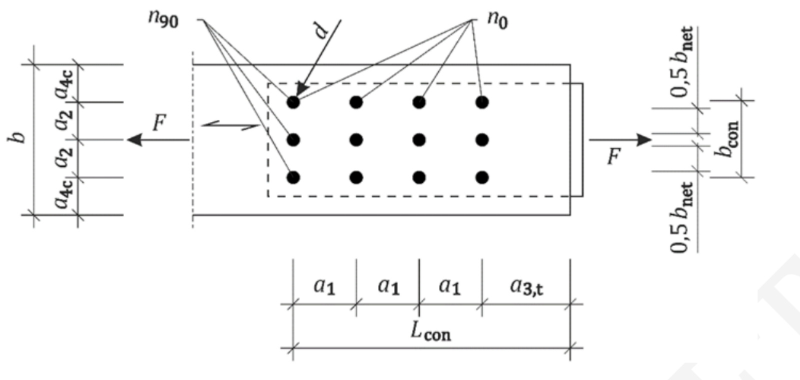
Approach 1

n_{row}	Capacity [kN]	Type
10	3786	Block shear timber
8	3786	Block shear timber
6	3786	Block shear timber
4	3231	Johansen
30	3830	Block shear timber

For this connection the capacity stays the same for 10, 8 and 6 rows. This is because the tensile capacity F_{td} of the block shear strength of the timber is normative. For 4 rows of bolts the capacity decreases and the Johansen failure mechanisms become normative. With 30 rows of bolts the capacity increases as the capacity of the side shear planes F_{vd} becomes larger than that of the tensile head. The block shear strength of timber is determined with the following formula:

$$F_{bs,Rd} = \max \begin{cases} F_{td} = k_t * b_{net} * t * f_{t,0,d} \\ F_{vd} = 2 * k_v * L_{con} * t * f_{v,d} \end{cases}$$

Where:

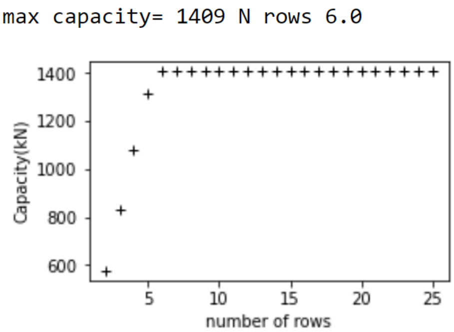


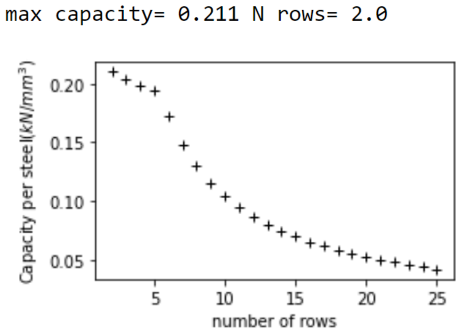
If the connection is determined by the block shear increasing the rows of bolts does not increase the connection capacity up until the point the block shear is determined by the side shear planes F_{vd} of the connection.

Approach 2

Plot 1:

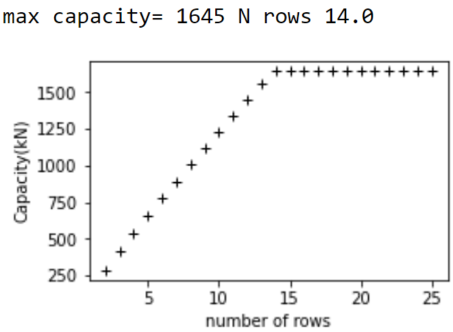
Parameters	
b	$400\ mm$
h	$800\ mm$
n_{plate}	2
t_1	$40\ mm$
r_{row}	8
t_{plate}	$10\ mm$
d	$16\ mm$
a_1	$80\ mm$

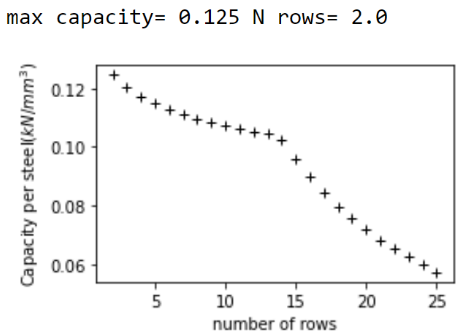




Plot 2:

Parameters	
b	$650\ mm$
h	$650\ mm$
n_{plate}	2
t_1	$40\ mm$
r_{row}	4
t_{plate}	$10\ mm$
d	$16\ mm$
a_1	$80\ mm$





When the block shear strength becomes normative for the connection the capacity will not increase when the rows of bolts n_{row} increase. In plot 1 this happens at six rows and in plot 2 at 14 rows. Adding bolts can however increase the stiffness of the connection.

D
Wind

The wind load is determined with NEN-EN 1991-1-4. It is assumed that the building is located in Rotterdam has a flat roof and square corners.

D.1. Wind Load

The wind load is determined with NEN-EN 1991-1-4. It is assumed that the building is located in Rotterdam has a flat roof and square corners. The building height is 68 meters and the plot size of the building is either 27.2 x 27.2 meters or 27.2 x 40.8 meters.

The wind load is determined with the following formula:

$$Q_w = c_s c_d \cdot c_f \cdot q_p\,(z_e) \tag{D.1}$$

$$c_s c_d = 1 \tag{D.2}$$

$$z_{\mathrm{s}} = 0.6 \cdot h \geq z_{\min} = 0.6 \cdot 68 = 40.8 \geq 7 \tag{D.3}$$

$$c_f = c_{pe} = +0.8 - -0.7 = 1.5 \tag{D.4}$$

This value for c_{pe} is determined with table NB.6 – 7.1 from NEN-EN 1991-1-4+A1+C2:2011/NB:2019+ C1:2020.

For building height $\sim 68\ \mathrm{m}$ urban area II

$$q_p\,(z_e) = 1.33 \tag{D.5}$$

This value for q_p is determined with table NB.5 from NEN-EN 1991-1-4+A1+C2:2011/NB:2019+C1:2020.

$$Q_w = 1 \cdot 1.5 \cdot 1.33 = 2.0\ kN/m^2 \tag{D.6}$$

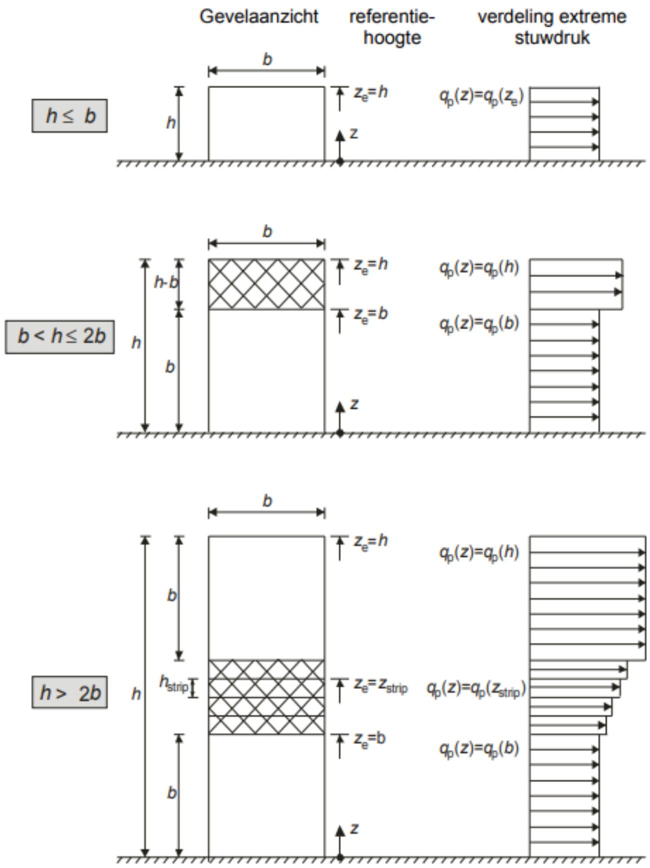


Figure D.1: Reference height, z_e, depending on h and b, and corresponding velocity pressure profile [55]

The velocity pressure q_p depends on the height and width of the building as shown below in figure D.1.

The building with plot size 27.2 x 27.2 meters will have three wind loads:

$$q_p\,(0 - 27.2) = 1Q_w = 1 \cdot 1.5 \cdot 1 = 1.5\ kN/m^2$$

$$q_p\,(54.4 - 68) = 1.33Q_w = 1 \cdot 1.5 \cdot 1.33 = 2\ kN/m^2$$

For the area in the middle of the building from 27.2 meters to 54.4 meters high the average of the two wind forces will be applied:

$$Q_w = 1.75\ kN/m^2$$

The building with plot size 27.2 x 40.8 meters will have the same three wind loads as mentioned above on the long facade of 40.8 meters. The short facade of 27.2 meters will get the following two loads:

$$q_p\,(0 - 40.8) = 1Q_w = 1 \cdot 1.5 \cdot 1.14 = 1.71\ kN/m^2$$

$$q_p\,(40.8 - 68) = 1.33Q_w = 1 \cdot 1.5 \cdot 1.33 = 2\ kN/m^2$$

D.2. Natural frequency caused by along-wind

Oosterhout (1996) suggests the following formula to calculate the natural frequency of timber buildings [78].

$$n = f(\alpha h) \cdot \sqrt{\frac{q}{m} \cdot \frac{h}{u_{\max}}} \tag{D.7}$$

Where:

- m= total mass in the building (kg)
- q= uniformly distributed wind load (N/m)
- h= height of the building (m)
- u_{max}= maximum global displacement (m)
- $f(\alpha h)$ is a function that depends on the model behaviour of the structure. $f(\alpha h)$= 0.187 is used as this is the average for bending and shear deformation[78]

D.3. Along-wind acceleration

The along-wind acceleration is calculated with annex C of NEN-EN 1991-1-4+A1+C2:2011 as this is the method that should be used according to the national annex NEN-EN 1991-1-4+A1+C2:2011/NB:2019+ C1:2020

The along wind acceleration can be calculated using the following formula:

$$a_{\max}(y,z) = \sigma_{a,x}(y,z) \cdot k_p \tag{D.8}$$

The following constants are used to calculate the acceleration:

- c_f =1.5
- ρ= 1.25 kg/m^3
- Ky = 1
- Kz= 3/2
- z_{min} = 7 m
- z_{max} = 200 m
- z_0 = 0.5 m
- $z_{0,II}$ = 0.05 m
- k_l = 1
- c_0 = 1
- c_{season} = 1
- c_{dir} = 1

- v_b = 27 m/s for Rotterdam
- δ_s = 0.12 for timber bridges
- L_t = 300 m
- z_t = 200 m
- G_y = 1/2
- G_z = 3/8
- c_y & c_z= 11.5
- ξ = 1.9 damping from Treet and used in Mjøstårnet

The following equations are used to calculate the acceleration:

$$\sigma_{\mathrm{a,x}}(y,z) = c_{\mathrm{f}} \cdot \rho \cdot I_{\mathrm{v}}\,(z_{\mathrm{s}}) \cdot v_{\mathrm{m}}^2\,(z_{\mathrm{s}}) \cdot R \cdot \frac{K_{\mathrm{y}} \cdot K_{\mathrm{z}} \cdot \Phi(y,z)}{\mu_{\mathrm{ref}} \cdot \Phi_{\max}} \tag{D.9}$$

$$I_v(z_s) = \frac{\sigma_v}{v_m(z_s)} = \frac{k_r}{c_0(z_s) \cdot \ln\,(z_s/z_0)} \tag{D.10}$$

$$v_{\mathrm{m}}(zs) = c_{\mathrm{r}}(zs) \cdot c_0(zs) \cdot v_b \tag{D.11}$$

$$c_r(z) = k_r \cdot \ln\left(\frac{z}{z_0}\right) \tag{D.12}$$

$$k_r = 0,19 \cdot \left(\frac{z_0}{z_{0,11}}\right)^{0,07} \tag{D.13}$$

$$v_{\mathrm{b}} = c_{\mathrm{dir}} \cdot c_{\mathrm{season}} \cdot v_{\mathrm{b,0}} \tag{D.14}$$

$$R^2 = \frac{\pi^2}{2 \cdot \delta} \cdot S_{\mathrm{L}}\left(z_{\mathrm{s}}, n_{1,x}\right) \cdot K_{\mathrm{s}}\left(n_{1,x}\right) \tag{D.15}$$

$$\delta = \delta_{\mathrm{s}} + \delta_{\mathrm{a}} + \delta_{\mathrm{d}} \tag{D.16}$$

$$\delta_{\mathrm{a}} = \frac{c_{\mathrm{f}} \cdot \rho \cdot v_{\mathrm{m}}\,(z_{\mathrm{s}})}{2 \cdot n_1 \cdot \mu_{\mathrm{e}}} \tag{D.17}$$

$$S_L(z,n) = \frac{n \cdot S_v(z,n)}{\sigma_v^2} = \frac{6,8 \cdot f_L(z,n)}{\left(1 + 10,2 \cdot f_L(z,n)\right)^{5/3}} \tag{D.18}$$

$$f_L(z_s,n) = \frac{n \cdot L(z_s)}{v_m(z)} \tag{D.19}$$

$$L(z_s) = L_t \cdot \left(\frac{z_s}{z_t}\right)^{\alpha} \tag{D.20}$$

$$\alpha = 0,67 + 0,05\ln(z_0) \tag{D.21}$$

$$K_s(n) = \frac{1}{1 + \sqrt{\left(G_y \cdot \phi_y\right)^2 + \left(G_z \cdot \phi_z\right)^2 + \left(\frac{2}{\pi} \cdot G_y \cdot \phi_y \cdot G_z \cdot \phi_z\right)^2}}$$
$$\phi_y = \frac{c_y \cdot b \cdot n}{v_m\,(z_s)}; \phi_z = \frac{c_z \cdot h \cdot n}{v_m\,(z_s)} \tag{D.22}$$

$$\phi(y,z) = \frac{z}{h}^{\xi} \tag{D.23}$$

Where ξ= 1.9 this is the damping determined for Treet and used for Mjostarnet.

$$\phi(max) = \frac{z}{h}^{\xi} \tag{D.24}$$

$$k_p = max(\sqrt{2 \cdot \ln(v \cdot T)} + \frac{0.6}{\sqrt{2 \cdot \ln(v \cdot T)}}, 3) \tag{D.25}$$

$$v = n_{1,x} \tag{D.26}$$

E

Timber element checks

E.1. ULS

E.1.1. Axial stress

The following formulas come from NEN-EN 1995-1-1+C1+A1:2011.

$\gamma_m = 1.25$ for laminated timber elements

$k_{mod} = 0.9$ for short loads on laminated timber elements

$$f_{c,0,d} = k_{\text{mod}} \cdot \frac{f_{c,0,gk}}{\gamma_m} \tag{E.1}$$

$$f_{t,0,d} = k_{\text{mod}} \cdot \frac{f_{t,0,gk}}{\gamma_m} \tag{E.2}$$

$$\sigma_{c,0,d} = \frac{N_{Ed}}{A} \leq f_{c,0,d} \tag{E.3}$$

$$\sigma_{t,0,d} = \frac{N_{Ed}}{A} \leq f_{t,0,d} \tag{E.4}$$

E.1.2. Shear stress

The following formula comes from NEN-EN 1995-1-1+C1+A1:2011

$$f_{v,d} = k_{\text{mod}} \cdot \frac{f_{v,gk}}{\gamma_m} \tag{E.5}$$

$$\tau_d = \frac{3}{2}\frac{V_d}{A} \leq f_{v,d} \tag{E.6}$$

E.1.3. Bending stress

The following formulas come from NEN-EN 1995-1-1+C1+A1:2011

$k_m = 0.7$ for rectangular cross-sections made of laminated timber

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{E.7}$$

$$\frac{\sigma_{m,z,d}}{f_{m,z,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1 \tag{E.8}$$

E.1.4. Combined Axial and Bending stress

The following formulas come from NEN-EN 1995-1-1+C1+A1:2011

Bending and tension:

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{E.9}$$

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{E.10}$$

Compression and bending:

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{E.11}$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{E.12}$$

E.1.5. Buckling

The following formulas come from NEN-EN 1995-1-1+C1+A1:2011

$$\sigma_{c,0,d} \leq k_c \cdot f_{c,0,d} \tag{E.13}$$

$$k_c = \frac{1}{k + \sqrt{k^2 - \lambda_{rel}^2}} \tag{E.14}$$

$$k = 0.5 \cdot \left(1 + \beta_c \cdot (\lambda_{\mathrm{rel}} - 0.3) + \lambda_{\mathrm{rel}}^2\right) \tag{E.15}$$

$$\beta_c = 0.1 \tag{E.16}$$

$$\lambda_{\mathrm{rel}} = \frac{\lambda}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} \tag{E.17}$$

$$\lambda = \frac{l_{eff}}{i} \tag{E.18}$$

$$i = \frac{I}{A} \tag{E.19}$$

$$u.c. = \frac{\sigma_{c,0,d}}{k_{c,y} \cdot f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \cdot \frac{\sigma_{m,z,d}}{f_{m,z,d}} \tag{E.20}$$

$$\text{u.c.} = \frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} + k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} \tag{E.21}$$

E.2. Fire safety

The following formulas come from NEN-EN 1995-1-2+C2:2011

Material stiffness:

For timber GL28c

$$f_{\mathrm{c}} = 24N/mm^2$$
$$f_{\mathrm{t}} = 19.5N/mm^2$$

$$f_{20} = k_{fi} \tag{E.22}$$

$$k_{fi} = 1.15$$

$$f_{\mathrm{c},20} = 1.15 \cdot 24 = 27.6N/mm^2$$
$$f_{\mathrm{t},20} = 1.15 \cdot 19.5 = 22.4N/mm^2$$

$$f_{\mathrm{d,fi}} = k_{\mathrm{mod,fi}} \frac{f_{20}}{\gamma_{M,\mathrm{fi}}} \tag{E.23}$$

$$k_{\mathrm{mod}} = 1$$

$$\gamma_{\mathrm{M,fl}} = 1$$

$$f_{\mathrm{c,fi}} = 1 \cdot \frac{27.6}{1} = 27.6N/mm^2$$

$$f_{\mathrm{t,fi}} = 1 \cdot \frac{22.4}{1} = 22.4N/mm^2$$

Reduced cross-section method:

The fire safety of the building needs to be $t = 120$ minutes. The smallest timber elements have a size of 250mm x 400 mm so $d_{\mathrm{char},0}$ can be used instead of $d_{\mathrm{char,n}}$.

$$\beta_0 = 0.65$$
$$t = 120$$

$$d_{\text{char, }0} = \beta_0 \cdot t = 0.65 \cdot 120 = 78mm \tag{E.24}$$

$$k_0 = 1$$

$$d_0 = 7$$

$$d_{\mathrm{ef}} = d_{\mathrm{char,n}} + k_0 \cdot d_0 = 78 + 1 \cdot 7 = 85mm \tag{E.25}$$

F

SCIA results

F.1. Diagrid

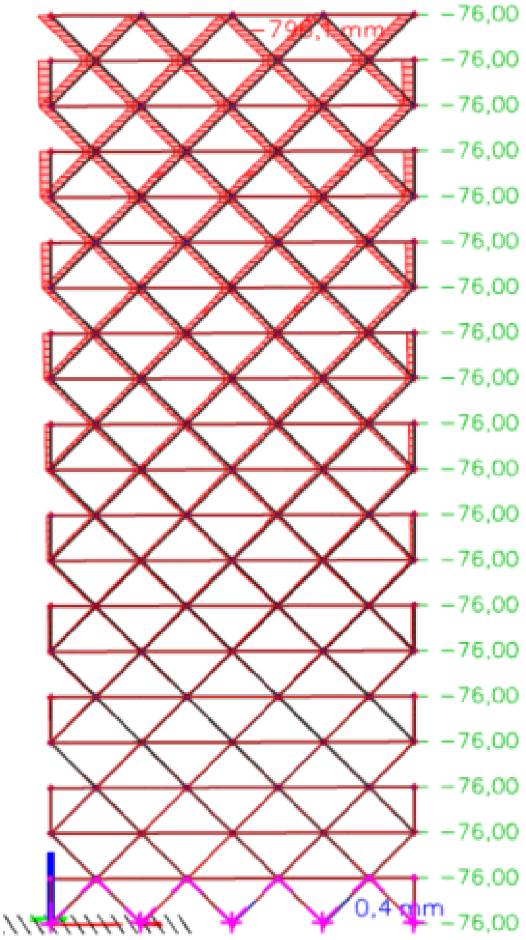


Figure F.1: Deformation of the structure in mm

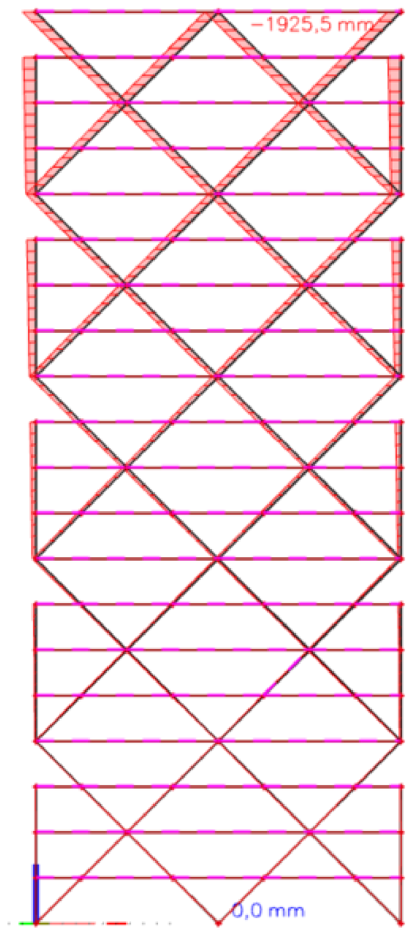


Figure F.2: Deformation of the structure in mm

F.2. External braced frame

F.2.1. Option 1: Single brace with slope 1:2

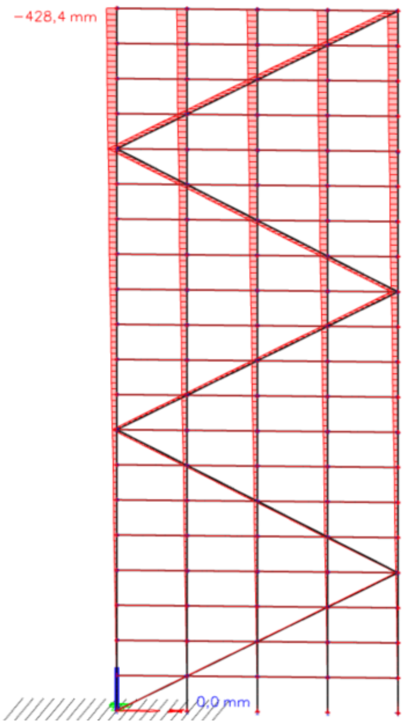


Figure F.3: Deformation of the structure in mm

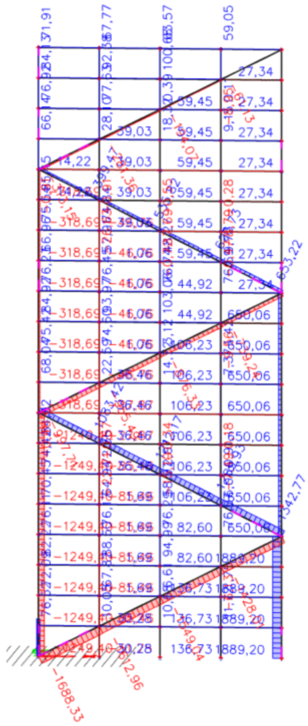


Figure F.4: Normal force of the structure in kN

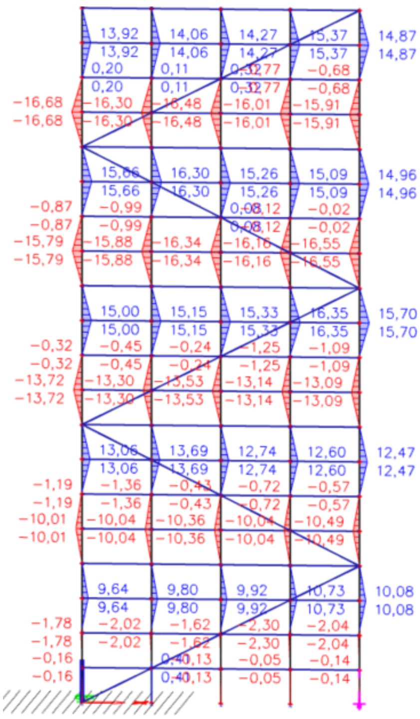


Figure F.5: Moment around the y-axis in kNm

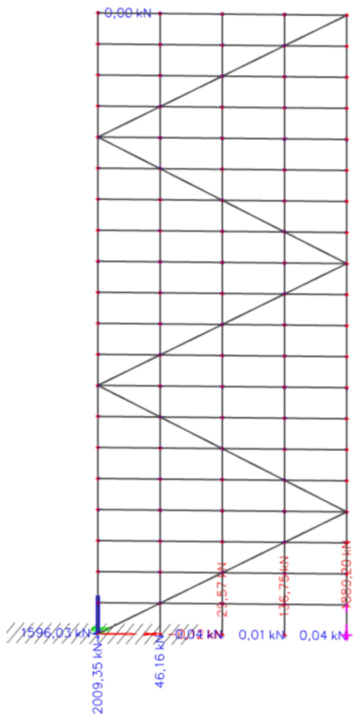


Figure F.6: Support forces of the structure in kN

F.2.2. Option 2: Double brace with slope 1:2

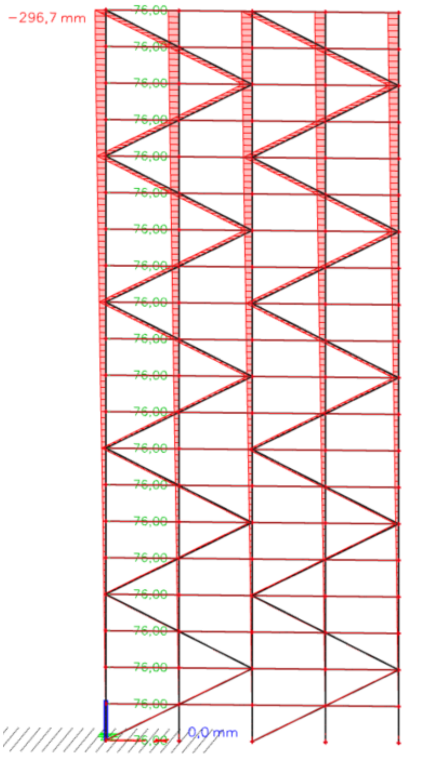


Figure F.7: Deformation of the structure in mm

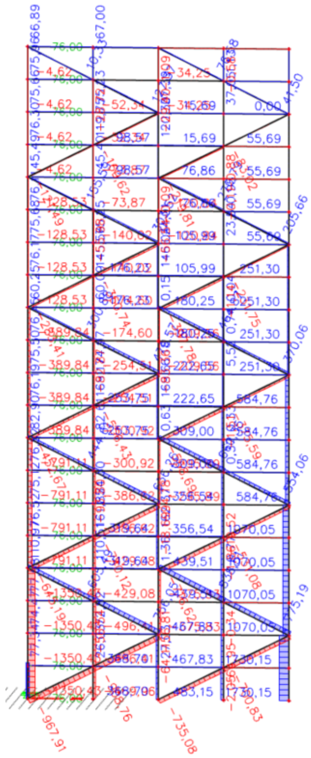


Figure F.8: Normal force of the structure in kN

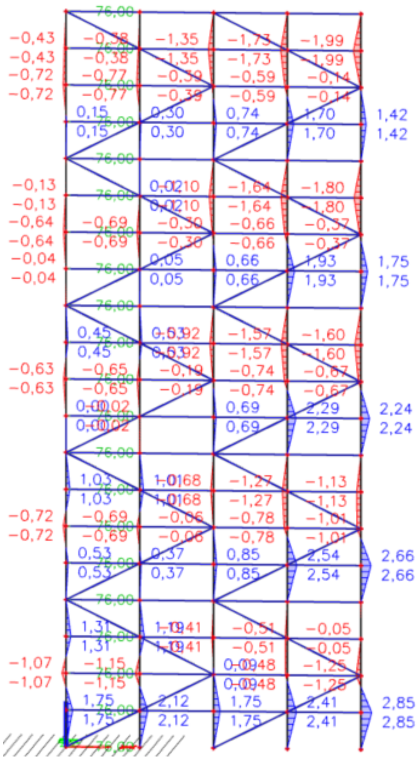


Figure F.9: Moment around the y-axis in kNm

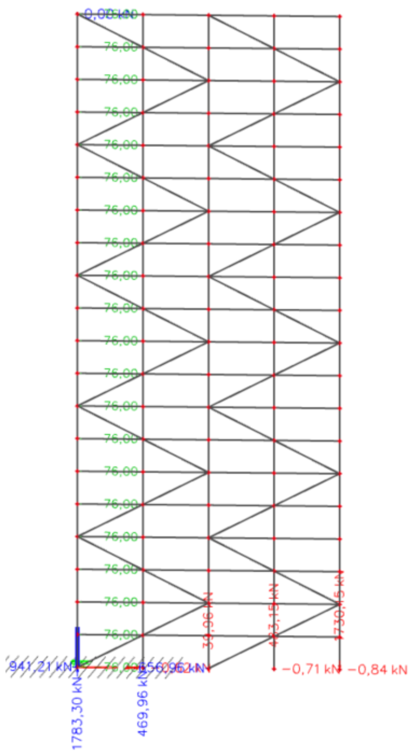


Figure F.10: Support forces of the structure in kN

F.2.3. Option 3: Single brace with slope 1:1

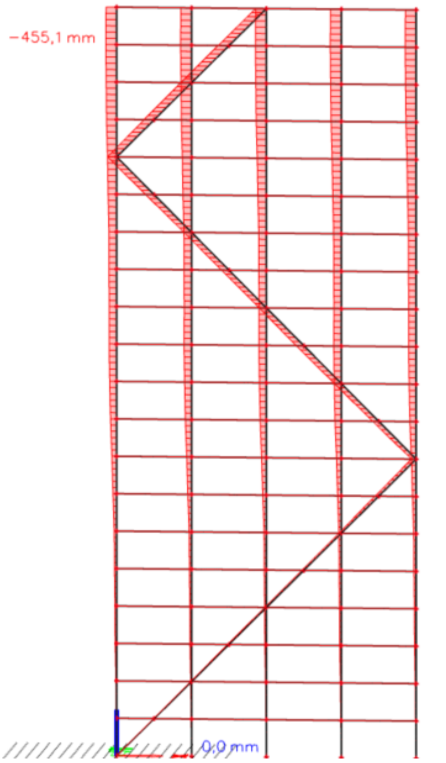


Figure F.11: Deformation of the structure in mm

Figure F.12: Normal force of the structure in kN

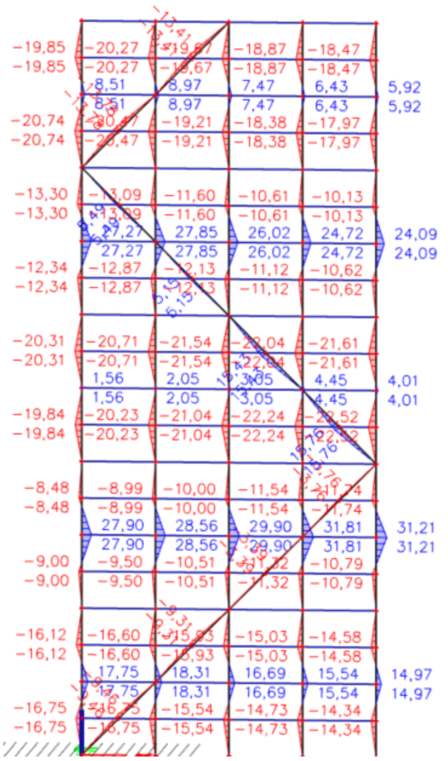


Figure F.13: Moment around the y-axis in kNm

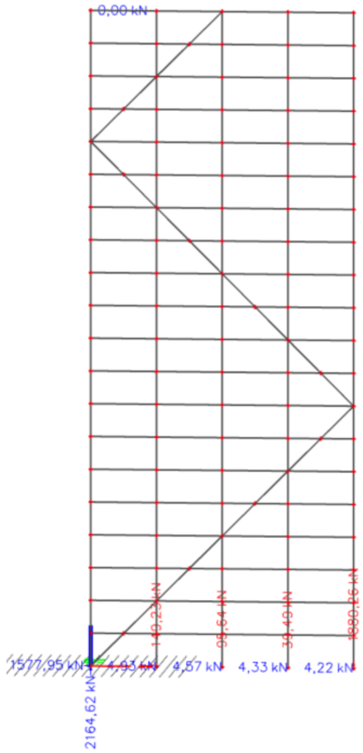


Figure F.14: Support forces of the structure in kN

F.2.4. Option 4: Double brace with slope 1:1

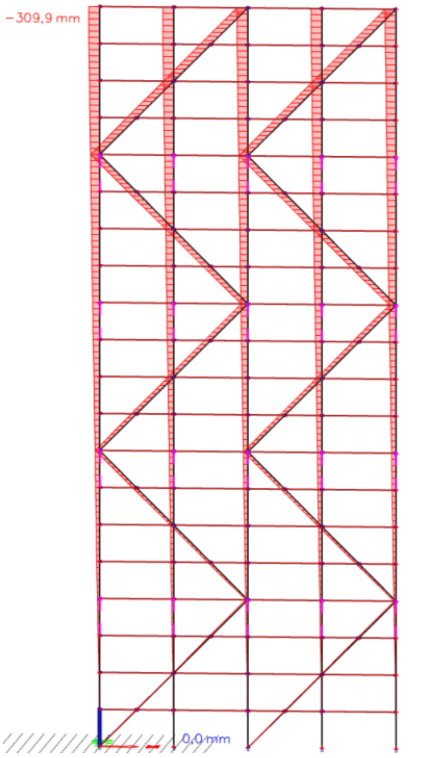


Figure F.15: Deformation of the structure in mm

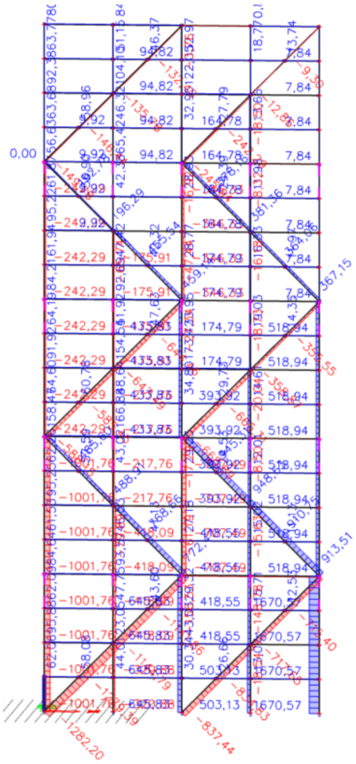


Figure F.16: Normal force of the structure in kN

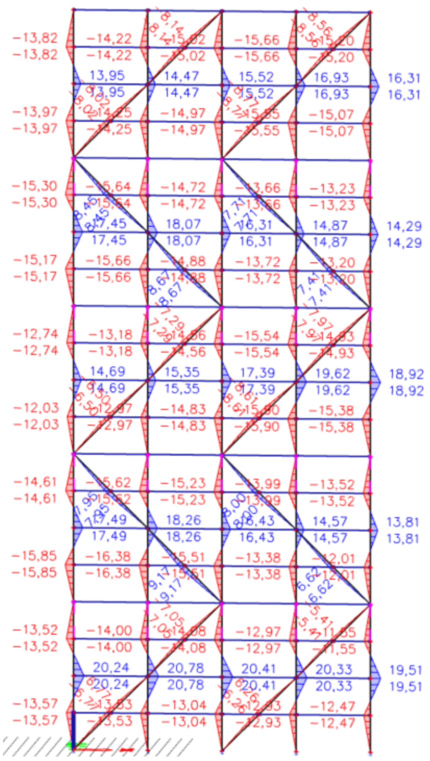


Figure F.17: Moment around the y-axis in kNm

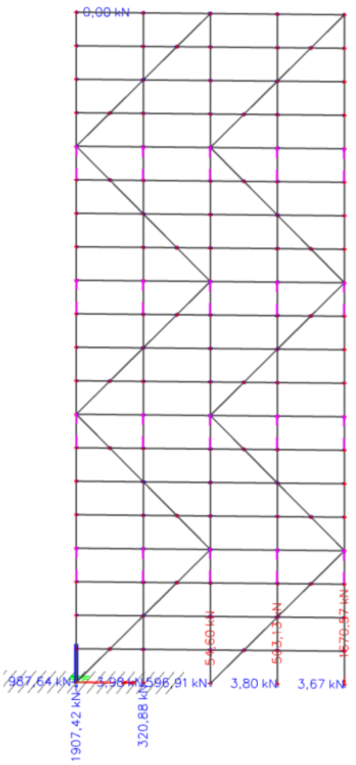


Figure F.18: Support forces of the structure in kN

F.2.5. Option 5: Quadruple brace with slope 1:2

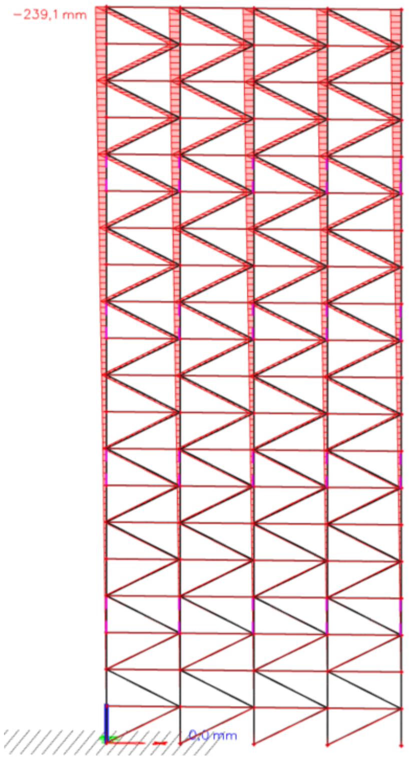


Figure F.19: Deformation of the structure in mm

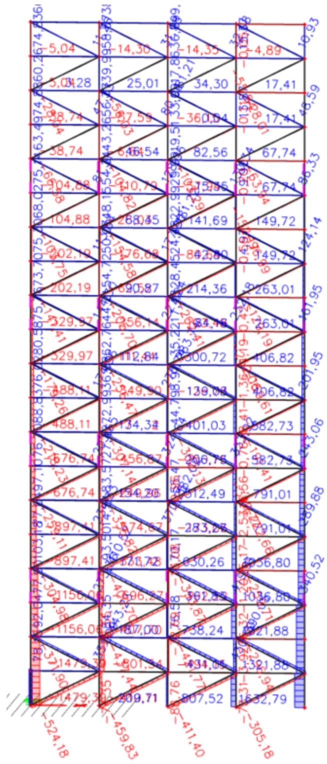


Figure F.20: Normal force of the structure in kN

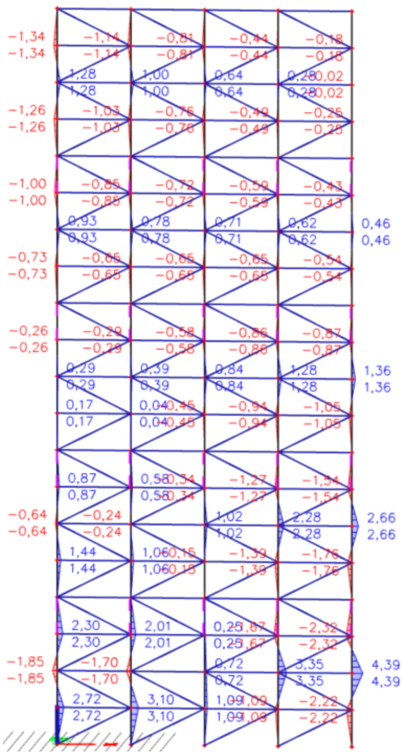


Figure F.21: Moment around the y-axis in kNm

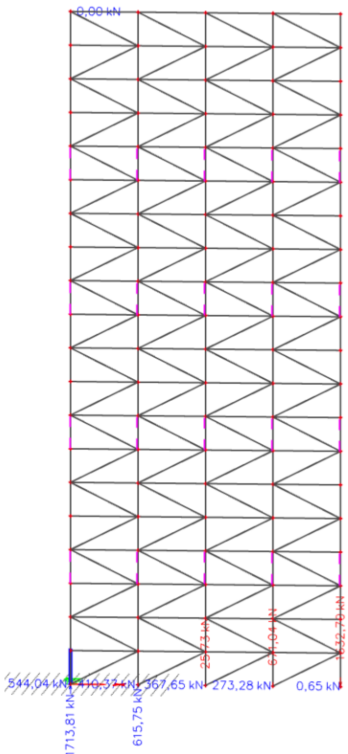


Figure F.22: Support forces of the structure in kN

F.2.6. Option 6: Quadruple brace with slope 1:1

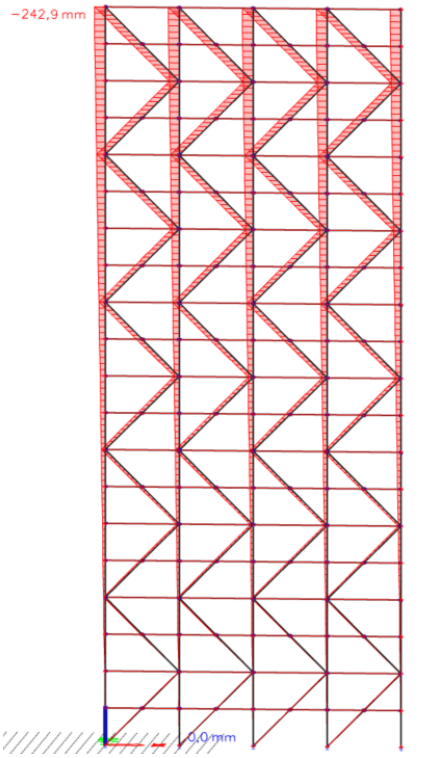


Figure F.23: Deformation of the structure in mm

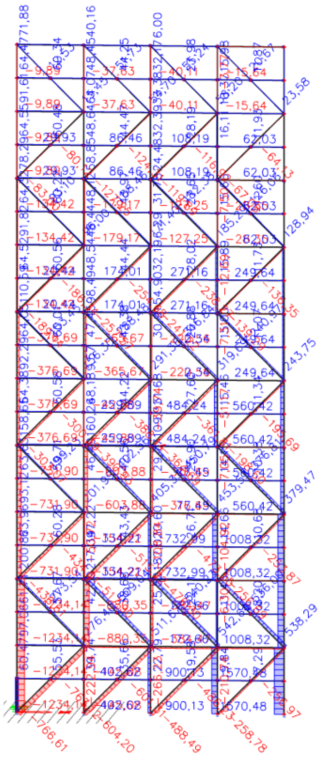


Figure F.24: Normal force of the structure in kN

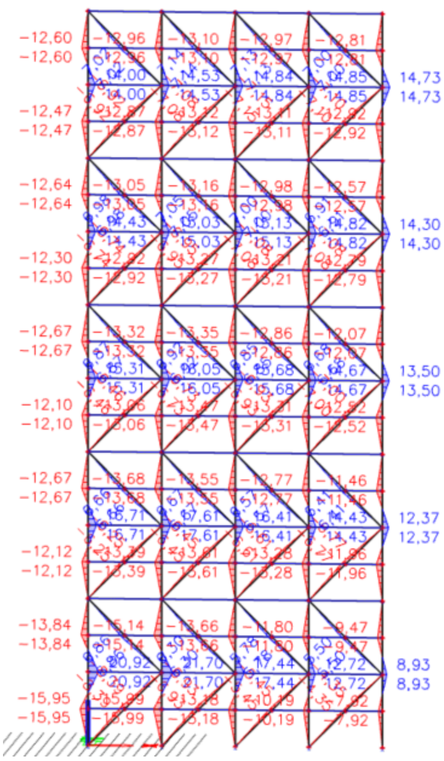


Figure F.25: Moment around the y-axis in kNm

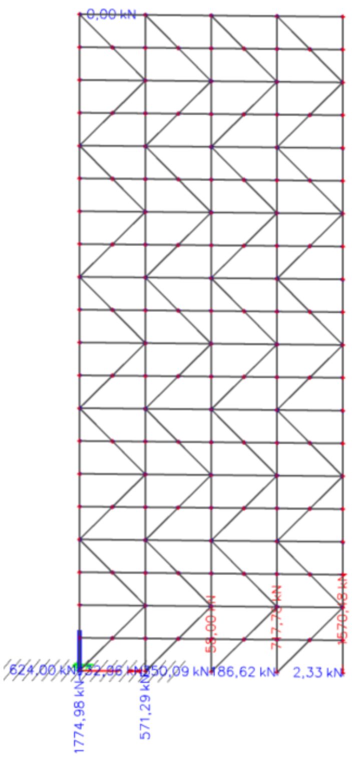


Figure F.26: Support forces of the structure in kN

G

Calculation method for material usage of internal structure

In this appendix the calculation method for the material of the internal structure is shown for one design. The design for which the material usage is shown is a building with floor plan 27.2 x 27.2 m, a floor span of 3.4 meters and permanent floor load 1.

The permanent load of this design is:

Floor load 1	q_k (kN/m^2)
Wooden floor	0.5
Topping	1
Sand	2
Total weight	3.5

Variable loads on the floors are:

	$q_k(kN/m^2)$	ψ_0	ψ_1	ψ_2
Imposed loads residencies, non-common floors	1.75	0.4	0.5	0.3
Imposed load movable walls	1.2	1	1	1

Table G.1: ψ-factors

G.1. Timber

G.1.1. Floors

The floors used in the building are lignatur LFE box floors. The required floor size can be determined with figure G.1. The load on the floor is $q_a + q_b = 3.5 + 1.2 + 1.75 = 6.45\ kN/m^2$ and the floor span is 3.4 meters. This requires a floor size of 140. However, to fit 200 kg/m^2 of sand a floor size of 220 is required. The amount of timber for this floor is [46]:

$$\text{Timber} = 86490\ mm^2/m^1$$

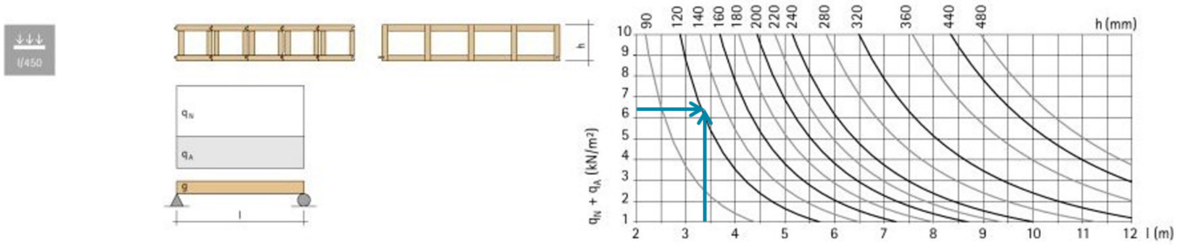


Figure G.1

G.1.2. Beams

For a span of 3.4 meters the permanent floor load is:

$$3.5 * 3.4 = 11.9\ kN/m$$

The variable loads are:

$$1.75 \cdot 3.4 = 5.95\ kN/m$$

$$1.2 \cdot 3.4 = 4.1\ kN/m$$

With calculation files provided by RHDHV the beams are checked on bending, shear and stability as well as during a fire situation. The load combination factors are included in the calculation file. The required beam size is determined by the fire safety and this size is a width of 320 mm and a height of 460 mm with material stiffness GL28c. With this beam size we get the following amount of timber and weight per beam:

$$\text{Timber} = 6.8 \cdot 0.32 \cdot 0.46 = 1\ m^3$$

$$\text{Weight} = 1 \cdot 390/102 = 3.8\ kN$$

G.1.3. Columns

The columns will span across four floors. The floor height is 3.4 meters so the total length of the column is 13.6 meters. With a span of 3.4 meters the floor area going to one column is:

$$\text{Area} = 3.4 \cdot 6.8 = 23.1\ m^2$$

The permanent floor load for one floor is:

$$3.5 * 23.1 = 81\ kN$$

The total permanent load is the sum of the permanent floor load and the weight of the beam times the load combination factor:

$$(81 + 3.8) \cdot 1.2 = 102\ kN$$

The variable loads are:

$$1.75 \cdot 23.1 \cdot 1.5 = 61\ kN$$

$$1.2 \cdot 23.1 \cdot 1 = 28\ kN$$

The total load per floor is:

$$102 + 61 + 28 = 191\ kN$$

For four floors:

$$191 * 4 = 764\ kN$$

This force is put into two calculation files from RHDHV. Both files test the column on axial stress, bending stress, combined stress and buckling. The first file tests for the ULS load combination and the second file tests the column during the fire situation. With a buckling length of 3.4 meters, material GL28c and a force of 764 kN the required column size is 350x350 mm. The amount of timber and weight per column is:

$$\text{Timber} = 0.35 \cdot 0.35 \cdot 3.4 \cdot 4 = 1.67\ m^3$$

$$\text{Weight} = 1.67 \cdot 390/102 = 6.4\ kN$$

The load for the following four floors is:

$$764 \cdot 2 + 6.4 = 1534\ kN$$

Again calculation files from RHDHV are used and they give a required column size of 420x420 mm. These steps are repeated for the other floors giving the following column sizes:

Floors	Size ($mm\ x\ mm$)	Timber (m^3)
17-20	350x350	1.67
13-16	420x420	2.40
9-12	470x470	3.00
5-8	520x520	3.68
1-4	550x550	4.11

G.1.4. Total amount

The total amount of timber for the option with floor size 27.2 x 27.2 m with a floor span of 3.4 m has 7 * 3 internal columns and 7 * 4 internal beams. The total amount of timber used becomes:

$$\text{Floor} = 27.2 \cdot 27.2 \cdot 20 \cdot 0.0865 = 1280\ m^3$$

$$\text{Beams} = 7 \cdot 4 \cdot 20 \cdot 1 = 560\ m^3$$

$$\text{Columns} = 7 \cdot 3 \cdot (1.67 + 2.40 + 3.00 + 3.68 + 4.11) = 312\ m^3$$

$$\text{Total} = 1280 + 560 + 312 = 2152\ m^3$$

G.2. Steel

The point load on the steel connection between the beam and the column receives half of the line load of the beam. With a span of 3.4 meters the floor area going to one beam connection is:

$$\text{Area} = 3.4 \cdot 3.4 = 11.6\ m^2$$

The permanent floor load for one floor is:

$$3.5 * 11.6 = 40\ kN$$

The total permanent load is the sum of the permanent floor load and the weight of the beam times the load combination factor:

$$(40 + 3.8 * 0.5) \cdot 1.2 = 50\ kN$$

The variable loads are:

$$1.75 \cdot 11.6 \cdot 1.5 = 30\ kN$$

$$1.2 \cdot 11.6 \cdot 1 = 14\ kN$$

The total load per floor is:

$$50 + 30 + 14 = 94\ kN$$

To calculate the connection capacity the calculation method of appendix B is used. The python connection design component of the parametric model is altered slightly and used to calculate the required number of bolts. The alteration is that now a value for a_2 and r_{rows} is given instead of calculated by the component. In figure G.2 the component with the in- and output for the connection is shown. The parameters for the connection design are:

- $h = 460\ mm$
- $b = 320\ mm$
- $\alpha = 90°$
- $d = 16\ mm$
- $e_2 = 25\ mm$
- $t_{plate} = 10\ mm$
- $a_3 = 112\ mm$
- $n_{plate} = 2$
- $t_1 = 40\ mm$
- $t_2 = 80\ mm$
- $a_1 = 80\ mm$
- $a_2 = 80\ mm$
- $n_{row} = 1$
- $r_{row} = 2$

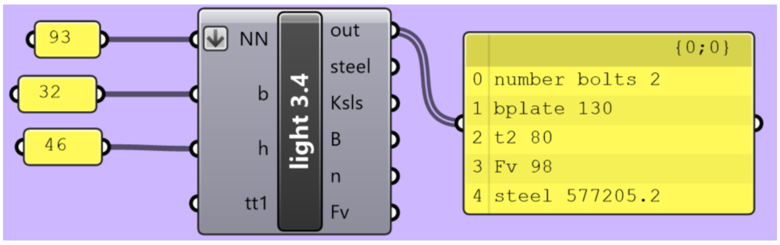
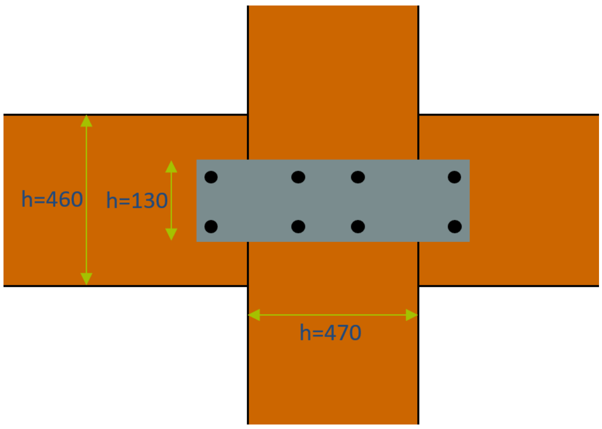


Figure G.2

The connection requires two bolts and will have a plate width of 130 mm. To calculate the amount of timber required in the whole building also the steel in the columns needs to be calculated. The average width of the columns is about 470 mm. The column will have to resist the beam forces on both sides so the loading on the column connection is twice as high as the loading on the beam connection. Therefore, four bolts are required in the column. For these connections all steel is also placed within 85 mm of timber for fire safety. The top of the beam is protected by the floor so the cross-section is not reduced there. This gives the following connection design:



The amount of steel used per column connection is:

$$\text{Plates} = 470 \cdot 130 \cdot 2 \cdot 10 = 1.22 \cdot 10^6\ mm^3$$

$$\text{Bolts} = \pi \cdot (16/2)^2 \cdot (320 - 2 \cdot 85) \cdot 4 = 0.12 \cdot 10^6\ mm^3$$

The total amount of steel in the internal structure becomes:

$$\text{Beams} = 0.58 \cdot 10^6 \cdot 7 \cdot 4 \cdot 20 \cdot 2 = 649 \cdot 10^6\ mm^3$$

$$\text{Columns} = (1.22 + 0.12) \cdot 10^6 \cdot 7 \cdot 3 \cdot 20 = 563 \cdot 10^6\ mm^3$$

$$\text{Total} = 649 + 563 = 1212 \cdot 10^6\ mm^3 = 1.21\ m^3$$

H

Results parametric model

In this appendix all the results collected by the parametric model are shown. There are six tables with results. The tables are divided per stability system design and plot size. All columns in the table will now be explained:

- **Floor span (m)**: This is the input parameter of the floor span in meters.
- **Floor load**: This is the input parameter of the permanent floor load. Load 1 is a permanent floor load of 3.5 kN/m^2, floor load 2 is a load of 5.3 kN/m^2 and load 3 is a load of 6.7 kN/m^2.
- **Width (mm)**: This is the input parameter of the timber element width in millimeters.
- **Timber ULS (m^3)**: This is the output of the amount of timber in the facade after the ULS member checks in cubic meters.
- **Timber fire (m^3)**: This is the output of the amount of timber in the facade after the fire member checks in cubic meters.
- **Timber connection (m^3)**: This is the output of the amount of timber in the facade after the connection design in cubic meters.
- **Timber SLS (m^3)**: This is the output of the amount of timber in the facade after the SLS checks in cubic meters.
- **Total timber (m^3)**: This is the output of the total amount of timber in the building in cubic meters. This includes the timber in the facade, and columns, beams and floor of the internal structure.
- **Steel facade (m^3)**: This is the output of the amount of steel in the facade after the connection design in cubic meters.
- **Total steel (m^3)**: This is the output of the total amount of steel in the building in cubic meters. This includes the steel in the facade calculated by the connection component and the steel used to connect the internal beams to the columns.
- **Displacement (mm)**: This is the output of the global lateral displacement of the model in millimeter. The displacement requirement for all the models was 136 mm.
- **Acceleration (m/s^2)**: This is the output of the along-wind acceleration of the model in m/s^2.
- **Acceleration requirement (m/s^2)**: This is the output of the acceleration requirement of the model in m/s^2. The acceleration should be lower than the acceleration requirement.

H.1. Single brace plot size 27.2 x 27.2 m

Floor span (m)	Floor load	Width (mm)	Timber ULS (m³)	Timber fire (m³)	Timber connection (m³)	Timber SLS (m³)	Total timber (m³)	Steel facade (m³)	Total steel (m³)	Displacement (mm)	Acceleration (m/s²)	Acceleration requirement (m/s²)
3.4	1	400	673	799	1046	9484	11636	5.0	6.2	90	0.152	0.152
3.4	1	450	710	826	1030	9289	11441	4.0	5.2	90	0.152	0.152
3.4	1	500	744	857	1029	9337	11489	3.7	4.9	90	0.152	0.152
3.4	1	550	786	920	1069	9151	11303	3.6	4.8	90	0.152	0.152
3.4	1	600	823	974	1110	9139	11291	3.6	4.8	89	0.152	0.152
3.4	1	650	861	1025	1164	9078	11230	3.7	4.9	89	0.152	0.152
3.4	2	400	750	867	1159	5450	8352	5.7	7.1	95	0.154	0.154
3.4	2	450	777	883	1119	6426	9328	4.6	6.0	95	0.154	0.154
3.4	2	500	813	925	1133	6400	9302	4.3	5.6	95	0.154	0.154
3.4	2	550	846	974	1159	5974	8849	4.0	5.3	94	0.154	0.154
3.4	2	600	893	1023	1194	5379	8281	4.1	5.5	94	0.153	0.153
3.4	2	650	919	1063	1236	5341	8243	4.1	5.5	94	0.153	0.153
3.4	3	400	795	904	1219	4160	7576	6.3	8.5	100	0.155	0.156
3.4	3	450	835	938	1197	4193	7609	5.1	7.2	100	0.155	0.156
3.4	3	500	863	975	1190	4090	7506	4.7	6.9	100	0.155	0.155
3.4	3	550	900	1012	1213	4132	7548	4.4	6.7	100	0.155	0.155
3.4	3	600	936	1070	1258	4073	7489	4.4	6.6	99	0.155	0.155
3.4	3	650	973	1116	1301	4085	7501	4.5	6.8	99	0.155	0.155
6.8	1	400	700	849	1135	8919	10996	4.5	5.4	64	0.141	0.141
6.8	1	450	773	898	1121	8922	10998	3.8	4.8	64	0.141	0.141
6.8	1	500	806	942	1133	8752	10829	3.2	4.2	64	0.140	0.141
6.8	1	550	843	992	1167	8754	10831	3.1	4.1	63	0.140	0.140
6.8	1	600	875	1041	1203	8629	10705	3.1	4.0	63	0.140	0.140
6.8	1	650	903	1081	1242	8556	10632	3.0	4.0	63	0.140	0.140
6.8	2	400	750	941	1276	4064	6545	5.4	6.5	73	0.145	0.145
6.8	2	450	828	977	1242	4044	6525	4.6	5.6	73	0.145	0.145
6.8	2	500	895	1024	1242	4000	6481	3.9	5.0	72	0.140	0.140
6.8	2	550	939	1084	1278	3962	6443	3.7	4.8	72	0.144	0.145
6.8	2	600	974	1132	1309	3900	6381	3.5	4.6	72	0.144	0.145
6.8	2	650	1000	1180	1342	3890	6371	3.5	4.6	71	0.144	0.145
6.8	3	400	742	978	1374	1510	4493	6.2	7.7	100	0.155	0.155
6.8	3	450	854	1037	1346	1549	4532	5.1	6.6	99	0.155	0.155
6.8	3	500	882	1079	1347	1541	4524	4.4	6.0	100	0.155	0.155
6.8	3	550	1016	1148	1354	1598	4581	4.1	5.7	98	0.154	0.155
6.8	3	600	1041	1197	1381	1629	4612	4.0	5.5	99	0.155	0.155
6.8	3	650	1079	1232	1409	1599	4582	3.8	5.4	101	0.156	0.156

H.2. Single brace plot size 27.2 x 40.8 m

Floor span (m)	Floor load	Width (mm)	Timber ULS (m^3)	Timber fire (m^3)	Timber connection (m^3)	Timber SLS (m^3)	Total timber (m^3)	Steel facade (m^3)	Total steel (m^3)	Displacement (mm)	Acceleration (m/s^2)	Acceleration requirement (m/s^2)
3.4	1	400	864	1019	1363	1677	4968	6.3	8.2	121	0.148	0.148
3.4	1	450	907	1042	1335	1699	4990	5.1	7.0	122	0.149	0.149
3.4	1	500	950	1087	1340	1729	5020	4.7	6.6	122	0.148	0.149
3.4	1	550	997	1161	1368	1744	5035	4.6	6.5	122	0.149	0.149
3.4	1	600	1050	1224	1413	1789	5080	4.7	6.6	122	0.148	0.149
3.4	1	650	1095	1293	1480	1838	5129	4.7	6.6	123	0.149	0.149
3.4	2	400	959	1095	1500	1548	5979	7.4	9.5	135	0.131	0.159
3.4	2	450	1006	1122	1456	1546	5977	5.9	8.1	135	0.131	0.159
3.4	2	500	1039	1161	1464	1573	6004	5.5	7.7	136	0.131	0.159
3.4	2	550	1080	1223	1489	1616	6047	5.1	7.3	135	0.131	0.159
3.4	2	600	1136	1295	1542	1669	6100	5.4	7.5	135	0.131	0.159
3.4	2	650	1170	1342	1574	1700	6131	5.3	7.4	135	0.131	0.159
3.4	3	400	1030	1156	1594	1619	6827	8.2	11.7	136	0.118	0.164
3.4	3	450	1071	1180	1551	1622	6830	6.6	10.1	135	0.118	0.164
3.4	3	500	1119	1229	1555	1646	6854	6.1	9.6	136	0.118	0.164
3.4	3	550	1143	1272	1568	1679	6887	5.8	9.3	136	0.118	0.164
3.4	3	600	1195	1333	1621	1733	6941	5.8	9.3	136	0.118	0.164
3.4	3	650	1242	1407	1669	1780	6988	6.0	9.5	136	0.118	0.164
6.8	1	400	844	1051	1420	1764	4974	5.9	7.5	121	0.148	0.148
6.8	1	450	922	1120	1407	1783	4993	4.9	6.6	121	0.148	0.149
6.8	1	500	961	1172	1424	1813	5023	4.4	6.0	122	0.149	0.149
6.8	1	550	1045	1229	1448	1849	5059	4.1	5.8	122	0.149	0.149
6.8	1	600	1094	1293	1491	1897	5107	4.1	5.7	123	0.149	0.149
6.8	1	650	1120	1338	1529	1926	5136	4.0	5.6	123	0.149	0.149
6.8	2	400	948	1151	1576	1616	5447	7.2	9.0	136	0.132	0.158
6.8	2	450	1041	1224	1556	1647	5478	6.0	7.8	135	0.132	0.158
6.8	2	500	1072	1274	1551	1653	5484	5.2	7.0	135	0.132	0.158
6.8	2	550	1113	1332	1586	1680	5511	5.0	6.8	136	0.132	0.158
6.8	2	600	1213	1404	1622	1724	5555	4.8	6.6	136	0.132	0.158
6.8	2	650	1249	1458	1666	1776	5607	4.8	6.6	136	0.132	0.159
6.8	3	400	979	1240	1714	1738	6333	8.2	10.7	134	0.118	0.163
6.8	3	450	1087	1276	1660	1707	6302	6.6	9.2	136	0.119	0.163
6.8	3	500	1164	1359	1649	1744	6339	5.9	8.4	136	0.118	0.163
6.8	3	550	1203	1414	1682	1819	6414	5.6	8.2	135	0.118	0.163
6.8	3	600	1243	1466	1715	1817	6412	5.3	7.9	135	0.118	0.163
6.8	3	650	1338	1524	1754	1861	6456	5.3	7.9	134	0.118	0.163

H.3. Double brace plot size 27.2 x 27.2 m

Floor span (m)	Floor load	Width (mm)	Timber ULS (m^3)	Timber fire (m^3)	Timber connection (m^3)	Timber SLS (m^3)	Total timber (m^3)	Steel facade (m^3)	Total steel (m^3)	Displacement (mm)	Acceleration (m/s^2)	Acceleration requirement (m/s^2)
3.4	1	400	712	878	1130	6953	9105	5.2	6.4	82	0.149	0.149
3.4	1	450	757	893	1085	7427	9579	4.2	5.4	89	0.151	0.152
3.4	1	500	802	952	1108	7296	9448	4.1	5.3	87	0.151	0.151
3.4	1	550	843	1003	1142	7307	9459	3.9	5.2	88	0.151	0.151
3.4	1	600	893	1061	1198	7258	9410	4.0	5.2	87	0.151	0.151
3.4	1	650	946	1130	1269	7287	9439	3.8	5.1	90	0.152	0.152
3.4	2	400	787	928	1224	4269	7171	5.9	7.3	84	0.150	0.150
3.4	2	450	823	955	1186	4601	7503	4.8	6.1	89	0.152	0.152
3.4	2	500	869	1005	1193	4481	7383	4.6	6.0	87	0.151	0.151
3.4	2	550	908	1055	1235	4563	7465	4.4	5.7	89	0.151	0.152
3.4	2	600	960	1125	1287	4626	7528	4.4	5.8	89	0.151	0.152
3.4	2	650	997	1167	1351	4674	7576	4.3	5.7	91	0.152	0.152
3.4	3	400	806	974	1292	2703	6119	6.4	8.7	86	0.150	0.151
3.4	3	450	881	1005	1257	3301	6717	5.2	7.4	91	0.152	0.153
3.4	3	500	915	1048	1267	2942	6358	5.0	7.3	89	0.152	0.152
3.4	3	550	954	1098	1290	3160	6576	4.8	7.0	90	0.152	0.152
3.4	3	600	1004	1159	1347	3127	6543	4.8	7.0	90	0.152	0.152
3.4	3	650	1048	1215	1408	3412	6828	4.7	7.0	93	0.153	0.153
6.8	1	400	718	920	1213	5746	7823	4.6	5.6	46	0.130	0.130
6.8	1	450	786	964	1187	6124	8201	3.9	4.9	47	0.131	0.131
6.8	1	500	825	1013	1205	5891	7968	3.5	4.5	46	0.130	0.130
6.8	1	550	895	1078	1245	6056	8132	3.4	4.3	47	0.131	0.131
6.8	1	600	944	1144	1283	6068	8145	3.4	4.3	47	0.131	0.131
6.8	1	650	978	1182	1350	6410	8486	3.3	4.3	48	0.131	0.132
6.8	2	400	717	988	1347	1952	4433	5.6	6.7	60	0.138	0.139
6.8	2	450	840	1042	1313	2127	4608	4.7	5.8	61	0.139	0.139
6.8	2	500	877	1084	1322	2037	4518	4.1	5.2	61	0.139	0.139
6.8	2	550	955	1152	1344	2118	4599	3.9	5.0	61	0.139	0.139
6.8	2	600	1034	1221	1392	2123	4604	3.8	4.9	61	0.139	0.139
6.8	2	650	1072	1286	1443	2122	4603	3.7	4.8	62	0.139	0.140
6.8	3	400	713	1036	1437	1437	4439	6.3	7.9	70	0.136	0.146
6.8	3	450	791	1085	1407	1407	4417	5.2	6.7	74	0.139	0.148
6.8	3	500	851	1134	1408	1408	4423	4.7	6.2	74	0.139	0.148
6.8	3	550	983	1193	1432	1432	4448	4.4	6.0	74	0.139	0.148
6.8	3	600	1061	1270	1467	1467	4481	4.3	5.8	73	0.138	0.147
6.8	3	650	1138	1327	1514	1514	4528	4.1	5.7	72	0.137	0.147

H.4. Double brace plot size 27.2 x 40.8 m

Floor span (m)	Floor load	Width (mm)	Timber ULS (m^3)	Timber fire (m^3)	Timber connection (m^3)	Timber SLS (m^3)	Total timber (m^3)	Steel facade (m^3)	Total steel (m^3)	Displacement (mm)	Acceleration (m/s^2)	Acceleration requirement (m/s^2)
3.4	1	400	889	1104	1431	1431	4742	6.7	8.6	96	0.135	0.142
3.4	1	450	968	1108	1369	1369	4687	5.4	7.3	99	0.137	0.143
3.4	1	500	1024	1184	1396	1396	4714	4.9	6.9	101	0.138	0.144
3.4	1	550	1076	1242	1457	1457	4773	4.8	6.7	102	0.139	0.144
3.4	1	600	1137	1320	1513	1513	4829	4.9	6.8	101	0.138	0.144
3.4	1	650	1207	1384	1601	1601	4910	5.0	6.9	100	0.137	0.144
3.4	2	400	979	1173	1549	1549	6003	7.7	9.8	93	0.114	0.149
3.4	2	450	1028	1187	1495	1495	5948	6.1	8.2	96	0.116	0.150
3.4	2	500	1114	1256	1512	1512	5972	5.6	7.8	98	0.116	0.150
3.4	2	550	1157	1312	1558	1558	6020	5.4	7.5	99	0.117	0.150
3.4	2	600	1223	1387	1623	1623	6079	5.6	7.7	99	0.116	0.150
3.4	2	650	1281	1447	1696	1696	6151	5.6	7.7	97	0.116	0.150
3.4	3	400	1038	1232	1639	1639	6872	8.5	12.0	92	0.102	0.153
3.4	3	450	1097	1248	1582	1582	6812	6.6	10.1	95	0.104	0.154
3.4	3	500	1162	1314	1594	1594	6830	6.1	9.6	96	0.104	0.154
3.4	3	550	1241	1384	1642	1642	6877	5.9	9.4	97	0.104	0.154
3.4	3	600	1285	1432	1697	1697	6933	5.9	9.4	97	0.104	0.154
3.4	3	650	1342	1506	1766	1766	7000	6.1	9.6	96	0.104	0.154
6.8	1	400	885	1141	1510	1510	4739	6.0	7.7	90	0.132	0.141
6.8	1	450	990	1205	1488	1488	4716	5.2	6.8	93	0.133	0.142
6.8	1	500	1045	1272	1518	1518	4744	4.6	6.2	93	0.134	0.142
6.8	1	550	1098	1343	1578	1578	4802	4.4	6.0	94	0.134	0.142
6.8	1	600	1192	1428	1631	1631	4856	4.3	5.9	94	0.134	0.142
6.8	1	650	1237	1485	1704	1704	4928	4.3	5.9	92	0.133	0.142
6.8	2	400	867	1218	1668	1668	5508	7.2	9.0	88	0.112	0.147
6.8	2	450	943	1284	1633	1633	5485	6.1	7.9	90	0.113	0.148
6.8	2	500	1112	1369	1662	1662	5512	5.4	7.2	91	0.114	0.148
6.8	2	550	1184	1440	1714	1714	5562	5.2	7.0	92	0.114	0.148
6.8	2	600	1261	1514	1750	1750	5600	4.9	6.8	91	0.114	0.148
6.8	2	650	1316	1584	1821	1821	5669	4.9	6.7	90	0.113	0.148
6.8	3	400	857	1280	1781	1781	6387	8.1	10.7	87	0.101	0.151
6.8	3	450	977	1344	1745	1745	6355	6.8	9.3	89	0.102	0.152
6.8	3	500	1057	1417	1753	1753	6369	6.0	8.6	90	0.102	0.152
6.8	3	550	1249	1511	1810	1810	6424	5.8	8.3	90	0.102	0.152
6.8	3	600	1319	1582	1855	1855	6466	5.5	8.1	90	0.102	0.152
6.8	3	650	1398	1645	1900	1900	6510	5.5	8.0	89	0.101	0.152

H.5. Diagrid plot size 27.2 x 27.2 m

Floor span (m)	Floor load	Width (mm)	Timber ULS (m³)	Timber fire (m³)	Timber connection (m³)	Timber SLS (m³)	Total timber (m³)	Steel facade (m³)	Total steel (m³)	Displacement (mm)	Acceleration (m/s²)	Acceleration requirement (m/s²)
3.4	400	1	995	1158	1654	5719	7871	18.0	19.2	46	0.130	0.130
3.4	450	1	1061	1201	1601	6626	8778	14.7	15.9	49	0.132	0.132
3.4	500	1	1096	1255	1601	7083	9235	13.5	14.7	51	0.133	0.133
3.4	550	1	1142	1305	1628	7353	9505	13.1	14.3	52	0.133	0.134
3.4	600	1	1184	1354	1665	7348	9500	13.2	14.4	52	0.134	0.134
3.4	650	1	1231	1411	1720	7250	9402	13.6	14.8	52	0.134	0.134
3.4	400	2	1100	1284	1853	1853	4755	21.8	23.2	58	0.135	0.138
3.4	450	2	1171	1325	1784	1784	4686	17.7	19.1	62	0.139	0.140
3.4	500	2	1220	1374	1763	1936	4838	15.8	17.2	63	0.140	0.140
3.4	550	2	1279	1416	1780	2128	5030	15.5	16.9	62	0.138	0.140
3.4	600	2	1315	1474	1817	2172	5074	15.6	16.9	62	0.139	0.140
3.4	650	2	1357	1521	1872	2055	4957	15.8	17.2	63	0.140	0.140
3.4	400	3	1192	1383	2010	2010	5426	25.0	27.2	55	0.126	0.139
3.4	450	3	1237	1402	1918	1918	5334	20.1	22.4	59	0.130	0.141
3.4	500	3	1311	1460	1903	1903	5319	17.8	20.1	61	0.132	0.141
3.4	550	3	1366	1508	1910	1910	5326	17.5	19.7	62	0.133	0.142
3.4	600	3	1419	1558	1927	1927	5343	17.2	19.5	62	0.133	0.142
3.4	650	3	1463	1620	1981	1981	5397	17.7	19.9	61	0.133	0.142
6.8	400	1	1083	1256	1839	5463	7540	21.5	22.5	44	0.128	0.129
6.8	450	1	1154	1291	1761	5913	7989	17.5	18.5	46	0.130	0.130
6.8	500	1	1226	1346	1746	6369	8445	15.8	16.8	48	0.131	0.131
6.8	550	1	1269	1391	1759	6579	8655	15.3	16.3	49	0.132	0.132
6.8	600	1	1309	1445	1800	6551	8628	15.5	16.4	49	0.132	0.132
6.8	650	1	1349	1496	1859	6759	8835	15.8	16.8	48	0.131	0.132
6.8	400	2	1186	1413	2093	2093	4574	26.7	27.8	53	0.131	0.136
6.8	450	2	1287	1440	1997	1997	4478	21.6	22.7	57	0.135	0.138
6.8	500	2	1326	1476	1953	1953	4434	18.8	19.9	60	0.138	0.139
6.8	550	2	1383	1511	1960	1960	4441	18.4	19.5	60	0.138	0.139
6.8	600	2	1476	1589	1976	1976	4457	18.3	19.4	60	0.138	0.139
6.8	650	2	1522	1644	2024	2024	4505	18.8	19.9	60	0.138	0.139
6.8	400	3	1209	1521	2288	2288	5271	30.6	32.2	50	0.117	0.139
6.8	450	3	1328	1544	2174	2174	5157	24.6	26.1	54	0.120	0.140
6.8	500	3	1448	1590	2124	2124	5107	21.4	23.0	56	0.123	0.141
6.8	550	3	1488	1638	2116	2116	5099	20.9	22.5	57	0.123	0.142
6.8	600	3	1546	1673	2133	2133	5116	20.5	22.1	57	0.124	0.142
6.8	650	3	1629	1746	2167	2167	5150	21.2	22.7	57	0.123	0.142

H.6. Diagrid plot size 27.2 x 40.8 m

Floor span (m)	Floor load	Width (mm)	Timber ULS (m^3)	Timber fire (m^3)	Timber connection (m^3)	Timber SLS (m^3)	Total timber (m^3)	Steel facade (m^3)	Total steel (m^3)	Displacement (mm)	Acceleration (m/s^2)	Acceleration requirement (m/s^2)
3.4	400	1	1254	1497	2105	2105	5396	22.8	24.7	68	0.115	0.135
3.4	450	1	1327	1541	2041	2041	5332	18.8	20.7	74	0.119	0.137
3.4	500	1	1396	1604	2024	2024	5315	17.3	19.2	76	0.122	0.137
3.4	550	1	1442	1672	2063	2063	5354	16.8	18.8	77	0.122	0.138
3.4	600	1	1493	1727	2116	2116	5407	16.8	18.7	78	0.122	0.138
3.4	650	1	1542	1796	2173	2173	5464	17.3	19.3	77	0.121	0.138
3.4	400	2	1363	1656	2379	2379	6810	28.3	30.5	62	0.095	0.139
3.4	450	2	1460	1699	2278	2278	6709	22.8	24.9	67	0.098	0.141
3.4	500	2	1521	1750	2255	2255	6686	20.5	22.7	70	0.100	0.142
3.4	550	2	1605	1819	2269	2269	6700	19.9	22.1	70	0.101	0.142
3.4	600	2	1679	1902	2320	2320	6751	20.1	22.2	70	0.100	0.142
3.4	650	2	1727	1959	2384	2384	6815	20.4	22.5	70	0.100	0.142
3.4	400	3	1457	1786	2586	2586	7794	32.5	36.0	58	0.084	0.142
3.4	450	3	1551	1801	2464	2464	7672	26.2	29.7	63	0.087	0.144
3.4	500	3	1645	1870	2427	2427	7635	23.0	26.5	66	0.089	0.145
3.4	550	3	1708	1931	2440	2440	7648	22.3	25.9	66	0.089	0.145
3.4	600	3	1778	1999	2473	2473	7681	22.3	25.8	66	0.089	0.145
3.4	650	3	1869	2087	2532	2532	7740	22.9	26.4	66	0.089	0.145
6.8	400	1	1318	1577	2284	2284	5494	26.6	28.2	63	0.111	0.133
6.8	450	1	1413	1620	2189	2189	5399	21.7	23.4	68	0.115	0.135
6.8	500	1	1479	1670	2170	2170	5380	19.6	21.2	71	0.117	0.136
6.8	550	1	1568	1747	2190	2190	5400	19.0	20.7	72	0.118	0.136
6.8	600	1	1614	1813	2232	2232	5442	19.1	20.7	72	0.118	0.136
6.8	650	1	1663	1865	2308	2308	5518	19.4	21.0	72	0.118	0.136
6.8	400	2	1460	1779	2609	2609	6440	33.1	34.9	58	0.092	0.137
6.8	450	2	1545	1800	2481	2481	6312	26.8	28.6	62	0.095	0.139
6.8	500	2	1634	1854	2430	2430	6261	23.4	25.2	65	0.098	0.140
6.8	550	2	1721	1915	2444	2444	6275	22.9	24.7	65	0.098	0.140
6.8	600	2	1771	1970	2470	2470	6301	22.7	24.5	66	0.098	0.140
6.8	650	2	1882	2067	2532	2532	6363	23.2	25.0	65	0.098	0.140
6.8	400	3	1500	1915	2866	2866	7461	38.3	40.9	54	0.082	0.140
6.8	450	3	1643	1940	2713	2713	7308	30.5	33.1	58	0.085	0.142
6.8	500	3	1747	1997	2648	2648	7243	26.7	29.3	61	0.086	0.143
6.8	550	3	1820	2039	2631	2631	7226	25.9	28.5	62	0.087	0.143
6.8	600	3	1908	2109	2667	2667	7262	25.7	28.2	62	0.087	0.143
6.8	650	3	1978	2172	2705	2705	7300	26.2	28.8	62	0.087	0.143