Precast Concrete in Framed Tube High-Rise Structures

Challenges and possibilities of prefabricated concrete in tube-in-tube structures of approximately 200m high

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Preface

The document in front of you is the final report for the Master’s thesis concluding my study Civil Engineering at Delft, University of Technology. My interest in high-rise structures is one of the reasons for choosing the Master track Building Engineering - Structural Design and I feel that the combination of the abstract and the more applied parts of this thesis titled *Precast Concrete in Framed Tube High-Rise Structures* are exemplar for this Master track. To say I enjoyed every minute of making this thesis would be a lie, but the fact that I could keep myself inspired for the last months tells me that I made the right choice for a study, Master track and subject for my thesis.

First, I would like to thank my graduation committee, consisting of the prof. ir. R. Nijssse, Dr. ir. drs. C.R. Braam, ing. W.H. van Dijk and ir. D.C. van Keulen. I would like to highlight the latter for being the committee member which helped me the most from the definition of the subject to the end of the writing of the thesis.

I would also like to thank Technosoft Nederland for providing me with a license for AxisVM to use during my graduation and helping me with questions even ir. D.C. van Keulen did not know the answers to.

Apart from W.H. van Dijk personally, I would like to thank the organisation of Ballast Nedam Bouw & Ontwikkeling Speciale Projecten for providing me an opportunity to perform my graduation thesis in co-operation with them and providing me with a compensation to do so. I sincerely hope that my thesis can be of use to them in the future.

I would like to thank all my friends and family for providing me with support, distractions and encouragements during the writing of my thesis. All my roommates I would like to thank for the countless breakfasts, lunches, dinners and other random encounters I had with them. I cannot imagine a place to write the largest parts of my thesis where I would feel more at home.

Lastly, I would like to thank my parents for all their financial and emotional support over the course of my entire study Civil Engineering. The assistance of Delft University of Technology alumnus Ir. E.J. Hummelen is acknowledged for proofreading my thesis.

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Summary

In this thesis a particular structural system for high-rise structures is discussed: the prefabricated concrete framed tube and tube-in-tube structures. These system consist of a prefabricated concrete perforated elements in the perimeter of the structure that are the main lateral load resisting elements in the building. For tube-in-tube structures, the perimeter elements work together with a structural core that also resists the high lateral loads that are due to the wind loading on high rise structures.

The thesis starts with a literature study on multiple aspects that are associated with the subject. First, an introduction into prefabricated high-rise and framed tube structures is given and some key behaviors and elements of this kind of structures are explained, such as the shear lag effect and aspects of connections between prefabricated elements. Hereafter, previous research associated with the subject is described. Most of these titles are Master’s Theses from Delft University of Technology, which is at the forefront for research on prefabricated concrete high-rise structures. Other references found are articles published in Asian countries or the United States of America, but those tend to focus on cast-in-place framed tube structures. Finally, sources describing the connection between elements are described, which is an important parameter in prefabricated concrete shear wall structures.

The literature study continues with an analysis of prefabricated high-rise structures in the Netherlands. It presents different structures already constructed and their relevance to the subject of the thesis. Also a description of the Dutch high-rise market is given as well as the justification for the construction of high-rise towers in this market.

The literature study leads to the section on the missing links in research and the research questions associated with these missing parts of information available in literature. As a result, the research part of the thesis is divided into three parts:

- Shear lag effect of non-rectangular framed tube structures
- Structural behavior of prefabricated framed tube structures
- Case Study Zalmhaventoren: Potential of fully prefabricated tube-in-tube structures

These missing links lead to two separate parts of this thesis.

Firstly a parameter research was done to answer the research questions associated with the first two missing links. This parameter research tries to answer research questions such as: How do framed tube structures without a rectangular floor plan behave with respect to shear lag? and describes the relation between parameters such as aspect ratio, slenderness and corner stiffness on the shear lag effect and other structural aspects.

It answers these questions by changing the parameters of a basic structure in the Finite Element Method (FEM) program AxisVM. This structure has a floor plan of 14.4m × 14.4m and has 25 floors with a 3.5m floor height, adding up to a total height of 87.5m and a slenderness just over 6. All models are made in 4 different configurations: a cast-in-place configuration, a prefabricated configuration with a masonry configuration for the elements and two similar prefabricated configurations with small and large windows. Two parameters are also formulated in this section to keep track of the shear lag effect: These are the shear lag factor P, for flange walls and the shear lag factor $P^*$ for web walls.

All these previously mentioned characteristics, except for the floor plan, are used for the answering of the research question on non-rectangular structures. The conclusions from this part of the parameter research are that the changes in shear lag effect are small if inside corners or corners with an angle not being 90 degrees are used. For structures with the floor plan of higher polygons, such as octagons and hexadecagons improved structural behavior was found. The reduction in peak stresses in the corners can be attributed to the shorter elements where shear lag can form and the more corners over which the peak stresses can be divided.

For the parameters researched using the square floor plan it was found that both the aspect ratio (the ratio between width and depth of the structure) and the introduction of structural vertical joints between the elements in masonry configuration had no to a negligible influence on the structural behavior. The effect of the corner stiffness on the structural behavior was more profound: especially reducing the corner stiffness has a noticeable negative effect on co-operation between the flange and webs walls, which in turn increases the deflection at the top of the structure. For the relation between the shear lag and the slenderness of the structure it was found that structures with a lower slenderness, that depend more on their shear stiffness to resist lateral loading, experience more shear lag than structures with a higher slenderness.

In the second part the following research question was answered: Is it possible to build a structurally feasible prefabricated tube-in-tube high-rise structure in the confines of the Dutch building market of approximately 200m high and how does it compare to other prefabricated solutions?

In the literature report a building was described that could be used to answer both parts of this research question. The Zalmhaventoren is a proposed residential shear wall high-rise structure in Rotterdam which has
been researched by ten Hagen for a Master’s thesis at Delft University of Technology. In this thesis, the original cast in-situ concrete Zalmhaventoren is redesigned to a prefabricated shear wall structure that is 201.3\(m\) high. A redesign of this structure gives insight in the possibilities of a fully prefabricated concrete structure and has a model that as comparable to the model researched by ten Hagen.

During the redesign the basic dimensions of the original structure were kept. The floor plan of the structure is 30\(m\) \(\times\) 29.5\(m\), has a core of 14.4\(m\) \(\times\) 12\(m\) and the floor to floor height is 3.05\(m\). Although originally designed as a residential structure, this floor height is also sufficient for other functions, such as an office function. To keep the structure comparable to the one designed by ten Hagen, 66 floors were used to reach the total height of 201.3\(m\). Besides these basic dimensions however, significant changes were made to convert the structure into a prefabricated tube-in-tube structure. The removal of the shear walls and addition of the lateral load resisting façades changed the floor plan of the structure. To give the tube the stiffness that is required when loaded by wind, extra material was added in the corners of the structure, where windows were present in the shear wall design. Also an extra column was added in the center of the frame for the largest balcony on each side of the structure. In the lobby of the structure similar changes were made to fit the new structural system of the building.

Another important part of the structural design was the determination of the stiffnesses of the different connections between the elements. Especially for the horizontal connections in the tube of the structure this proved difficult, because of the differences in normal stress between different parts in the same element. A 2-level stepped stiffness was given to these tube elements to approximate the actual stiffnesses in this type of connections. For the corner connections (of the Interlocking Halfway Connection type) the stiffness was determined for the connection in compression and in tension. An iterative calculation ensured that the correct stiffness was attributed to the correct corner connection.

After the model was made, the results were analyzed. Probably the most important result, for it is often governing in high-rise structures, is the deflection at the top of the structure. For a structure of this height, the maximum allowable deflection is \(w_{\text{max}} = 403\text{mm}\). For the four different wind directions, the deflection was between 282\(\text{mm}\) and 327\(\text{mm}\). Also the shear stress between elements was checked and found to be within the limits set in the Eurocode. For two elements an estimation of the reinforcement was made. The reinforcement of the lintel was viable, but for the column near the lobby of the structure, a reinforcement percentage was found that was higher than the recommended maximum value in the Eurocode. However, if the integrity of the concrete can be assured or the less strict Dutch national annex is used, the found percentage does not interfere with the feasibility of the structure.

Along with the main model, some alternate models were made to research the effect of certain aspects on the most important result: the top deflection of the tower. The first one was the difference between a cast-in-place model and the prefabricated one. The increase in top deflection for the prefabricated structure was around 8\%. Not negligible, but not high enough to stop considering prefabricated tube-in-tube structures from being viable. The second check was if removing the balconies, and with them the less stiff frames around the balconies, and replacing them with ordinary stiffer windows had a positive effect on the top deflection. It was found that implementing this change at the same element thickness reduced the top deflection by 34\% to 42\%. Even when the thickness of all the elements was reduced from 400\(\text{mm}\) to 300\(\text{mm}\), the top deflection of a structure without the balconies was better than for the model with balconies and a element thickness of 400\(\text{mm}\).

In the comparison with the structure from the work of ten Hagen, it became clear that both lateral stability resisting systems could work for the Zalmhaventoren. Both had advantages and disadvantages that could be weighted either way by different developers if the structure was designed purely as a residential building. However, if the tower was designed as being a multipurpose building, the open floor plan of the tube-in-tube structure provides not only more flexibility at completion, but especially over the entire lifetime of the structure. Ultimately this research gives insight into the structural behavior of the prefabricated tube-in-tube structure and proved that the possibilities of the prefabricated tube-in-tube structure lie even beyond the 200\(m\), especially if the technologies associated with this stability system keep improving over the coming years.
1 Introduction in prefabricated high-rise and framed tube structures

This section is an introduction to the basic aspects of the subject of this thesis. An introduction into high-rise structures is given, along with the place of framed tube structures within this field. These parts are based on [12] and [27]. An introduction into prefabrication is also part of this section, briefly explaining aspects as the fabrication, transport and common connections of wall elements.

1.1 High-Rise and framed tube structures

High-rise structures differ from other structures in a very observable way, they are higher than ordinary buildings. Because high-rise structures are higher, several aspects gain a more influential role. First of all there are the vertical loads in the structure. Because there is a lot of floor area on a relatively small footprint, the forces in the lower parts of the structure become very high. This is also the case for the foundations of structures of this kind.

However, the higher vertical loads are not the biggest challenge in building high-rise structures. The lateral loads, mainly due to wind forces on the building are the biggest challenge a high-rise structure has to overcome. A high-rise structure can be modeled as a clamped beam under lateral loading. Not only do the wind forces themselves increase at higher altitudes, but also the lever arm between these wind forces and the foundations. This means that the moments near the end of the modeled beam are very high, and a high stiffness is required to cope with these moments and the forces accompanied by them. To deal with these and the many other challenges accompanied with building a high-rise structure it is very important that professionals from all disciplines are involved at an early design stage. Only in this way, a good integrated design can be achieved.

1.1.1 Brief historical context

The construction of high-rise structures started at the end of the 19th century in Chicago. After a grand fire destroyed a large part of the city, the city had to be rebuilt. A combination of technical innovations such as the elevator and the long for prestige of engineers and investors drove the height of the new buildings up. In 1885 the first real high-rise building was constructed: the Home Insurance Building, reaching a height of 55\(m\). The structure of this building consisted of a steel skeleton of columns and girders which formed a rigid frame to cope with the high lateral forces. Several other buildings were constructed in Chicago as well with the same structural system. However, in 1891, a building was constructed using an old, more massive building method. The Monadnock Building was a 16 story high masonry building. To resist the high forces at the bottom of the building, the walls were almost 2\(m\) thick. Such wall thicknesses and the limited window sizes proved fatal for this building method, which was discontinued after the Monadnock Building. The impossibility to build higher using older building methods, combined with the desire to build higher structures in cities other than Chicago gave the steel frame building method a boost which spurred the maximum height to new levels.

In the first 3 decades of the 20th century the high-rise structures really reached for the sky. Within this period the new structures became taller very rapidly. The main city in which this happened was not Chicago, but New York City. An important step towards higher construction was the Woolworth Building. Completed in 1913, it stood 241\(m\) tall and became the tallest structure on the planet for 17 years. In 1930 the Crystler building took this honor away from the Woolworth building, reaching a height of 320\(m\). Only to be beaten by the Empire
State Building (381m) one year later, which stayed the tallest structure for over 40 years. All these record breaking structures had the same structural principle and consisted of a steel frame structure. In the 1950’s Mies van der Rohe was a vastly influential figure in architecture. His ideas caused an architectural shift which led to the development of not only architectural but also some new structural concepts. One of these new structural concepts was a very efficient way to cope with lateral forces by using the outer façades not only to transfer vertical loads to the foundation but also resist the horizontal wind loads. The tube structure was born and people recognized the advantage of not having to put stabilizing shear walls or other reinforcement elements in the plan of the building, making such a structure very versatile.

In the late 1950’s Hubert Beck in Germany published his work which led to the recognition of the core walls being a stabilizing element in a structure. This eventually led to the improved stability system of the tube-in-tube, in which a stabilizing façade and a stabilizing core work together to resist the lateral forces on a high-rise structure. But even before the tube-in-tube structure, his calculation method of shear walls initiated a start of a concrete high-rise culture in Frankfurt and other cities in Europe.

The next chapter in the race for the highest structure was not a concrete structure though, nor was it constructed in Europe. In 1972 the two towers of the World Trade Center in New York City were built, each reaching an incredible height of 417m. The stability structure of these Twin Towers was a steel tube structure. Steel columns were very closely spaced in the entire perimeter of the building, creating a dense tube which resisted the lateral wind forces on the towers. Only two years after their completion however, the record for highest building changed cities within the United States. Chicago, the city where the race for the highest structure started, took back its title by means of the Sears Tower (it is currently named: Willis Tower). The tower was 442m high and used a new structural system called a bundled tube. In this structural system, multiple smaller tubes are combined to form a lateral stability system. Using this system, it is easy to discontinue parts of the building (and as a result, also the stability system) towards the higher levels where less lateral stiffness is required. In this way, a very efficient structure is made.

The Sears Tower was the last building in the United States that held the title for tallest structure in the world. It can also be considered the last structure that had a single lateral stability principle, because all the towers that broke records after the Sears tower consisted of a combined system. From 1998 to 2003, the Petronas Towers in Kuala Lumpur, Malaysia, were the tallest structures in the world, with both towers reaching a height of 452m. Hereafter, the Taipei 101 in Taipei, Taiwan was the tallest in the world. This 509m tall structure was built in an very hostile environment for a high-rise structure, with Taipei being earthquake and typhoon prone.

Figure 2: The Twin Towers of the World Trade Center in New York City (Courtesy of wikipedia.org)
To cope with these extra horizontal forces, a mass tuned damper was constructed near the top of the structure. The tallest structure in the world at the moment of writing of this report is the Burj Khalifa, which did not just take the record from the Taipei 101, but smashed it by reaching a height of 828\,m. The tower in Dubai will not keep its record very long however, because another city in the middle east, Jeddah, Saudi Arabia, is constructing a tower that will break the 1000\,m mark. The Kingdom Tower should reach a staggering 1007\,m when it will be finished, practically doubling the height of the tallest structure of a decade before it.

1.1.2 Structural Behavior of Framed Tube Structures

The façade elements of high-rise structures had always been used solely to divert vertical loads to the foundation before the structural system of the framed tube was devised. In the framed tube stability system however, diverting the horizontal loading to the foundations became a key role of the structural elements in the façades. The idea behind the structural system is that every façade is a stiff element in its own plane. This is usually achieved by a rigid frame of closely spaced columns and deep spandrel beams, but this can also be achieved by jointed walls or diagonals. Considering a basic rectangular floor plan, these four elements are connected orthogonally rigid in the corners of the structure, forming a tube in the three dimensional space. The structure will now perform as a tube, having two web façades in the wind direction and two flange façades perpendicular to the wind direction. The reason why this system performs so well against lateral loads is the combination of different factors. First of all, the flange façades are activated due to the tube action of the structure. Secondly, because the stabilizing elements are at the outermost positions in the structure, the entire available lever arm is used to take up the lateral loading. Although the structure created by the combination of façade elements represents a clamped rectangular tube, the stress distribution is not the same. A clamped rectangular tube (made from an idealized elastic material) under lateral loading has a linear normal stress distribution in the web elements and a uniform normal stress distribution in the flange elements. For a framed tube structure however, this is not the normal stress pattern that is observed. Because the elastic continuity of a framed tube structure consisting of columns and beams

![Figure 3: The distribution of the normal stress on the bottom floor, in a tube structure without (left) and with (right) the shear lag effect (Courtesy of [12])](image)

is linked to the shear rigidity of the frames the beams and columns make, the total rigidity of a façade and therefore the entire tube is lower. This results in lower axial stresses in the middle columns of the web façades. This effect is compensated for by higher stresses near the corner of the building, which themselves lead to a non-uniform stress distribution in the flange façades. These stress distributions can be seen in Figure 3. This effect is called shear lag and is an important aspect when designing a framed tube structure. The lateral forces in a framed tube structure are transferred to the foundations by two different components. The first component is the bending deformation caused by the lengthening of the columns on the windward side.
and the shortening of the columns on the leeward side of the building. A part of the lateral load is transferred to the foundations in this way. The other component is the shear deformation of the structure. The shear forces resulting from the horizontal loading on the structure are taken up by bending of the columns and beams in the web façades of the building. Therefore additional deformations of the entire tube are present. The highest loads in a framed tube structure occur as axial loads at the corner columns at the bottom of the structure. Because of the vertical loads, the horizontal loads and the additional loading due to shear lag these elements are loaded with forces that are a multiple of the stresses associated with just the vertical loading. To prevent these elements from getting too massive and causing trouble in creating a good functional design of the lower floors, additional measures can be taken to relieve the framed tube from some of the lateral loading. These measures can include a stiffening core (creating a tube-in-tube structure), additional shear walls or diagonals in the façades. In a reaction on the regular shear lag effect another effect is found in framed tube structures. Negative shear lag [26] is an effect that has the opposite effect of the positive shear lag effect, because it reverses the highest normal forces in the flange wall to the center instead of the corners. This effect takes place because of the differences in vertical translation induced in the lower part of the structure where positive shear lag is present. The concave upwards vertical translation profile induces a reaction which has a convex upwards profile. In the higher regions of a framed tube structure this effect can be dominant over the positive shear lag.

1.2 High-rise structures in prefabricated concrete

High-rise structures are constructed in different materials. Structures in steel, in concrete and combinations of these materials are all suitable to create structurally and financially feasible structures. The choice for a material is influenced heavily by culture. For instance, in the United States steel construction is most commonly used and in Asia and the Middle East cast-in-place concrete is the most commonly made choice for a main building material.

For prefabricated concrete high-rise structures however, there is no region in the world where a culture for that material prevails, at least not yet. However, in the Netherlands prefabricated concrete is on the rise. In the last decade increasingly high prefabricated concrete structures or structures with prefabricated concrete elements were built, mostly in The Hague and Rotterdam. Some of these structures can be found in the section on relevant buildings. This section will describe several aspects of prefabricated concrete high-rise structures. First of all the advantages of the material will be described. After that, a section will be dedicated to common connections that are used in shear wall structures. Lastly some requirements for floor systems which fit the prefabricated building method will be described.

1.2.1 Arguments in favor of building in prefabricated concrete

There are a number of arguments in favor of building a high-rise structure in prefabricated concrete. The most important one is that a faster construction is possible. The reason behind this faster construction, compared to cast-in-place structures, is that there is no waiting time between floors. When a story is constructed from prefabricated elements, the elements are at their full strength almost immediately. For cast-in-place structures the concrete first has to cure, the formwork removed and placed one floor higher before construction on the next floor can start. The preparation time of prefabricated construction however, in which the elements are engineered and produced can be longer than for traditionally built structures. As described in the thesis of van der Meij [32], there is a potential to construct buildings even faster when using prefabricated methods. Because there is no concrete curing, a building could be constructed around the clock for an entire week instead of the five 8 hour days that are the standard in the Netherlands. The reason why a shorter construction time is so beneficial for a project mainly has to do with economics. Project developers have to borrow money to finance projects. Especially a high-rise structure demands a large loan because of the high prize of a large structure. The developer will start making money at the moment he can rent out the square meters in the structure. If the structure is completed earlier, a developer can make back its money earlier so he has to pay less interest on his large loan. These benefits can outweigh the currently higher initial costs of making a prefabricated concrete structure instead of a traditionally build one. Another beneficial aspect of a shorter building time is that people living near the construction site will have a smaller amount of time in which they might experience noise and other construction related disturbances. Another argument for construction in prefabricated elements is that there is less labor necessary on site. One can see this as a financial gain, but there are other aspects at play here as well. Especially if the construction of a high-rise building nears its end, vertical transportation becomes a critical factor in construction. Not only the vertical transportation system of materials, but also the vertical transportation system of people will reach its maximum capacity, slowing down construction. But even before the higher levels are reached less people on the building site will make the work of a site manager easier and the possibility of people being in each others way less likely.
The quality of concrete depends on the conditions under which it is cast. For cast-in-place structures the concrete quality can differ throughout the structure, because the conditions under which the concrete is cast may vary due to for instance the weather conditions. An argument to use prefabricated concrete is that production of the elements takes place under controlled circumstances in a factory, decreasing the chances of faulty concrete anywhere in the structure. Also for services that could be incorporated within the concrete, such as ducts and cables, production in a controlled environment might decrease the risk of mistakes.

The last argument for the use of a building method using prefabricated concrete is that a smaller building site is possible if the logistics are handled well. Because construction of high-rise structures usually takes place in dense urban areas, making room for a construction site is always challenging. If the structural elements are transported to the building site at exactly the time they are needed, a method called Just-in-Time (JIT), no storage area for large elements has to be present on the building site. This reduces the area needed for a building site. And as earlier mentioned, square meters for a construction site in a city center are hard to come by.

1.2.2 Common connections and suitable floor systems

The main difference between a cast-in-place structure and a structure consisting of prefabricated concrete elements is the discontinuity of the concrete in the latter one. Because of this discontinuity joints appear in the structure. The characteristics of these joints are important to the structural behavior of the structure. First the vertical joints will be discussed. For vertical joints there are two main types, structural joints and non-structural joints. These two types are dependent on the configuration of the elements of the wall these elements are in. In a vertically stacked configuration, continuous vertical joints appear from the bottom to the top of the wall. When this wall is loaded laterally, shear stresses have to be taken up by these vertical joints. So for this configuration structural joints have to be present. In a masonry configuration however, there are no continuous vertical joints and the shear stresses are taken up by the elements themselves. Here, non-structural joints, which are easier and cheaper to construct, will suffice.

For horizontal joints not many different types exist. They basically all use the same design, which can be changed to fit the requirements by using a little more or a little less reinforcement or using mortar of a higher or lower quality. The design consists of protruding reinforcement bars at the top of each element and gaines in the bottom of each element. During installation, the protruding bars are placed in the gaines of the next element. Hereafter, the recesses and the approximately 20mm of space between the elements are filled with mortar to connect the elements together and obtain some axial and shear stiffness in the horizontal joint.

All structures, so also prefabricated high-rise structures, need floors in them. Many different floor systems are available in the market for different kinds of applications and conditions. In prefabricated high-rise construction,
some important aspects have to be taken into account when choosing a floor system.

- Because faster construction is one of the benefits of prefabricated construction, a floor system which fits in the same rapid construction philosophy could be used to keep the construction time to a minimum

- In high rise construction a large number of floors is present. Reducing the weight of the floor system reduces the overall weight of the structure and stresses near the bottom of the structure

- In framed tube and tube-in-tube construction, the floors can play an important role by transferring the lateral loads in their plane to the different lateral resisting elements of the structure

Some floors systems that could be considered because they fit at least some of the properties mentioned above are hollow core slabs with a top layer, concrete planks and weight saving two way spanning floors such as the Bubbledeck system.
2 Relevant Research

During the literature study of the thesis, articles, theses and other sources were found that were relevant to the subject of this thesis. For this section the sources are divided in Master’s theses from Delft University of Technology and other sources. For the most research specifically done to prefabricated concrete high rise was conducted in Delft. For more sources and more extensive information on the described sources Appendix A, the literature report, can be regarded.

2.1 Delft University of Technology Master’s Theses

The oldest thesis considered relevant for this research is the work of Faessen [9]. In this thesis made in the year 2000, design rules for concrete framed tubes were determined using a Finite Element Method (FEM) program. Parameters such as the building height, floor plan, beam height and column width were examined and their influence on the deflection and forces in the corner column put into equations. Some conclusions with respect to prefabricated framed tube structures are quite harsh, stating that a framed tube structure could never be higher than 30 stories. These conclusions have been proven to be unlikely by other research done after this one. The next thesis considered relevant is the work of Prakoso [22], which is a continuation of the work by Faessen. Both theses have the same title and the end result of both are sets of design rules for framed tube structures. The work of Prakoso however has a greater range for the parameters for which the design rules are applicable and has more reliable conclusions about framed tube structure from prefabricated concrete, stating that structure could be as high as 50 floors.

A very important thesis has been done by Falger in 2002 [10]. The title of this thesis translates to Prefabricated Concrete Stability Structures with Open Vertical Joint in Masonry Configuration and was performed in collaboration with the Stufib Study Association. In an extensive literature study he found some important design rules for stability structures with open vertical non-structural joints in the works of J.G.A. Snelders, TU Eindhoven, 1994 and J.W. van Dorst, TU Delft, 1995, which are:

- The better the non-structural vertical joints are distributed over the width of the wall, the closer the force distribution is to that of a cast-in-place wall
- The positive effects of a masonry configuration is lost if the overlap between elements in different layers is less than 25% of the element width
- To make the elements work together the width of the smallest element should not be smaller than the height of the element

After his literature study, Falger continues with a parameter study in which he has investigated four different wall types with a different amount of openings in the wall. These types are constructed with 3 different configurations, a cast-in-place configuration, a prefabricated wall with a vertical configuration and a prefabricated wall with a masonry configuration. From these different wall types and configurations, Falger has found several interesting conclusions, the most relevant are:

- The masonry configuration leads in all cases to less than 10% increase in top deflection of the wall. These values are comparable with the stiffest joint type and the vertical configuration
- Because these differences are quite small, it is acceptable to calculate prefabricated shear wall structures with a cast-in-place calculation in the design phase of a structural calculation
- The shear stresses that are usually taken up by the vertical joint are in the case of a masonry configuration taken up by the elements below and above the non-structural joint. The increase of shear stresses percentage wise is significant, but the stresses remain relatively low in absolute value
- Although the deflections are comparable to the cast-in-place configurations, the internal forces are not. Especially for the perforated walls the shear forces can be significantly higher.

In addition to these conclusions other conclusions can be found with respect to the different vertical joint stiffnesses in the vertical configuration.

In the work of de Boer [8] the masonry configuration is compared to the vertical configuration, as was also done in the research by Falger, but from a construction site perspective. Briefly, it describes that the joints in a masonry configuration, which are non-structural and easier to make, are a cheaper and in some cases faster way of constructing prefabricated shear wall structures. In combination with the work by Falger, this led to the choice of only using a masonry type configuration in this thesis.

Pieterse [20] investigated whether in a shear wall with a vertical configuration the edge beam and floors are beneficial to the lateral stability. Although this method is proven to have potential for low-rise structures, it...
is unfit for high-rise construction because the increase in top deflection is significant and top deflection is often governing in high-rise construction.

A more relevant study has been done by Tolsma [30]. His thesis titled Precast Concrete Cores in High-Rise Buildings investigates the applicability of prefabricated concrete core structures. He states that a prefabricated core is a viable alternative to a cast-in-place core, being only slightly less stiff. In order to create a prefabricated concrete core, the elements have to be connected to each other through the corner connections. Tolsma investigates three different types of corner connections, as can be seen in Figure 6. One of these connections (IACC) is deemed unfit for this use. The other two do perform properly in the research by Tolsma, with the Interlocking Halfway Connection (IHC) being slightly better. Therefore, this corner connection was used throughout this thesis. These connections are not only for use in core structures, but for all similar connections, such as perpendicular walls in a framed tube structure. This makes this thesis in particular relevant for a research into prefabricated tube-in-tube structures.

The most recent thesis relevant to this thesis is the work of ten Hagen [29]. In this thesis ten Hagen redesigns a high-rise tower in Rotterdam, the Zalmhaventoren, to a prefabricated concrete structure. The stability system of this tower are internal shear walls in both directions of the tower. He also goes into detail with respect to the construction of such a tower in order to find out whether such a tower is logistically and structurally possible. Shortly, this question can be answered positively. Interesting about ten Hagens thesis is that he also works with the FEM-program AxisVM and has investigated and calculated several connection types relevant to this thesis as well. Some of his conclusions are:

- The stiffness of large elements in a masonry configuration is only marginally (4%) smaller than a cast-in-place wall
- The horizontal connections behave as stiff as the surrounding concrete due to the high normal forces
- The shear stiffness of the horizontal connections and the stiffness of the vertical connections in a masonry configuration both do not influence the behavior of the structure.
- The distribution of shear forces is vastly different between a cast-in-place wall and one with a masonry configuration
2.2 Other sources

Most of the other sources found relevant to this thesis are from regions of the world where building tall is more common than in the Netherlands, but building prefabricated structures is never considered, such as Asia or the United States of America.

In the work of Shin et al. [25] a parameter study was done on a "concrete frame wall tube building", which could be described as a tube-in-tube structure in which only two walls are perforated. For these two perforated walls the column depth, column width, beam depth and beam width are varied to examine the changes in structural behavior of the 174m tall case study building in New York City. Some of the most important conclusions can be summarized as follows: The tube action of a high-rise framed tube structure can best be improved by increasing the column depth and shear lag can be reduced by increasing the beam depth. Increasing the column width is recommended only for reducing the overall deflections of a building. Lastly, the beam width has the least influence on the lateral force resistance of the four examined parameters.

In two articles by Kung-Kun Lee et al. [18] [17] the structural behavior of tube(s)-in-tube structures is examined. These articles provide insight in several ways, including a distribution of positive and negative shear lag over the height of a structure, a unit for measuring shear lag (P), a parameter study on the influence of the stiffnesses of the columns and beams on the structural behavior and a new method for calculating tube(s)-in-tube structures without a FEM-program. This was the first article about negative shear lag found in literature, although it is better explained by another article [26]. Furthermore, the shear lag factor P is used extensively as a measurement of shear lag in the parameter research of this thesis.

Another article in the literature report has been written by D.C. van Keulen (a committee member of this thesis) and J. Vambersky [36]. The article is titled Design and Displacements of Precast Concrete Shear Wall Structures and is very relevant to this thesis. Focused on shear walls in prefabricated concrete, the influence of several aspects is researched, such as:

- Stiffness of the horizontal joints
- Element size and lay-out
- Element openings
- Slenderness of the wall

Summarized, the conclusions are as follows: The stiffer the horizontal joints, the bigger the elements, the smaller the holes and the more slender the structure, the less is the percentage wise deformation increase of a prefabricated shear wall compared to a similar cast-in-place shear wall structure. The last article in the literature report has also been written by D.C. van Keulen and its title translates to FEM in prefabricated concrete [35]. In this article the important aspects of modeling a prefabricated concrete shear wall in a FEM-program are discussed. The most important information with regard to this thesis is the calculation of the normal and shear rigidities between elements in shear walls. Although the normal stiffness is rather straightforward, the shear stiffness is more complex. Van Keulen describes five aspects that contribute to the shear stiffness:

- Adhesion between the materials
• Resistance due to the roughness of the interface
• Friction due to the normal stresses
• Friction due to additional normal stresses because of the extension of the reinforcement
• Dowel action of the reinforcement

Because these different modes appear for different connection slips the Eurocode equation is adapted for two different cases: one with strong adhesion and one with weak adhesion. The corresponding connection slip is also given.

Figure 8: Different factors contributing to the shear stiffness of a joint. On the left the adhesion and friction normal stress, in the middle the dowel action of the reinforcement and on the right the friction due to the extra normal force because of the extension of the reinforcement (Courtesy of [35])
3 Prefabricated High-Rise Structures in a Dutch Setting

For this thesis the Dutch construction market is chosen as a geographical setting. The reason behind this choice does not only lie in the nationality of the writer and the university for this thesis, but also in the location of existing prefabricated high rise structures. In this section framed tube and prefabricated high-rise structures already in existence will pass in review to give an image of the current state of the Dutch trends in both categories. Another subsection deals with the justification of building tall structures in the Netherlands.

3.1 Existent Buildings

For a more extensive look into these structures one should look into the Appendix A, which contains parts of the literature report associated to this thesis.

3.1.1 Het Strijkijzer, Den Haag

At the time of its construction the "Strijkijzer" in The Hague was the tallest fully prefabricated structure in the Netherlands. The building is a mostly residential tower with a triangular plan consisting of a continuously L-shaped part and two sections where the ends of these wings are connected. It is 42 stories high and has a height of around 132 m. The stability of the structure comes from shear resisting walls in the circumference of the structure as well as from shear resisting walls between residential units. Originally designed as a in-situ cast structure, somewhere along the way the choice was made for a faster (but slightly more expensive) building method. By reducing the construction time from 32 to 20 months this increase in manufacturing costs was beneficial for the overall financial result. The Strijkijzer is a good example of how changing from a cast-in-place structure to a prefabricated structure does not influence the top deflection all that much and that interlocking prefabricated elements with non-structural vertical connections are well suited for high rise construction.

3.1.2 JuBi Towers, Den Haag

The JuBi towers are two towers in the city center of The Hague housing the ministry of Security and Justice and the ministry of the Interior and Kingdom Relations. Both towers are 140 m high and consist of 41 stories. The construction was finished at the end of the year 2012. The stability system of both towers is a tube-in-tube system consisting of an irregular shaped floor plan and several cores. The outside tubes are constructed in prefabricated concrete, the cores are constructed using a climbing formwork and the floors are made with the Bubbledeck system. The prefabricated elements in the tube are stacked in a masonry configuration and the panels are connected by welded steel plates in the lintels of the elements. The complex shape of the floor plan, the setbacks in the façade and the limited space for a construction site all made this project a real challenge for all partners involved in the construction.

3.1.3 Erasmus MC, Rotterdam

The Erasmus MC in Rotterdam is a good example of an existing prefabricated concrete tube-in-tube structure in the Netherlands. The tower is part of a university medical center and hospital and was constructed because space on site was limited and a lot of new floor area was required. The tower is 31 floors high and has a height of 120 m. The tube elements consist not only of the load bearing part of the structure, but is incorporated into a sandwich panel which consists of the façade skin as well. Using this kind of panels sped up the construction time. The elements are stacked in a masonry configuration. For the construction of the tower a hoisting shed was build. The hoisting shed used is a contraption which functions as a crane and working area at the same time and is able to lift itself on top of the structure. Because the hoisting shed protects the workers and the precast elements from the weather outside and fewer working days are lost, which results in a faster construction time for the structure.

3.1.4 Maastoren, Rotterdam

The Maastoren in Rotterdam is the tallest building in the Netherlands. It reaches a height of 165 m and the construction was finished in November of the year 2009. In the 44 floor structure the lower 30 floors have a floor plan shaped as a capital letter H, with one of the wings kinked in order to line up with the river Nieuwe Maas. For the top 14 floors only one of the two towers is continued. The stability system of the structure is a combination of an cast-in-place core and prefabricated concrete load bearing façade around the entire perimeter of the structure. The two lateral stability systems are linked together by walls on each floor of the structure. The elements in the façade are stacked in a masonry configuration and for the corner connections a staggered connection (SC) is used. This structure is very relevant to this thesis because of its (partly) prefabricated tube-in-tube structure with a masonry configuration and irregular shaped floor plan.
3.1.5 First, Rotterdam

First is a building currently under construction in the city center of Rotterdam, near the central train station. When it will be finished the predominantly prefabricated concrete structure will reach a height of 125 m. Just as the Maastoren, the floor plan of First will be shaped as a slightly modified capital letter H. In which the connection between the two wings will house a core that has a function in both stability and vertical transportation. However, the stability system of this structure is not a framed tube or a tube-in-tube solution. Both these systems where looked into by the engineers of the project, but boundary conditions with respect to the façade and the openness of the lower levels prevented both these options to be applied to this structure. Instead, an outrigger system was used to provide lateral stability for the structure. This makes First a good example in which a tube-in-tube system does not match the conditions of a structure.

3.2 Building tall in the Netherlands

One could see the choice of the Dutch market as a limitation of the height of a structure, because culture, economic situation and regulations in the Netherlands have not produced structures higher than 165 m.
regulations in the Netherlands, compared to for instance the United States or the United Kingdom, are stricter with respect to daylight in the workplace. As a result, an office can be at most 7.2m from the back wall to the window. In an article [33], van der Windt makes a quick calculation for a structure in this context. He estimates the maximum width of a structure is composed by the following elements from one façade to the other:

- 7.2m office
- 1.8m corridor
- 9.0m core
- 1.8m corridor
- 7.2m office

This adds up to $7.2m + 1.8m + 9.0m + 1.8m + 7.2m = 27m$. He further argues that with this information, in combination with an estimated slenderness for which economical construction is possible, the maximum height of a Dutch building can be found. With the estimated economical slenderness of 8, this maximum height is: $27m \times 8 = 216m$.

But regulations are not the only aspect of the Dutch construction market. The economical situation also constrains the construction of high-rise structures. As described by Koster [16], the existence of high-rise structures used to be explained only by simple economic theory. The explanation is as follows: At a popular site the price of land is very high. To be able to still make a profit a high density structure has to be constructed on a small plot. Building tall is the most logical way to achieve this high density per land area. One could think that using this theory on the Dutch building market, a height could be found which is both structurally and economically feasible. However, on a congress of Stichting Hoogbouw [19] (High-Rise Foundation) the problem of this theory for the Dutch market is explained. The only location in the Netherlands where the price of land is high enough to solely justify high-rise construction is in the city center of Amsterdam. However, because the city center is a UNESCO World Heritage site such construction is neither allowed nor desirable. Outside of the city center, or in a city such as Rotterdam without an old city center, the price of land is not high enough.

If high rise in the Netherlands cannot be explained by simple economical theory, something else has to play a role, for there are high rise structures in the Netherlands. Koster explains that in the Dutch market, companies are willing to pay a premium of 4% per 10m of extra building height. The reason why companies are willing to pay more is explained by 3 factors:

- The view effect
- The landmark effect
- Within-building agglomeration economies

The first two are simple enough to understand. To be able to see the city from a nice perspective and to be seen by the city are both worth the investment for a company. Within-building agglomeration economy is about workers being more productive in a high-rise structure because of the high worker density and the knowledge spillovers that are more likely to happen than in smaller scale working environments.

Another research also implies that there is more to high-rise construction than simple economic models. Helsley [13] explains that these models do not account for the “inherent value placed on being the tallest” that seems to exist based on history. He explains his theory both by analyzing the race for the tallest building in the world in the 20th century and by stating that the standard economic model makes predictions that are not seen in reality. There are three predictions that are described in this article:

- Because of greater agglomeration economies, the largest cities should have the tallest buildings
- Areas near tall structures should contain buildings of similar heights, because the conditions between lots are not that varied
- All tall structures should be economical

All three of these predictions have been proven untrue to a certain extent. The tallest structure has not always been in the largest city, sometimes a tall structure is significantly taller than its closest rival from the same city and often the tallest structure in the world has not been economical. Lastly, Helsley adds that not just being the tallest in the world holds value, but also being the tallest of a region, country or city.

Taking this literature in account, a clear picture results for the state of high-rise in the Netherlands. Because a high-rise structure cannot be justified from economic feasibility alone within the Dutch building market, other aspects have to be taken into account. And as has been described by both Koster and Helsley, there is value in being tall, or even the tallest. Going forward in this thesis it is good to keep these factors in mind.
4 Missing Links and Research Questions

In this section some aspects related to prefabricated concrete framed tube high-rise structures will be described, which are not present or not very profoundly described in current literature. These are called missing links in research and these aspects will lead to research questions which can be answered in the continuation of this Master’s thesis.

4.1 Shear lag effect of non-rectangular framed tube structures

Shear lag is an important effect present in framed tube structures and definitely has a big influence on the overall force distribution in a building. Because of its importance, its workings and effects on framed tube structures are described in literature, even in literature which gives a more global overview of high-rise construction, such as [12] and [27]. Some more applied information can be found in the master’s thesis of Balbaid [7] or the research of Shin et al. [25]. In these works the results of the effect calculated by a FEM-program are discussed and the effects of certain parameters (column and beam dimensions, concrete class) on the shear lag effect are researched. In the work of Shin et al. negative shear lag is addressed as well, something which lacks in discussions on the topic. Furthermore, the effects of shear lag of tube-in-tube structures is not mentioned in all previously stated literature.

The research done by Lee et al., [18] and [17], was an already more intricate work on the effect of shear lag. A parameter study using a FEM-program was used to describe the effect of the stiffnesses of different elements in the structure on the magnitude of the shear lag was done. But more importantly, a calculation method was given which can be used to describe the effects of both positive and negative shear lag in a structure without the need of a complicated FEM-model. Furthermore, this method allows for one or more interior cores to be present in the model as well, giving engineers more opportunities to use this method and model the interior cores accurately in an early design stage. This method however cannot be used for non-rectangular and especially non-symmetric floor plans.

For extra information on negative shear lag a reference of Shin et al., and Lee et al. was included in the literature study. This research [26] gives an explanation on the workings of negative shear lag and the influence of some non-dimensional parameters on the positive and the negative shear lag. Again, only rectangular floor plans are considered.

The problem description of this missing link can therefore be described as:

Shear lag is an important effect in framed tube high-rise structures. However, its effect is described solely for structures with rectangular floor plans. Because of this, engineers creating structures with non-rectangular floor plans cannot make estimations of the shear lag effect in an early design stage. They can only discover its effects after analysis of a 3 dimensional FEM-model, which is very time consuming to make.

The accompanying research question is the following:

How do framed tube structures without a rectangular floor plan behave with respect to shear lag?

This research question can be divided into several sub-questions, stating the focus of the shear lag research of non-rectangular structures.

- What is the effect on the shear lag if more sides are added to a square floor plan, approaching a circle?

![Figure 11: Square floor plan transforming into a circular floor plan](image)

- Which shear lag behavior can be found in structures with inside corners?
- What is the influence of corners with an angle different than 90 degrees?

4.2 Structural behavior of prefabricated framed tube structures

In addition to the non-rectangular framed tube structures, missing links can be found too in the subject of the more common rectangular framed tube structures, as long as the structures are from prefabricated concrete. For prefabricated structures most of the recent research is into shear walls in a 2 dimensional plane, such as
Falger [9] and van Keulen and Vambersky [36]. Research on 3 dimensional structures has had different structural systems as a subject, such as prefabricated cores (Tolsma [30]) and a stability system with perpendicular shear walls (ten Hagen [29]). This can be defined as a missing link in research and therefore produce some unanswered research questions:

- **What is the influence of the aspect ratio (ratio between width and depth of the building) on the shear lag factor in a prefabricated concrete framed tube structure?**
- **What is the influence of the slenderness on the shear lag factor in a prefabricated concrete framed tube structure?**
- **Does changing the non-structural joints in a masonry configuration to structural joints for a framed tube structure have a positive influence on the structural behavior?**
- **What is the influence of the corner stiffness on the structural behavior of a prefabricated concrete framed tube structure?**

### 4.3 Case Study Zalmhaventoren: Potential of fully prefabricated tube-in-tube structures

With the increasing height of framed tube high-rise structures also the need for additional stability systems increases. One of the systems which is quite popular and can be constructed with the use of prefabricated concrete is a stability core within the framed tube. Tolsma concluded in his research [30] that prefabricated cores were a viable alternative for cast-in-place ones. A stabilizing core and framed tube façade together form a so-called tube-in-tube structure. On these kind of structures though, not nearly as much research is available as on framed tube structures without a stabilizing core and this can be considered a missing link. This leads to the following problem statement:

*Although additional stability systems are needed to construct higher framed tube structures, way less research has been done on tube-in-tube structures compared to framed tube structures. For tube-in-tube structures constructed in prefabricated concrete even less information is available, so its potential remains unknown.*

Leading to the following research question:

*Is it possible to build a structurally feasible prefabricated tube-in-tube high-rise structure in the confines of the Dutch building market of approximately 200 m high and how does it compare to other prefabricated solutions?*  

As will be explained in more detail in an upcoming section of this report, this research question is answered by a case study on the Zalmhaventoren. The Zalmhaventoren is a proposed residential high-rise project in Rotterdam and will be redesigned as a tube-in-tube structure. This building has also been redesigned by Sven ten Hagen [29] as a prefabricated structure with shear walls as a lateral load resisting system. A qualitative comparison between the two prefabricated designs can be made thereafter, giving insight into the advantages and disadvantages of prefabricated tube-in-tube high-rise structures.
5 Parameter Research

5.1 Choosing a FEM-program

In the research phase of this thesis models have to be made to find the relations between certain parameters and the structural behavior of a framed tube building. Because the complexity of large prefabricated concrete structures like these it is impossible to do these calculations by hand. Therefore, a suitable computer program has to be found. The type of program which is suitable for these kind of models are Finite Element Method (FEM) programs. A program like this uses a numerical method in which a structure is divided in smaller subdomains on which simple element equations are used to approximate the overall behavior of a structure.

AxisVM is an FEM-program made by interCAD, which is distributed by Technosoft in the Netherlands. This program is used in practice by different civil engineering companies all over the world. The choice for this program lies in a few different factors. First of all, it has a reasonably user friendly interface and does not have a very steep learning curve. Secondly, one of the graduation committee members uses this program and is able to assist if problems arise during the model making or the interpretation of the results. Lastly, in the thesis by ten Hagen [29], which also deals with prefabricated high rise structures, the same program was used. In his thesis a comparison is made with another FEM-program, SCIA-engineer. In this comparison it becomes clear that AxisVM is a better program to deal with prefabricated concrete wall structures, which will be used extensively in this thesis. In Figure 5.1 the interface of AxisVM is shown.

Figure 13: A view of the interface of the FEM-program used throughout this thesis, AxisVM
5.2 General Model Parameters

In this section the general model parameters of all the models in the parameter research will be discussed. There are several reasons to keep a few general parameters the same throughout multiple models. To research the effect of one parameter on a system all other parameters should remain constant, in order to ensure that the change in results is inflicted by the change of that specific parameter. But even in models that are not part of research on the same parameter it is easier to have a few general parameters in place. It can be easier to create models that are more similar to each other in the FEM-program and the chance of mistakes in interpretation working on different models at the same time is reduced. It is also important to note that these general model parameters are not very important for the results of this parameter research. The changes in structural behavior are directly linked to the one parameter that is being researched at that moment. Results of different models (for instance deflection or shear lag factor) are compared to each other rather than the absolute values used for insight into the structural behavior of prefabricated framed tube structures.

5.2.1 Element Parameters

For all the models (with an exception for the cast-in-place models) all elements have the same parameters with respect to construction material and element size. All the elements used are made from prefabricated reinforced concrete with concrete class C50/60. This is one of the strongest concretes that is not considered to be a VHSC (Very High Strength Concrete). The production of these types of concrete classes is hard to obtain with cast-in-place concrete, but when prefabricated elements are made in a factory these kind high concrete classes are not a problem.

All the prefabricated shear wall elements in the structure have a thickness of 500mm. This is obviously a fairly large value for an element thickness, but was chosen because it was the same thickness used in the thesis by Tolsma [30], which makes the results comparable. By using the same thickness as he did, some connection stiffnesses he found could be used in the models of this parameter research.

The height of each element is set to be equal to the floor to floor height, which is 3.5m. This value was chosen because it is a moderate value between the values used in residential construction (often just over 3m) and office buildings (closer to 4m). By using a value in between these two types of structures the research can be used by designers of both types of buildings.

All the walls are constructed from elements in a masonry configuration. Only two element sizes are used, which are based on modular values: 3.6m and 7.2m. Each 7.2m element has an overlap of half its own length with the layers above and below the element level, resulting in the best load transfer between element layers as has been described in this thesis.

5.2.2 Structure Dimensions

Because this research is made with the Dutch construction industry in mind, the structural models reflect structures with the Dutch building codes and regulations in mind. An important regulation that differs from the regulations in e.g. the United Kingdom, the United States or Asian countries is the maximum depth of an office. Offices cannot extend further back than 7.2m from a window, in order for workers to be close to a daylight source at all times. Because the models do not include cores, the maximum depth of the building, or a wing of a building is estimated as: $2 \times 7.2m = 14.4m$.

The height of the structure is estimated by using the above determined maximum depth of the building and a plausible and economic slenderness of the structure. The slenderness of the structures ($\frac{h}{W}$) is estimated to be around 6. This gives a height for the models around $6 \times 14.4m = 86.4m$. Because a floor height of 3.5m was chosen, 25 floors lead to $25 \times 3.5 = 87.5m$, which has a slenderness of just over 6.

5.2.3 4 different configurations

For all research topics in the following section four different configurations are used. These are:

- A cast-in-place configuration without any windows
- A prefabricated configuration without any windows
- A prefabricated configuration with small windows ($1.6m \times 1.5m$)
- A prefabricated configuration with large windows ($2.4m \times 2.1m$)

In this way, insight can be found into the behavior of the more complex structures by comparing them to a cast-in-place structure with the same dimensions and loading. It is to be expected that the cast-in-place structure is the most rigid of the four configurations and the one with the largest windows the most flexible. In Figure 14 the four different configurations are drawn on a three-story wall piece.
5.2.4 Shear Lag Unit

In order to do research on the shear lag phenomenon, it is important that there is a quantifiable unit available to compare results of different models to each other. In previous research such a unit has been described. The work of Lee et al. [17] states the following:

“To measure the magnitude of the shear-lag phenomenon, the ratio \((p)\) of the axial force in the corner column to that in the centre column is defined as the shear-lag factor.”

To illustrate this parameter an explanatory image can be found in Figure 15. In this image the value of the normal force \((n_y)\) is plotted in a plan view of a flange wall of a structure which experiences shear lag.

For the work done by Lee et al. this parameter \(P\) is sufficient to describe the shear lag behavior of the structure. Because the shear lag behavior of only rectangular structures was researched, the shear lag factor \(P\) which describes the behavior in the flange wall of the structure describes the shear lag effect in the entire structure. For this research however, in which also non-rectangular floor plans are researched, the shear lag factor \(P\) alone is not sufficient.

In walls of the structure which cannot be considered as flanges the shear lag factor cannot be determined in the same way as in the flange walls. In web walls the value of the normal force \((n_y)\) is close to or equal to zero in the middle of the wall. If \(P = \frac{n_{y,A}}{n_{y,B}}\) would be used this would lead to a value of \(P\) that approaches infinity for values of \(n_{y,B} = 0\). Therefore a second shear lag factor \(P^*\) has been proposed which is based on the slope of the \(n_y\) in the middle part of the wall compared to the value in the corner. Again an explanatory image is provided in Figure 16.

The expression proposed reads:

\[
P^* = \frac{n_{y,A}}{\frac{1}{2} \cdot (n_{y,C} - n_{y,B}) + n_{y,B}}
\]

For this derivation of the shear lag factor \(P^*\) it is unimportant whether \(n_{y,B}\) is the same sign as the value of \(n_{y,A}\). It can also be noted that when \(n_{y,B} = 0\) the expression is reduced to: \(P^* = \frac{n_{y,A}}{\frac{1}{2} n_{y,C}}\).

One last important thing to consider is that although both \(P\) and \(P^*\) are both units to quantify the shear lag in a structure (or part of a structure) and are mostly in the same order of magnitude, they are units which are derived in different ways and should not be compared to each other but only to other values of the same unit.

5.2.5 Mesh Size

When determining the shear lag units described it is important to understand that the values in the corners are calculated by a Finite Element Method (FEM). Programs that use this method do not give the exact answer...
to a calculation, but rather approximate the values in a model. The value for this approximation is based on all different model parameters. A lot of these parameters depend on the structure which is being modeled, but some are not.

One parameter that is not based on real world behavior of the structure is the element mesh size. This is the size of the smaller sub-domains in which the entire structure is divided to make a calculation. Generally, a smaller mesh size does increase the accuracy of the results for a model. The trade-off is that models with a smaller mesh size take up a longer computation time. It is important to find a good balance between the accuracy and computation time for the models of this thesis. For the entire parameter research section of this thesis, rectangular shaped mesh elements are used.

By making a choice for a mesh size in the models it is important to have the goals for this thesis in mind. Because most of the research will be based on the stresses and forces near the base of the tower, rather than at the top, a uniform mesh size over the entire height of the structure might not be the most suitable option. Instead, a denser mesh on the lower levels and a sparser mesh in the rest of the structure will be chosen.

In the models of this thesis shear lag will be present. As explained before, this causes the stresses in the corner of a framed tube structure to be significantly higher than the stresses in the other parts of the flanges or webs of the structure. In the previous section two shear lag factors were introduced in which the stress values in the corner are a key factor in the calculation. Because the value of the stress spikes in these corners quite severely, the mesh size near these corners in the base of the tower have an influence on the values found. This is because the values of the stress are determined per mesh element. If the elements near the corners are small, the average stress in these elements is higher than if the elements would be larger and the stress would be smeared out over a larger area. To get a sense of this effect and choose a mesh which balances accuracy and computation time, a study has been done to see what the effect of the mesh size is on the shear lag factors $P$ and $P^*$. The results of this study can be seen in Figure 17.

![Figure 16: An explanatory image of the derivation of the shear lag factor $P^*$](image)

$$P^* = \frac{n_y, A}{n_y, D} = \frac{n_y, A}{2(n_y, C - n_y, B) + n_y, B}$$

![Figure 17: The relation between the size of the mesh elements in a model and the shear lag factors $P$ (left) and $P^*$ (right) of the cast-in-place and prefabricated configurations without openings](image)
The relation between mesh size and the shear lag effect is clearly visible in the graphs for both $P$ and $P^*$. As expected, a smaller mesh size produces a higher shear lag factor. This is the case for both the cast-in-place and the prefabricated configurations. The configurations with windows were not researched because the windows are a factor that made it hard to get a fair comparison between the models with different mesh sizes. To find an appropriate mesh size that does not has a too large computation time, for the bottom of the structure a mesh size of 0.35m was chosen. The difference between this mesh size and the smallest researched one (0.1m) is less than 3.5% for all researched configurations and shear lag factors. This mesh size was used in the bottom three stories of the building. For the remaining structure a mesh size of 1m was used. This non-uniform mesh size configuration gives good results in a reasonable computation time and is therefore used throughout this research.

5.2.6 Connections

The type of connections between elements in a model of prefabricated elements is an important parameter, for it determines the way the different elements fit together. To get a good insight into the behavior of the structures in the models, the order of magnitude of these connections is important, although, as explained earlier, parameter research is comparative research. To get to these values in the right order of magnitude, values from older research will be used and if necessary slightly modified to fit the model that they will be used in. The orientation of the local axes of a connection can be found in Figure 18.

![Figure 18: The orientation of the local axes in connections between elements](image)

**Horizontal Connections**

The stiffness of the horizontal connections is based on a combination of the used mortar strength and the reinforcement present. As previously mentioned, if the order of magnitude is correct this suffices for this part of this thesis. Therefore values were taken from another research project by Vanbersky and van Keulen [36]. For a "normal mortar connection" the following values are used:

$$K_x = 1.8 \cdot 10^6 kN/m/m$$
$$K_y = 3.4 \cdot 10^7 kN/m/m$$

Because the models experienced some numerical problems one of the committee members advised to give a value of $K_z = 1.0 \cdot 10^7 kN/m/m$ to the last remaining non-rotational stiffness. All the rotational stiffnesses are $K_{xx} = K_{yy} = K_{zz} = 0$.

Near the corner connections of the structure the value of $K_y$ has been lowered after it was found that in the single point where the corner connection and the horizontal connection met a high force was transferred. The result of this transmission was that the stiffness of the corner connection had little influence on the transmission of forces between the flange and web walls of the structure, which is clearly not a realistic result. So in the 100mm closest to the corner connection the stiffness in the $y$-direction is $K_y = 1.0 \cdot 10^4 kN/m/m$, which is a low, non-zero value. The implications for the model will be discussed in the section on the AxisVM model.

**Vertical Connections**

The vertical connections which are not corner connections are the easiest to describe for the model. Because it is assumed that these are open, non-structural connections. The stiffness of these connections is zero in every direction. This leads to:

$$K_x = K_y = K_z = K_{xx} = K_{yy} = K_{zz} = 0$$
Corner Connections

For the values of the corner connection another earlier research is consulted. In the research by Tolsma [30] multiple corner connections have been researched. For this research phase one of those corner connections is used: the Interlocking Halfway Connection (IHC). Tolsma first determines the discrete stiffness in a 2D FEM-analysis of the connection, $K_{\text{discrete}} = 2879 \frac{kN}{m}$. Because the stiffness used in the models is a stiffness over the height of the connection ($kN/m/m$) this value has to be smeared over the connection. This gives:

$$K_{\text{smeared},x} = \frac{K_{\text{discrete}}}{h} = \frac{2879}{3.5} = 8.22 \cdot 10^5 \frac{kN}{m/m}$$

in which $h$ is the height of the connection. This is the stiffness derived from the compression force in the concrete and acts in the local $x$-direction.

In the other directions the stiffness is derived from a protruding bar which is placed in a gain in the other element. Tolsma gives an indication of this stiffness for a bar with diameter of $25mm$, which can withstand a shear force of $1.68 \cdot 10^{-3}MN$. If a slip of $1mm$ is assumed, the stiffness becomes:

$$K_u = \frac{F_u}{\delta_u} = \frac{1.68 \cdot 10^{-3}}{0.001} = 1.68 \frac{MN}{m}$$

The smeared stiffness over the height of the connection can now be determined by filling in the distance between two protruding bars (in this case $1.75m$). This gives a stiffness

$$K_{\text{smeared},y} = K_{\text{smeared},z} = \frac{1.68}{1.75} = 0.96 \frac{MN}{m^2} = 0.96 \cdot 10^3 \frac{kN}{m/m}$$

This is the stiffness for both the $y$-direction and the $z$-direction. The reason that this is the same stiffness is that they both rely on the same protruding bar for their stiffness, and this bar has the same capacity in each direction. Due to a later found error in these calculations, the values used in the model are double the values calculated here: $K_x = 1.65 \cdot 10^6 \frac{kN}{m/m}$ and $K_y = K_z = 1.92 \cdot 10^6 \frac{kN}{m/m}$. Because this same error is present in all models, this has no effect on the results, for parameter research is comparative research. The rotational stiffnesses are again: $K_{xx} = K_{yy} = K_{zz} = 0$

5.2.7 Loads

The loading in a structure consists of multiple parts such as self weight, live load on the floors and wind loads. Not all of these loads are used in this part of the research however. Because the focus of the parameter research lies with the shear lag effect and the lateral deflection of the structures only the lateral loads are taken into account, which are the wind loads on the structures. All the vertical loads, due to both self weight and live loads on floors are neglected. Because only the lateral loads on the structure are present, insight in the shear lag effect is more easily derived from the results. The absence of second order effects on the lateral deflection will not impact the research results for all models will not experience second order effects.

The loading caused by wind for the parameter research is a value that should be in the right order of magnitude and applied the same way for all different building shapes. In order to keep this simple, some effects have been neglected. For instance, the wind loading is equal independent of the height on the structure and the windward and leeward faces of a structure produce the same loads on the structure. Another simplification is how the loads are applied on the structure. Rather than an area load perpendicular to the face the wind acts on, the loads are applied as line loads parallel to the wind direction in the shear walls that are in this direction. In practice, the floors of a structure would act as a diaphragm to transfer the wind loading there, but there are no floors present in these models to do this.

In Figure 19 the wind loads on the structures can be seen. The colored areas indicate the wind loaded areas and are corresponding to the lines with the same color which represent the line loads applied. All the loads on the structures with the different floor plans can be constructed in the same way. The magnitude of the loads can also be seen. They are estimated to be $4kN$ per $1m$ per one floor height. Which, as an area load accounts to: $\frac{4kN/m}{3.5m} = 1.14 \frac{kN}{m^2}$.

5.2.8 AxisVM model

To construct three dimensional elements in AxisVM that can be subjected to loads in all different directions a domain should be made with the shell property. This shell should then be given to correct thickness ($500mm$) and concrete class (C50/60). This concrete class is part of the Eurocode [4] and has the following material properties:
The colored areas indicated the wind loaded areas and are corresponding to the lines with the same color which represent the line loads applied:

- $E = 37000 \frac{N}{mm^2}$
- $\nu = 0.2$
- $\rho = 2500 \frac{kg}{m^3}$
- $f_{ck} = 50.0 \frac{N}{mm^2}$

The connection stiffness in the models is derived from the previous subsection on connections. They are entered into the model as so-called Edge Hinges between two Shell elements. The open vertical joints are created by using edge hinges with a stiffness of zero in all directions. If no edge hinge would be created, the program would assume the two shell elements were connected as if they were one monolithic entity. This is the way the cast-in-place models are created.

Because early models of the corner connections gave some unwanted and unrealistic results, two measures were taken to make sure they would behave more as they would in reality. The first of the two problems that was encountered consisted of a slight difference in results between a corner connection modeled as an edge hinge from the web wall to the flange wall and an edge hinge from the flange wall to the web wall. To overcome this problem, a very small Rib element was created in each corner, to which both wall elements could be connected. This rib element is so small ($1 \times 1 mm$) that it does not have a significant effect on the structural behavior of the models. Because the one edge hinge is thereby replaced by two edge hinges in series, the calculated stiffness for the one edge hinge has to be multiplied by 2 for both edge hinges.

$$\frac{1}{k_{eq}} = \frac{1}{k_1} + \frac{1}{k_2}$$

$$\frac{1}{1.92 \cdot 10^3 kN/m/m} = \frac{1}{3.84 \cdot 10^3 kN/m/m} + \frac{1}{3.84 \cdot 10^3 kN/m/m}$$

The second problem was already explained in the section on the horizontal connection, where the horizontal connection in the point where it met with the corner connection resulted in unrealistic results. To be able to give the horizontal connection a different value near the corner, an extra element was introduced near the corners with dimensions of $0.1m \times 3.5m \times 0.5m$, which is a strip of the normal element with the length of $100mm$. In Figure 20 a corner connection in the AxisVM-model is shown, with both the ribs and the extra element.

In the bottom layer of the structure, where the stress results are used to determine the shear lag factors $P$ and $P^\ast$, the vertical connections within the flange and web walls are left out of the model. This measure was taken to ensure that the readings of the shear lag factors are not influenced by local effects due to the configuration of the walls, but reflect the overall behavior of the structure, which is the goal of this thesis. In reality this
would mean that the walls of the structure would be cast in-situ, or that the elements within the walls were structurally connected instead of open seams as in the rest of the structure.
5.3 Research Topics

In this section the results of different research topics will be presented. These topics align with the research questions in section 4.

5.3.1 Aspect Ratio

The first research topic is the effect of the width to depth ratio, the so called aspect ratio to the shear lag effect. Written as an expression, the aspect ratio is defined by:

\[ A_r = \frac{W}{D} \]

In the work of Singh [26] a similar study has been performed on cast-in-place framed tube structures. The results from this study were summarized as follows: "It was seen that aspect ratio \( A_r \) does not have significant effect on the shear-lag behavior of the flange frame."

The reason for investigating the effect of \( A_r \) on shear lag once again is that the work of Singh et al. was performed on a cast-in-place framed tube structure, which had a way lower slenderness than the structures to be researched in this thesis. Their structure had a height of 120 \( m \) but a floor plan of 42 \( m \times 54 \) \( m \), leading to a slenderness that is \( < 3 \). For this investigation the structures have a height of 87.5 \( m \) and a slenderness of just above 6 in the most slender direction. All other properties are also described in the previous section on general parameters. The different floor plans and their value for \( A_r \) can be found in Figure 21. The loading of the different structures is based on the assumption that the wind load on one floor for the structure with a \( A_r = 1 \) results in a line load in the web walls of \( \frac{1kN}{m} \), as was explained in the section on loads. This is a roughly estimated value, but this does not matter for the results are in the shear lag factor \( P \) or \( P^* \), which is a unitless factor. In the following table, the loads for the different structures are given, based on their area loaded by the wind \( A_{wind} \) and the length of the web walls (which is the depth of the structure \( D \)).

<table>
<thead>
<tr>
<th>( A_r )</th>
<th>( A_{wind} )</th>
<th>( D )</th>
<th>( q_{wind} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.4</td>
<td>3.5 ( m \times 14.4 ) ( m )</td>
<td>36 ( m )</td>
<td>1.6 ( m ) ( kN )</td>
</tr>
<tr>
<td>0.5</td>
<td>3.5 ( m \times 14.4 ) ( m )</td>
<td>28.8 ( m )</td>
<td>2.0 ( m ) ( kN )</td>
</tr>
<tr>
<td>0.667</td>
<td>3.5 ( m \times 14.4 ) ( m )</td>
<td>21.6 ( m )</td>
<td>2.667 ( m ) ( kN )</td>
</tr>
<tr>
<td>1.0</td>
<td>3.5 ( m \times 14.4 ) ( m )</td>
<td>14.4 ( m )</td>
<td>4.0 ( m ) ( kN )</td>
</tr>
<tr>
<td>1.5</td>
<td>3.5 ( m \times 21.6 ) ( m )</td>
<td>14.4 ( m )</td>
<td>6.0 ( m ) ( kN )</td>
</tr>
<tr>
<td>2.0</td>
<td>3.5 ( m \times 28.8 ) ( m )</td>
<td>14.4 ( m )</td>
<td>8.0 ( m ) ( kN )</td>
</tr>
<tr>
<td>2.5</td>
<td>3.5 ( m \times 36 ) ( m )</td>
<td>14.4 ( m )</td>
<td>10.0 ( m ) ( kN )</td>
</tr>
</tbody>
</table>

The results are presented in two graphs in this section. The first one shows the shear lag factor \( P \) against the aspect ratio \( A_r \). The results in the graph in Figure 22 are only shown for the aspect ratios \( < 1 \). The reason behind this is that the flange length (which is equal to the width of the structure \( W \)) is the same for these aspect ratios. The results of the shear lag factor for structures with a longer flange length were influenced by the flange length itself rather than the properties of the structure, like the rigidity of the bays and the connections.
between the elements. A similar case can be found in the graph in Figure 23 for the shear lag factor $P^*$. Here only the values are shown for the models with an aspect ratio $> 1$, because they all have the same length for their web length (or building depth $D$).

![Figure 22: Relation between the aspect factor $A_r$ and the shear lag factor $P$](image1)

The first thing which can be noted about these results is the difference between the four different configurations. As is to be expected, the shear lag factor increases from the most stiff cast-in-place configuration to the most flexible configuration with the large holes. Furthermore, it is clear that the aspect ratio $A_r$ does not have a large influence on the shear lag factor $P$. The largest percentage increase between an aspect ratio of 1.0 and 0.4 is only around 5%. There is also no relation between the used configuration and this percentage increase.

![Figure 23: Relation between the aspect factor $A_r$ and the shear lag factor $P^*$](image2)

For the results of the relation between the shear lag factor $P^*$ and the aspect ratio $A_r$ some of the same observations can be made as for the previous diagram. The differences between the four considered configurations show a similar pattern as in the diagram on the shear lag factor $P$. Also, the lines of all configurations have a fairly low slope, indicating a small effect of the aspect ratio on the shear lag factor. The largest percentage difference between $A_r = 1$ and $A_r = 2.5$ is just over 12%. However, in this diagram a higher aspect ratio also means a higher shear lag factor $P^*$, where this was the other way around for the other shear lag factor: $P$. 

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In conclusion it is safe to say that the conclusion of Singh is still true for framed tube structures which are prefabricated and have a higher slenderness. The effect that $A_r$ has on the shear lag effect is not significant.

### 5.3.2 Polygons

In this part the changing parameter will be the floor plan, just as in the previous section. But instead of changing the aspect ratio of the structure, the shape of the floor plans will be changed by adding additional corners to the structure, approaching a circular structure. Three floor plans will be compared, a square floor plan, one in the shape of an octagon (8 corners) and one in the shape of a hexadecagon (16 corners) All of them have a circumference of 57.6m. These floor plans can be seen in Figure 24.

![Figure 24: The three different building shapes used in the Polygon research topic. Blue indicates the location of the line loads](image)

The blue lines in this image indicate the line loads on the different structures. The length of the blue lines is 14.4m in each of the different floor plans. The loading in the octagonal and hexadecagonal floor plans is not in the direction of the wall itself but in the same direction as the loading is in the square floor plan. This means that the load has to be transferred between the elements using not only stiffness in the $x$, $y$, and $z$-direction, but also some rotational stiffness is required. This was added to the AxisVM models only for this research topic. Because all models have an entirely different shape and the length of the walls changes from model to model, it is not possible to use the shear lag factor to compare the different structures. So other indicators for the structural behavior should be found. The two choices were the deflection in the $y$-direction (in the direction of the line loads) and the highest reaction force in the $z$-direction $R_{z,max}$. The results for the deflection are shown in Figure 25.

![Figure 25: Relation between the different floor plans and the deflection in the $y$-direction](image)

As can be seen in Figure 25, the four different configurations are also used for this research topic. And as is to be expected, the deflection is higher for the less stiff of these configurations. Comparing the different floor plan shapes, it can be seen that for all the configurations the deflection of the square floor plan is slightly higher.
than that of the other two. This can be explained by the lower moment of inertia in the square floor plan compared to the other two. The rate between the deflection of the structure of the square floor plan to the octagon and hexadecagon is independent from the openness of the structural elements. From these results no positive influence can be seen from changing the floor plan to a approach a circle.

The relation between the different configurations and the maximum value for the reaction force in the \( z \)-direction is that the cast-in-place and prefabricated configurations are fairly close but the configurations with the holes have significantly higher values. This can be explained by the holes themselves. The larger the holes are, the smaller the area remaining to transfer the vertical and shear forces. The difference between the different floor plan shapes and the value for \( R_{z,max} \) is the best indicator for the behavior of the structure for shear lag. The more corners and sides a structure has, the lower the maximum value for the reaction force in the \( z \)-direction is. For the configuration with the big holes, the difference between these maximum values for the square and the hexadecagonal floor plan is nearly 50%.

To explain this difference it helps to look at the total reaction force in the foundation in the \( z \)-direction. For the prefabricated configuration this is shown in Figure 27.

The value \( R_{z,max} \) is highest in a corner for each of the different floor plans. The difference in the value comes from the number of corners and the length of the elements in the construction. A larger number of corners and shorter elements, which are present in the octagonal and the hexadecagonal structures lowers the value of \( R_{z,max} \). In shorter elements the peaks due to the shear lag effect cannot develop because the middle and the edges of an element are closer together. Also, if the number of corners is higher, the peaks are spread out over multiple corners, lowering the maximum reaction force. So a higher number of corners and smaller elements is beneficial to the force distribution. However, from a practical standpoint it might not be desired to build a
structure in this way.

5.3.3 Angle

The third research topic is the effect of the corner angle to the deflection and shear lag effect. For this research, it is assumed that the stiffness of the corner connection is not dependent on the corner angle. This assumption can be made because the only aspect that may differ in an IHC (Interlocking Halfway Connection) between different corner angles could be the compression area between the two elements and this change is most likely negligible. The size and capacity of the protruding bars will stay the same independent on the corner angle. Four different building shapes have been modeled for this research topic. All models are rhombi (or diamonds). This means that all walls have the same length (14.4m), consist of two pairs of parallel sides and have two pairs of the same corner angles. All these buildings shapes will be worked out in the previously mentioned four configurations and have a height of 87.5m. The building shapes can be seen in Figure 28.

![Figure 28: The four different building shapes used in the Angle research topic](image)

The loading of the models is simplified to get a clear view of the different structures with the same load. However in reality the wind loads in the y-direction would be higher for the model with the more acute and obtuse angles, for it has a higher wind loaded area. The loading of all models is again based on the assumption that the wind load on one floor for the structure with a $A_r = 1$ results in a line load in the web walls of $4\frac{kN}{m}$. These loads were added to the models in the direction of the $x$ and $y$-axes, not the direction of the walls themselves. The first results shown are for the deflection in the $y$-direction $e_Y$ with loading in that same $y$-direction. The results are shown in Figure 29.

![Figure 29: The relation between the Corner Angle and the deflection $e_Y$ for loading in the $y$-direction](image)

As was seen in the previous section on the aspect ratio, the cast-in-place configuration is the most stiff and the big holes configuration is the least stiff. The maximum deflection reflects this difference in stiffness for all building shapes. Another aspect that can be seen in the graph is that all lines are relatively flat, indicating only a limited relation between the corner angle and the deflection. However, a 15% difference between the deflection of the model with all 90° corners and the model with 60° and 120° corners can be found for all the
four different configurations. This difference can be explained by the change in moment of inertia between the square floor plan and the diamond shaped floor plans, which is even higher than 15% between the square floor plan model and the model with 60° and 120° corners. From this result, no negative effect of corners different than 90° can be found.

The results of the maximum deflection in the $x$-direction for loading in the $x$-direction were that there was no relation between the corner angle and the deflection at all. All the different configurations had the same deflection for the 4 different building shapes. The deflection in the $y$-direction however, did show a clear relation with the corner angle. The smaller the acute corner angle, the larger the deflection in the $y$-direction. This effect is magnified by a lower structural stiffness. The reason for this deflection perpendicular to the direction of the loads can be found in the torsion experienced by the models with more acute and obtuse angles. A graph with the relation between the corner angle and the deflection perpendicular to the load can be seen in Figure 30.

![Figure 30: The relation between the Corner Angle and the deflection $\varepsilon_Y$ for loading in the $x$-direction](image)

The relation between the shear lag effect and the corner angle has also been analyzed. The shear lag factors $P$ and $P^*$ (as described in a previous section) were investigated for all models. As could be seen in the subsection on the aspect ratio, a lower lateral building stiffness means a larger shear lag factor. This was however not the parameter that was researched in this research topic. The relation between the shear lag factor $P$ and $P^*$ and the corner angle can be seen in Figure 31.

![Figure 31: The relation between the Corner Angle and the shear lag factor $P$ (on the left) and $P^*$ (on the right)](image)

For all angles, from 60° to 120°, the relation between the angle and the two shear lag factors $P$ and $P^*$ have been determined. As can be seen by the flat lines in the two graphs, there is no significant effect between the corner angle and either the shear lag factor $P$ or the shear lag factor $P^*$.

### 5.3.4 Inside Corners

The fourth research topic is about the difference between the shear lag behavior of inside and outside corners in a framed tube structure. In a normal rectangular structure all the corners are outside corners, but in structures...
with a different floor plan, inside corners can be present as well. Although the difference has been explained earlier in this thesis, a quick reminder for the reader about the difference between inside and outside corners. An outside corner has an angle of around 90° inside the building, which is the case for all normal rectangular structures. An inside corner, which is present if the floor plan of a structure has e.g. a H-shape or a T-shape has an angle in the interior of the building of around 270°.

To research the difference in shear lag effect between inside and outside corners multiple models have been made for structures with non-rectangular shapes. The structures all contain both inside and outside corners. Both the inside and outside corners are modeled as a Interlocking Halfway Connection and have the same stiffnesses in the different directions. Using the shear lag factors $P$ and $P^*$ described earlier the results will be compared to see if there is any significant difference between the two. The different floor plans are:

- A +-shaped floor plan
- A T-shaped floor plan
- An H-shaped floor plan loaded in two different directions

The similarity between all these floor plans is that they are symmetric, so that behavior from asymmetry will not occur and alter the results. Another similarity is that all walls are either 14.4m long or a multiple of 14.4m. All different floor plans will be modeled in the four different configurations used throughout the parameter research.

The results will be presented in Figure 32 and Figure 33. In these images the floor plans will be presented and the corners featured in the results indicated. The choice for the corners is based on the length of the walls connected to corner (which should be equal for all corners that are compared to each other) and based on symmetry, to exclude the corners with the same values. As has been explained earlier, values of $P$ and $P^*$ are not to be compared to each other for they are different units with different origins and therefore have separate graphs.

The first image will show the results from the different models for the shear lag parameter $P$. A distinction has been made between inside and outside corner in order to compare the results.

![Figure 32: A comparison for the shear lag factor $P$ for inside and outside corners in different structures](image)

As can be seen in Figure 32, the shear lag factor $P$ increases for the less stiff configurations modeled for the research. This is the case for both the inside as the outside corners. Although the differences between the inside and outside corners are rather small, a pattern can be seen in the graph. For the inside corners the shear lag
factor $P$ is slightly higher than for the outside corners. The difference increases for the less rigid configurations from 2.5% for the cast-in-place configuration to 4.3% for the big holes configuration.

A similar image was made for the shear lag factor $P^*$ and can be found in Figure 33. A few of the same things

![Image](https://via.placeholder.com/150)

Figure 33: A comparison for the shear lag factor $P^*$ for inside and outside corners in different structures

can be seen as in the image about the shear lag factor $P$. The shear lag factor $P^*$ increases for the less stiff configuration for both the inside and outside corners. And again, the differences between the inside and outside corners are small. Finding a similar pattern in the graph as in the previous case is harder to do in this case. Therefore a numerical approach was taken. In the following table the mean values and standard deviations of both the inside and outside corners are given per configuration:

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Cast-in-place</th>
<th>Prefabricated</th>
<th>Small Holes</th>
<th>Big Holes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inside Corner Mean</td>
<td>1.1821</td>
<td>1.2067</td>
<td>1.3556</td>
<td>1.4916</td>
</tr>
<tr>
<td>Inside Corner SD</td>
<td>0.0380</td>
<td>0.0711</td>
<td>0.0702</td>
<td>0.04686</td>
</tr>
<tr>
<td>Outside Corner Mean</td>
<td>1.2279</td>
<td>1.2511</td>
<td>1.4053</td>
<td>1.5151</td>
</tr>
<tr>
<td>Outside Corner SD</td>
<td>0.0366</td>
<td>0.0707</td>
<td>0.0781</td>
<td>0.0728</td>
</tr>
</tbody>
</table>

From the table it can be seen that the mean values for the shear lag factor $P^*$ of outside corners are slightly higher, around 3% for all configurations but the configuration with the big holes, in which case the difference is even smaller. The standard deviation of all the configurations are close together as well. Again the big holes configuration deviates from this pattern, in which case the standard deviation is higher for the outside corners. From these results the it can be concluded that inside corners do not lead to more structural problems caused by shear lag than outside corners.

5.3.5 Corner Stiffness

In this subsection the relation between the force distribution in the bottom of a structure and the stiffness of the corner connection will be described. The structure for which this relation will be determined has a square $14.4m \times 14.4m$ floor plan and is $87.5m$ high. All other parameters are as described in the general parameters section.

The stiffness of the corner connection is an interesting parameter in a framed tube structure like the ones that
are studied in this thesis. The connection binds the flange walls and the web walls together and determines to what level they work together as one in the structure. As has been described earlier, the standard stiffnesses in corner connections are the following:

\[ K_x = 1.65 \cdot 10^6 kN/m/m \]
\[ K_y = 1.92 \cdot 10^3 kN/m/m \]
\[ K_z = 1.92 \cdot 10^3 kN/m/m \]

In which the \( K_x \) is the normal stiffness and \( K_y \) and \( K_z \) are the shear stiffnesses. These stiffnesses are smeared over the height of the connection. To examine the influence of this parameter on the behavior of the structure, a comparable and relevant unit should be looked into. The unit chosen is the total force in the \( z \)-direction at the support of the flange wall. If the load input is the same for all the models with different corner stiffnesses this unit determines how well the flange and webs walls work together. In Figure 34 the reaction force in the \( z \)-direction of a model is displayed for the entirety of the structure. The area in red is the aforementioned total reaction force in the \( z \)-direction.

Figure 34: The reaction force in the \( z \)-direction for a structure. The area in red is the total force in the \( z \)-direction for one flange wall

The corner stiffnesses are varied greatly to examine the influence they have on the behavior of the structure. They are multiplied by the following factors: 0.01, 0.1, 1, 10, 100 and 1000. The results are plotted to a logarithmic scale in the graph in Figure 35.

As can be seen in Figure 35, there is a clear relation between the corner stiffness and the total vertical reaction force in the flange walls of the structure. Because the loading is modeled to be present in the web walls, all the forces ending up as a reaction force in the flange walls has had to be transmitted through the corner connection. A lower connection stiffness means that less of the shear forces can be transmitted this way lowering the value of the total reaction force in the vertical direction in the flange walls. This relation is clearly visible in the graph. With an increasing corner stiffness factor the total vertical reaction force increases as well. Increasing the corner stiffness to above the standard values used for this investigation does not have a large effect on the results, with the values for 10, 100 an 1000 times the normal stiffness being almost equal for all three investigated configurations.

The three investigated configurations differ slightly from one another. For the standard corner stiffness and higher stiffnesses they are very comparable, but if the corner stiffness is lower than its standard values differences appear. The prefabricated configuration has the lowest transfer of forces to the flange walls and the big holes configuration has the largest transfer of forces the the flange wall. This can be explained by the web walls’ own ability to cope with the forces on the structure. The more rigid the web walls are, the less they need the flange walls to cope with the forces. And if the forces can only be transferred through a less stiff connection, a larger part of the forces will be transferred to the foundation through the web wall itself.

Another aspect that can be investigated is the relation between the corner stiffness and the maximum deflection in the \( y \)-direction, which is the direction of the forces on the structure. Again, this relation has been tested over a large range to be able to get a good view of the effects on the structural behavior of this parameter.
As can be seen in Figure 36, there is a relation noticeable between these two values. As in the last case, the relation is most clear if the corner stiffness is reduced considerably. Increasing the corner stiffness to a value higher than the standard value used has barely to no effect on the maximum deflection of the structure. For lower values of the corner stiffness the maximum deflection increases. This can be explained by the fact that the flange walls of the structure are used less and all the forces are on the web walls of the structure, causing larger deflection. The difference between the three configurations is quite visible in the graph. As can be seen in previous sections, the less stiff configurations have a larger deflection than the stiffer configurations. This is also the case for this investigation. The impact of reducing the corner stiffness is slightly higher for the stiffer configurations. This can be seen in the following table:
<table>
<thead>
<tr>
<th>Configuration</th>
<th>Prefabricated</th>
<th>Small Holes</th>
<th>Big Holes</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varepsilon_{y,\max,0.001}$</td>
<td>32.098mm</td>
<td>47.614mm</td>
<td>78.81mm</td>
</tr>
<tr>
<td>$\varepsilon_{y,\max,1}$</td>
<td>10.437mm</td>
<td>16.812mm</td>
<td>34.527mm</td>
</tr>
<tr>
<td>Ratio</td>
<td>3.074</td>
<td>2.832</td>
<td>2.284</td>
</tr>
</tbody>
</table>

5.3.6 Open Seam Stiffness

The vertical seams within the walls of the structures have been modeled as open, non-structural connections throughout all the parameter research. They can be modeled as such because of the masonry configuration of the elements. In this section it will be investigated whether changing these connections to structural connections makes any difference to the structural behavior of the structure and if it could be worthwhile doing so in a structure with a framed tube system to resist lateral loads.

The stiffness of this structural connection has to be a realistic value. To assure this, once again values from the work of van Keulen [36] are used. For a profiled unreinforced connection the shear stiffness is given as:

$$K_x = 5.0 \cdot 10^5 kN/m/m$$

For the normal stiffness a higher stiffness was taken:

$$K_y = 2.0 \cdot 10^7 kN/m/m$$

The other stiffnesses are $K_z = K_{xx} = K_{yy} = K_{zz} = 0$. Once again to get a good view of the influence of this parameter on the structure this value is multiplied by a the following factors: 0.01, 0.1, 1, 10, 100, 1000 and compared to the configuration with the open seams. First the effect on the shear lag factor $P$ was researched.

The results can be found in Figure 37. The lines for all configurations are almost horizontal, indicating that

![Figure 37: The relation between the stiffness of the open seam connection and the shear lag factor $P$.](image)

there is no relation between the stiffness of these connections and the shear lag in the structure. Another aspect which was researched is the relation between the connection stiffness in these seams and the maximum deflection in the $y$-direction $\varepsilon_Y$. These results can be seen in Figure 38. Once again, the fairly horizontal lines indicate a relation between these two values which is not very strong. The most noticeable changes are for the big holes configuration if the stiffness of the connection is really high, but even that difference is only 2.5% between the normal connection and the 1000× stiffer one.

With both these investigated relations in mind it is easy to see that giving structural properties to the connections which are normally open in a masonry configuration only has a very small influence on the structural behavior of a structure. This relation can be considered negligible and therefore giving structural properties to these kind of connections is a waste of time and money in the building process.

5.3.7 Slenderness

The last research topic in this section is on the effect of the slenderness of the structure on the shear lag of that structure. To do this research, models were made for structures from 5 to 50 floors, with 5 floor differences. All these models have the same $14.4m \times 14.4m$ floor plan. This way structures were created with a slenderness
Precast Concrete in Framed Tube High-Rise Structures

Figure 38: The relation between the stiffness of the open seam connection and the maximum deflection of the structure in the $y$-direction

$(\frac{H}{W})$ of 1.2 to 12. These structures were made for all four configurations and the values for the shear lag factors $P$ and $P^*$ determined. The results can be found in Figure 39 and 40. The results for the relation between the

![Graph showing the relation between slenderness and shear lag factor $P$.](image)

Figure 39: The relation between the slenderness and the shear lag factor $P$

slenderness and the shear lag factors $P$ and $P^*$ have quite some similarities. Once again, it can be seen that the least stiff configuration, the configuration with the big holes, has the largest shear lag factor and the most stiff configuration of the cast-in-place model has the lowest shear lag factors for all slendernesses. Both graphs also show that the shear lag factors $P$ and $P^*$ increase if the slenderness decreases. Especially for the models with only 5 or 10 floors the increase in shear lag factor is high. This strong relation is not present for higher slendernesses, were the lines are fairly horizontal.

The reason why the shear lag factors, and therefore the shear lag itself, is higher for the structures with a low slenderness can be found in the way structures with different slendernesses cope with lateral forces. A structure can cope with lateral forces in two ways. The first one is by bending deformation, the second one is by shear deformation. For different slendernesses a different ratio between these two is present. In structures with a high slenderness, bending deformation is the most important way of dealing with lateral loading. For structures with a low slenderness shear is the main way of dealing with lateral loads. This is why for the structures with a lower slenderness the shear lag factor is higher. For these structures the percentage of lateral force brought to the foundations by shear is high. This leads to higher shear forces in the structure, which lead to more shear lag as a result. This is especially the case for the configurations which are not that well fit for transferring shear forces efficiently, like the configuration with the big holes. This is why the structure with this configuration with the lowest slenderness has a shear lag factor $P$ of 12.43 and does not even fit in the graph in Figure 39.
5.4 Summarized graphical representation of the parameter research

The found relations between the researched parameters and the shear lag factors and deflections could be of use in the earliest design stages for prefabricated framed tube structures. Rather than having to read this entire section or working through the conclusions or summary of this Master’s thesis, a tool is designed to give a designer as much information as possible in a single glance. The advantage of this, is that a designer in such an early stage probably lacks the time to do an extensive literature study and this makes the process easier. In this way the academic research and the building practice are closer together. On the next page a summarized graphical representation of the parameter research can be found on A3 format. This presentation contains both the framework of the parameter research and the most important results, all with references to the correct sections of this Master’s thesis.
This page should be replaced by an A3 containing a summarized graphical representation of the parameter research.
This page should be replaced by an A3 containing the a summarized graphical representation of the parameter research
6 Case Study Zalmhaventoren

One of the research questions in this thesis is the following:

Is it possible to build a structurally feasible prefabricated tube-in-tube high-rise structure in the confines of the Dutch building market of approximately 200m high and how does it compare to other prefabricated solutions?

Because this report is focused on the Dutch building market, some boundary conditions are created to ensure the designed building fits in its confines. These are:

- Wall elements may not exceed a thickness of 500mm
- The majority of the elements should be made from a standard concrete class. VHSC (Very High Strength Concrete) and UHSC (Ultra High Strength Concrete) may only be used sparsely and where needed
- The slenderness of the building will not exceed 8
- The Eurocode will be used to test the structural properties
- The Dutch building codes will be used for determining the daylight conditions of the structure

In this section a design for a structure that fits these requirements will be chosen, described and the steps used to convert it in the FEM-program explained.

6.1 Redesign of the Zalmhaventoren

6.1.1 Subject Choice

In addition to the requirements mentioned earlier, another aspect is important for a design choice. Where within the Netherlands would it make the most sense to construct a structure of a significant height and how tall should such a structure be? From the literature study it was found that nowhere in the Netherlands the economic situation can solely justify the construction of structures higher than the tallest structure in the Netherlands today (Maastoren, 165m). However, being the tallest structure of a country or city can sometimes be a consideration for building tall anyway. One could argue that reaching a specific height barrier with a round number can have the same value. For instance, the tallest structure under construction right now is the Kingdom Tower in Jeddah, Saudi Arabia. In early stages the structure was designed to reach one mile in height (1609m) but when that was deemed impossible the height was changed to be just more than one kilometer in height (1008m). These heights are for Dutch standards very unrealistic, but reaching a height of 200m might be a barrier that can be reached.

Instead of designing a structure from scratch and implementing different stability systems in it, the choice has been made to investigate completed or proposed structures and alter those designs to fit the requirements in structural system and height. A structure for this purpose was found in the literature study. In the work of ten Hagen [29] he alters a structure to fit his prefabricated structural system and other requirements. This structure is a proposed residential tower in Rotterdam called the Zalmhaventoren. An artist rendering can be seen in Figure 41.

The original structure was designed as an in situ concrete residential tower which relied on shear walls for its lateral stability. The floor plan was 30m × 29.5m and the height 189.1m. Ten Hagen changed this into a prefabricated concrete structure with the same stability system and a height of 202.25m, to pass the barrier of 200m. If this structure was redesigned as a prefabricated tube-in-tube structure within this thesis, a comparison between the two prefabricated designs for the Zalmhaventoren can be made.

6.1.2 Changes to the floor plan

Redesigning a structure to fit another stability system however, is not something that can be done without altering some key aspects of the building. These changes can best be seen in the floor plan of the structure. In Figure 42 the floor plan of the original Zalmhaventoren (which is also the one ten Hagen used) is shown next to the redesigned structure with the tube-in-tube lateral stability system. As can be seen in the image, the original floor plans are dominated by the shear walls, two in one and three in the other direction. An important function of these shear walls is that they resist the lateral wind load enacted on the structure. These shear walls also function as the main vertical load resisting elements, on which the floors rest. Another part of the vertical load bearing systems is present in the left and right façade, which are load bearing as well. The other two façades only have to support their own weight and do not transfer any forces from the floors to the foundation. Another aspect visible in the original floor plans are the balconies. Each façade has a large and a smaller balcony. The exact size and placement of the balconies is related to the shear walls, because they cannot be split up by a
shear wall. In between the balconies there is a small recess in the floor.
On the right side of the image the floor plan of the redesigned version of the Zalmhaventoren is shown. Most noticeable is the lack of shear walls here, giving an open floor space to work with. Instead, a core in the center of the floor plan is used and around the entire perimeter a tube structure is created. This has some implications for the façades however. First of all, the windows that used to be in the corners of the structure have been removed. In a tube structure it is important that the different sides work together and the best way to make sure they do is to connect them in the corner. Also, an extra column has been added in the center of the large balcony to keep the lateral stability structure more continuous despite the balconies. The little recess in the floor between the balconies has also been removed.

6.1.3 Outer appearance

The changes in the structural system also have some influence on the architectural appearance of the structure. Changes to the corners and balconies would be most noticeable. To check how significant this changes the appearance of the structure, 3d-models have been made of several floors of the structure. These models can be seen in Figure 43.
The differences between the two models can be seen in the image. Especially the corners have a different look.

![Shear walls](image1.png) ![Tube-in-tube](image2.png)

Figure 43: 3d models of several floors in the Zalmhaventoren. The model on the left is based on the original plans of the Zalmhaventoren, the model on the right is the redesigned floor plan with tube-in-tube as a lateral stability system.

for the two different designs. Because the extra column on the large balcony is not clad in the same material as the other columns, this difference is more subtle. Overall, the 3d models are rather similar in appearance and this change should not be a problem. However, architecture is a more subjective field and an architect could disagree with these changes to his or her design. If that were to be the case, a collaborate solution would have to be found to resolve this issue.

6.1.4 Floor to ceiling and overall height

The open floor space that is created by using the tube-in-tube system can add value to the structure as a whole. By designing the structure in such a way that it is not only fit for residential use, but also for office use for instance, the structure can be given a new purpose later in its lifetime that fits the market situation of that time. To ensure this, regulations for loads and floor heights, as well as installations should be kept in mind. The Dutch Bouwbeshuit specifies the floor to ceiling height for different purposes as follows:

<table>
<thead>
<tr>
<th>Building function</th>
<th>minimum floor to ceiling height</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residential</td>
<td>2.6m</td>
</tr>
<tr>
<td>Office</td>
<td>2.6m</td>
</tr>
<tr>
<td>Education</td>
<td>2.6m</td>
</tr>
<tr>
<td>Other functions</td>
<td>2.6m</td>
</tr>
</tbody>
</table>

For all functions the minimum value for the floor to ceiling height is the same. However, because for instance an office function requires a lot of installations which are usually put in a lowered ceiling, these kind of installations would have to be fitted into the floor itself to keep the required floor to ceiling height if a switch from residential use to office would be made later. The original Zalmhaventoren had a total floor height of 3.05m. That would leave $3.05m - 2.6m = 0.45m$ for a floor which can both bridge the span in the structure and incorporate these extra installations within. 0.45m would seem to be enough for such a solution.

Using this floor to floor height of 3.05m and the goal to reach a height of 200m, it can be determined how many floors are needed. The number of floors needed is: $\frac{200m}{3.05m} = 66.57 \approx 66$ floors. The total height of the structure will then be: $66 \times 3.05m = 201.3m$. The assumption in this calculation is that all floors are the same height. In the original design a panorama floor with a different height was near the top of the structure, but this is not implemented in the model for the effect of this one floor in the top of the structure on the overall behavior is likely very small. For the lobby, a double floor height of $2 \times 3.05m = 6.10m$ is used.
6.1.5 Floors

Because all the shear walls are removed from the floor plan, the floor system has to be changed as well. In the original Zalmhaventoren one-way spanning floor systems were used between the shear walls of the structure. Now, a two-way spanning floor system is used, spanning from the core to the façade of the structure. This is possible because in the tube-in-tube design all façades are load bearing, in contrast to only two load bearing façades in the original design. The exact floor system or the detailing of the connection will not be given in this thesis, for multiple floor systems could fit a prefabricated tube-in-tube structure and they all require different detailing. Instead, the requirements of the floor system will be given.

- Two-way spanning floor system acting a diaphragm between the tube and core of the structure
- Being able to span \(8.75m\) from core to façade in one direction without intermediate supports
- Being able to span \(\sqrt{8.75^2 + 7.8^2} = 11.72m\) from core corner to tube corner without intermediate supports
- Having a smaller or equal dead weight than the estimated \(7.2\frac{KN}{m^2}\)
- All the installations can be casted in the concrete during construction
- The thickness of the floor should not exceed \(0.45m\)

On the market today there are multiple weight saving two-way spanning floor systems that could satisfy these requirements. In a section 8.2 one of these options is calculated for the dimensions and floor loads in the structure.

6.1.6 Element configuration

The prefabricated elements of both the tube and the core of the structure are stacked in a masonry configuration. The implementation of a masonry pattern was complicated by the irregular nature of the façade in the original design of the Zalmhaventoren. Due to the placement of the balconies with respect to the shear walls and only half of the façades being load bearing, the slightly longer façades (30\(m\)) are created with multiples of 0.9\(m\). The other two façades (29.5\(m\)) use multiples of 1.0\(m\). Also, van Keulen [36] concluded that lintels should not contain the connection between elements, but that connections are rather placed in the columns of a shear structure. This leads to the element configuration in Figure 44. The configuration of the elements is a masonry pattern, with the complications described above makes that there are few elements in the tube that are repeated in the 2 floor repeating pattern. They also vary greatly in length. In the core, in which these limitations have a smaller impact the configuration is cleaner.

Through multiple iterations of the model the thickness of the elements and the concrete classes were determined. Three different concrete classes were used in order to satisfy the requirement not to use VHSC and UHSC in parts of the structure where this is not necessary. The following table lists which concrete class is used in which part of the structure.

<table>
<thead>
<tr>
<th>Floors</th>
<th>Core</th>
<th>Tube</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-2 (Lobby)</td>
<td>C80/95</td>
<td>C90/105</td>
</tr>
<tr>
<td>3-17</td>
<td>C80/95</td>
<td>C80/95</td>
</tr>
<tr>
<td>18-66</td>
<td>C50/60</td>
<td>C50/60</td>
</tr>
</tbody>
</table>

In which the following Young’s moduli are used:

<table>
<thead>
<tr>
<th>Concrete Class</th>
<th>Young’s modulus (E)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C50/60</td>
<td>(37000\frac{N}{mm^2})</td>
</tr>
<tr>
<td>C80/95</td>
<td>(42000\frac{N}{mm^2})</td>
</tr>
<tr>
<td>C90/105</td>
<td>(44000\frac{N}{mm^2})</td>
</tr>
</tbody>
</table>

6.1.7 Lobby

Another part of the structure that had to be changed due to the changes in stability system is the lobby of the structure. The lobby has a height of two floors, 6.10\(m\). In about half of the area a floor is present, the rest of the lobby has a double floor to ceiling height. Because the lobby is a space that has to be inviting for users and a lot of transport of people takes place there, the shear walls of the original design had to be removed. In the original design, the shear walls are replaced by partial walls and large columns. The perimeter of the lobby consists of two open and two closed sides.

Because there are no shear walls present in the redesign of the Zalmhaventoren and the façade of the structure plays a key part in the transfer of lateral forces to the foundations the plan of the lobby changes quite dramatically. This can be seen in Figure 46.

The image shows that a lot of columns and walls are removed from the areas where the shear walls used to
be in the original design. This gives the lobby a more open plan which is beneficial for an area where so much traffic takes place. However, because the tube system has to transfer the forces to the foundation, more columns proved necessary in the front façade. To transfer the high normal forces in the columns above this section to these columns extra high beams have been used. In the open façade on the side of the building the columns of the lobby were aligned with the columns that are located above the lobby, so that these beams would not be necessary. Despite these changes, the area around the lobby remains the place where peak stresses appear, as will be shown in section 7.4. A three-dimensional image of the two open sides of the lobby are shown in Figure 46.

6.2 Connections

Because in this part of the thesis not only comparisons between different models are made, as was done in the parameter research, it is important that the stiffness values of the connections are not only in the right order of magnitude, but actually the correct value. Therefore a more intricate calculation for the connection stiffnesses will be made.

6.2.1 Connections used

As in the parameter research section, the following connections are used:

- Unprofiled reinforced horizontal connections
- Open non-structural vertical connections
- Interlocking Halfway Connections (IHC) in the corners

6.2.2 Horizontal Connections

For the horizontal connections both the normal and shear stiffness have to be determined. Both these derivations are based on the work of van Keulen [35] which has also been used by Ten Hagen [29].
Figure 45: The changes made to the floor plan of the lobby of the Zalmhaventoren during the redesign. The parts in red are removed in the redesign, the parts in blue are added.

**Normal Stiffness** $k_y$

The normal stiffness is derived using the following equations:

\[ F = k \cdot \Delta L \]  
\[ \epsilon = \frac{\Delta L}{L} \]  
\[ \sigma = \epsilon \cdot E \]  
\[ F = \sigma \cdot A \]  

Which can be combined in the following way:

\[ 2&3: \quad \Delta L = \frac{\sigma \cdot L}{E} \]  
\[ 1&5: \quad F = \frac{k \cdot \sigma \cdot L}{E} \]  
\[ 4&6: \quad k = \frac{A \cdot E}{L} \]  

In which $k$ is the connection stiffness $[N/mm]$, $A$ is the considered area $[mm^2]$, $E$ is the Young's modulus of the mortar $[N/mm^2]$ and $L$ is the connection thickness $[mm]$. Because AxisVM works with the the unit $kN/m/m$ the
Shear stiffness $k_x$

The derivation of the shear stiffness starts with the equation for shear at the interface between concrete cast at a different time found in Eurocode NEN-EN 1992 1-1 [4]. Which reads:

$$v_{Rdi} = \sigma_n + \rho f_{yd}(\mu \sin \alpha + \cos \alpha) \leq 0.5 \nu f_{cd}$$

If it is assumed that the connection is loaded in compression and the reinforcement is perpendicular to the concrete interface, the equation simplifies to:

$$v_{Rdi} = \mu \sigma_n + \mu \rho f_{yd} \leq 0.5 \nu f_{cd}$$

in which $v_{Rdi}$ is the design shear resistance at the interface $[N/m^2]$, $\mu$ is a factor dependent on the smoothness of the interface (here: $\mu = 0.5$ [1]), $\sigma_n$ is the minimum normal stress at the interface $[N/m^2]$, $\rho$ is the fraction between the area of the reinforcement and the total area investigated [1], $f_{yd}$ the design yield strength of the reinforcement $[N/m^2]$, $\nu$ is the strength reduction factor for concrete in shear ($\nu = 0.6(1 - \frac{f_{ck}}{250}) [1]$), $f_{ck}$ the characteristic compressive strength of the concrete $[N/m^2]$ and $f_{cd}$ the design compressive strength of the concrete $[N/m^2]$.

With this value, in combination with the formula of Straman, a connection stiffness can be found:

$$k = \frac{v_{Rdi}}{\delta_x}$$

in which $k_x$ is the shear stiffness of the connection $[N/m]$ and $\delta$ is the slip of the connection $[mm]$. When this value for the connection stiffness is multiplied by the thickness of the element $[mm]$ and multiplied by a thousand, it leads to the unit used in AxisVM, $kN/m/m$:

$$k_x = \frac{k \cdot t}{1000}$$

6.2.3 Vertical Connections

The vertical connections within the structure are assumed to be non-structural and have a value of zero for the connection stiffness in the model within AxisVM.
6.2.4 Corner Connections

The derivation of the stiffness of the corner connections show some similarities with the derivation of the stiffness of the horizontal connections. For both the normal and the shear stiffness of the connection the same basic equations are used, although they are slightly modified. However, calculations for the tensile stiffness of the connection are here introduced.

**Normal stiffness \( k_z \) compression**

For the normal stiffness the following equation can be used:

\[
 k_z = \frac{A \cdot E}{L}
\]

Note that the subscript \( z \) here refers to the global coordinates where the subscript of the \( k_y \) in the previous section on horizontal connections refers to the local \( y \) coordinate. The direction of these stiffnesses are the same, but are different because of the way they are used in the FEM-program, which will be explained in an upcoming section.

In the equation the value for \( A \) is given by the total area of the connection, which in the case of a corner connection is the wall thickness squared: \( A = t_{wall}^2 \)

**Normal stiffness \( k_z \) tension**

Due to the loading in the structure, not all corner connections might be experiencing a compression force. Therefore, also the tension stiffness in the global \( z \) direction has to be calculated. As is explained in the work by van Keulen [35], this stiffness can be calculated by the following expression:

\[
 \frac{1}{k_{z,tension}} = \frac{1}{k_{s,\text{end},1}} + \frac{1}{k_{s,\text{end},2}} + \frac{1}{k_{\text{mortar}}}
\]

In which \( k_{s,\text{end},x} \) is the pull out stiffness of the reinforcement rods (of which there are two) and \( k_{\text{mortar}} \) the stiffness of the mortar. The pull out stiffness is derived by the simplified approach as described in fib bulletin 43: Structural connections for precast concrete buildings [1]. This approach starts with an equilibrium condition between the total bond resistance and the applied tensile force to the bar.

\[
 \sigma_s \cdot \pi \phi^2 = \tau_{bm} \cdot \pi \phi \cdot l_t
\]

in which \( \sigma_s \) is the steel stress at the active end \([N/mm^2]\), \( \phi \) the diameter of the reinforcement rod \([mm]\), \( l_t \) the mobilized transmission length of the rod \([mm]\) and \( \tau_{bm} \) the (in this simplification assumed) average bond stress along the transmission length \([N/mm]\). Combined with the equation for the end slip:

\[
 s_{\text{end}} = \frac{1}{2} \frac{\sigma_s}{E_s} \cdot l_t + \frac{\sigma_s}{E_s} \cdot 2 \cdot \phi
\]

this gives:

\[
 s_{\text{end}} = \frac{1}{8} \frac{\sigma_s^2}{E_s} \cdot \phi + \frac{\sigma_s}{E_s} \cdot 2 \cdot \phi
\]

in which \( E_s \) is Young’s Modulus of the steel \([N/mm^2]\).

\( \tau_{bm} \) is given by \( \tau_{bm} = \tau_{b,max} \cdot \alpha_t \) in which \( \alpha_t \) is a factor depending on the bar diameter and \( \tau_{b,max} \) is the maximum bond stress given by the following relation if the bond conditions can be considered good: \( 2.5 \sqrt{f_{cc}} \).

In turn, \( f_{cc} \) is given by \( f_{cm} + 8 MPa \).

The last step is to determine the pull out stiffness using this equation:

\[
 k_{s,\text{end}} = \frac{f_y \cdot A_s}{s_{\text{end}}}
\]

in which the force at the yielding point of the steel is used with respect to the pull out length \( s_{\text{end}} \).

**Shear stiffnesses \( k_x \) and \( k_y \)**

For the shear stiffness of the corner connections it is important where in the structure the connection is situated. The Interlocking Halfway Connections connects the elements together on both floor level and in between two floors. The influence of this floor on the shear stiffness is important. If a floor is present and in the same plane as the connection, the stiffness in this plane can be considered to be very high, nearing infinity. The parameters of the connection itself do not have a noticeable effect in this case. The different locations of the connections can be seen in Figure 47.
Precast Concrete in Framed Tube High-Rise Structures

Figure 47: The locations of the Interlocking Halfway Connections and their influence on the shear stiffness

For the value of the shear stress in the connections where no floor is present, the same equation is used as was used for the shear stress in horizontal connections:

\[ v_{Rdi} = \mu \sigma_n + \mu \rho f_{yd} \leq 0.5 \nu f_{cd} \]

Again the formula of Straman is used in which the design shear resistance at the interface is divided by the slip of the connection:

\[ k_x = \frac{v_{Rdi}}{\delta_x} \left[ \frac{N}{mm^2} \right] \]

This value can now be multiplied by the area of the connection, which is the wall thickness squared: \( A = t_{wall}^2 \). The unit remaining will then be \( \frac{N}{mm} \) which is equal to the \( \frac{KN}{m} \) which are used in the AxisVM model.

For the calculation of the shear stiffness the same equation can be used for connections in which compression and tension are present. The difference between the two calculations lies in the change of the parameter for \( \sigma_n \).

For connections under compression this \( \sigma_n \) will be given a positive value, for connections in tension \( \sigma_n \) will be given a value of zero.

6.2.5 Calculation

Assumptions and fixed parameters

In this subsection the stiffness values will be determined for all connections within the structure. As mentioned earlier, there are multiple types of connections within the structure: vertical, horizontal and corner connections. But in addition to this, a distinction has to be made with respect to the location of the connection in the structure. For the normal stiffness the Young’s modulus of the mortar is estimated by the value of the concrete around it. And although most of the elements are made from the same concrete type (C50/60), the values used in the model are based on whether the concrete is likely to be uncracked (parts in compression) or cracked (parts in tension). The values are calculated as followed: \( E_{uncracked} = \frac{37000}{1.5} = 24666 \frac{N}{mm^2} \) or \( E_{cracked} = \frac{37000}{3} = 12333 \frac{N}{mm^2} \). These values for \( E_{uncracked} \) and \( E_{cracked} \) are estimations. A fairly safe assumption which is used in the modeling phase is that all lintels could possibly experience tension and therefore are given the value of \( E_{cracked} \) for its Young’s Modulus. The columns on the other hand are experiencing a compression force and therefore use the value of \( E_{uncracked} \) for their Young’s Modulus.

This difference between possible tension and compression forces is also important for the shear stiffness of the connection. The value for the shear resistance is, as given earlier:

\[ v_{Rdi} = \mu \sigma_n + \mu \rho f_{yd} \leq 0.5 \nu f_{cd} \]
The value $\sigma_n$ represents the normal stiffness in the connection and is only in use if there is a compression force present in the connection. If there is tension in the connection the value for $\sigma_n$ is assumed $\sigma_n = 0$. In this case, the reinforcement provides the shear stiffness of the connection through the argument $\mu \rho f_y d$ in the equation. The same distinction between the lintels and columns used for the different Young’s Moduli can now be made. For areas where the value of $\sigma_n$ is equal to zero (the lintels) or is given a non-zero (the columns) value. A visible representation of these distinctions can be found in Figure 48.

The stiffness of the elements is dependent on several aspects, one of which is the reinforcement percentage in the connection. In Figure 49 the placement of the reinforcement is given for the horizontal and the corner connection. This placement will be used throughout the entire structure.

The percentages can be calculated in the following way. For the horizontal connection there is a reinforcement bar of $\phi 25\text{mm}$ each $150\text{mm}$. One rod has an area of $A_{rod,25\text{mm}} = \frac{1}{4} \cdot \pi \cdot 25^2 = 491\text{mm}^2$. Each meter $1000 \div 150 = 6.66$ of these rods are present. The reinforcement percentage now becomes: $\rho_{hor} = \frac{6.66 \cdot 491\text{mm}^2}{1000\text{mm} \times 400\text{mm}} = 0.818\%$.

The calculations for the corner connections are even simpler. $\rho_{cor} = \frac{1}{4} \pi 32\text{mm}^2 = 0.50\%$

Some parameters will be fixed for the calculation of these connection stiffnesses. These will be presented below:
• Concrete Class C50/60 or C80/95 (C50/60 will be used in the sample calculations)
• Element height \( h_{element} = 3.05 \) m
• Element thickness \( t_{element} = 0.4 \) m
• Connection thickness \( L = 20 \) mm
• Young’s modulus of the mortar \( E_{C50/60,uncracked} = \frac{37000}{1.5} = 24666 \frac{N}{mm^2} \) or \( E_{C50/60,cracked} = \frac{37000}{3} = 12333 \frac{N}{mm^2} \)
• Reinforcement percentage \( \rho_{hor} = 0.818\% \) and \( \rho_{cor} = 0.5\% \)
• Design yield strength reinforcement \( f_y = 435 \frac{N}{mm^2} \)
• Slip in all connections \( \delta_x = 1 \) mm
• Normal stress in connection \( \sigma_n = 5 \frac{N}{mm^2} \) if compression forces present or \( \sigma_n = 0 \frac{N}{mm^2} \) if there is not.
• Strength reduction factor \( \nu = 0.6(1 - \frac{50}{250}) = 0.48 \)

**Normal stiffness of the horizontal connection, cracked concrete,** \( k_y \)
The first calculation is the normal stiffness of the horizontal connection with cracked concrete, which is present in the lintels of the tube elements:

\[
k_y = \frac{A \cdot E_{cracked}}{L} = \frac{(1000 \times 400) \cdot 12333}{20} = 2.47 \cdot 10^8 \frac{N}{mm} = 2.47 \cdot 10^8 \frac{kN}{m} \\
k_y = \frac{2.47 \cdot 10^8}{1m} = 2.47 \cdot 10^8 \frac{kN}{m/m}
\]

**Normal stiffness of the horizontal connection, uncracked concrete,** \( k_y \)
The calculation of the normal stiffness in a horizontal connection with uncracked concrete is very similar. This stiffness value is used in the column parts of the tube elements as well as in most core elements:

\[
k_y = \frac{A \cdot E_{uncracked}}{L} = \frac{(1000 \times 400) \cdot 24666}{20} = 4.93 \cdot 10^8 \frac{N}{mm} = 4.93 \cdot 10^8 \frac{kN}{m} \\
k_y = \frac{4.93 \cdot 10^8}{1m} = 4.93 \cdot 10^8 \frac{kN}{m/m}
\]

**Normal stiffness in the corner connection, compression,** \( k_z \)
The calculation of the normal stiffness of the corner connection in compression is similar to the previous calculations on the normal stiffness in the horizontal connections. For this calculation it is assumed that the concrete is uncracked:

\[
k_z = \frac{A \cdot E_{uncracked}}{L} = \frac{(400 \times 400) \cdot 24666}{20} = 1.97 \cdot 10^8 \frac{N}{mm} \\
k_z = 1.97 \cdot 10^8 \frac{kN}{m}
\]

**Normal stiffness in the corner connection, tension,** \( k_z \)
To determine the normal stiffness in tension for the corner connection, the pull out stiffness and the stiffness of the mortar have to be found. The pull out stiffness can be determined after the values for \( \tau_{bm} \) and \( s_{end} \) are found. \( \tau_{bm} \) is given by \( \tau_{bm} = \tau_{b,max} \cdot \alpha_t \). In which \( \tau_{b,max} \) is found through \( \tau_{b,max} = 2.5\sqrt{f_{cm}} \). In turn, \( f_{cm} \) is given by \( f_{cm} = f_{cm,C50/60} + 8MPa = 50 + 8 = 58MPa \). Which leads to: \( \tau_{b,max} = 2.5\sqrt{58} = 19.04 \frac{N}{mm} \) and
\[ \tau_{bm} = 19.04 \cdot 0.45 = 8.58. \]

The next step is calculating \( s_{end} \) through:

\[
s_{end} = \frac{1}{8} \frac{\sigma^2 \cdot \phi}{E_s \cdot \tau_{bm}} + \frac{\sigma_s}{E_s} \cdot 2 \cdot \phi = \frac{1}{8} \frac{435^2 \cdot 32}{210000 \cdot 8.58} + \frac{435}{210000} \cdot 2 \cdot 32 = 0.553 \text{mm} \]

With this value now the pull out stiffness can be determined:

\[
k_{s,end} = \frac{f_y \cdot A_s}{s_{end}} \]
\[
A_s = \frac{1}{4} \pi \cdot 32^2 = 804.25 \text{mm}^2 \]
\[
k_{s,end} = \frac{435 \cdot \frac{N}{\text{mm}^2} \cdot 804.25 \text{mm}^2}{0.553 \text{mm}} = 632636 \frac{kN}{m} \]

The stiffness of the mortar can be found with:

\[
k_z = \frac{A \cdot E_{cracked}}{L} = \frac{(400 \times 400) \cdot 12333}{20} = 9.86 \cdot 10^7 \frac{N}{\text{mm}} \]
\[
k_z = 9.86 \cdot 10^7 \frac{kN}{m} \]

Now the normal stiffness of the connection can be calculated through

\[
\frac{1}{k_{z,tension}} = \frac{1}{k_{s,end,1}} + \frac{1}{k_{s,end,2}} + \frac{1}{k_{\text{mortar}}} = \frac{1}{632636} + \frac{1}{632636} + \frac{1}{9.86 \cdot 10^7} \rightarrow k_{z,tension} = 3.15 \cdot 10^5 \frac{kN}{m} \]

**Shear stiffness in the horizontal connection, lintels, \( k_x \)**

For the calculation of the shear stiffness in the horizontal connection there are two modes, as explained earlier. For the one without any normal force in the connection the calculation reads:

\[
v_{Rdi} = \mu \rho f_{\text{rel}} \leq 0.5 \nu f_{ed} \]
\[
= 0.5 \cdot 0.00818 \cdot 435 \leq 0.5 \cdot 0.48 \cdot 33.33 \]
\[
= 1.78 \leq 8 \]
\[
k = \frac{v_{Rdi}}{\delta_x} = \frac{1.78}{1} \]
\[
k_{AxisVM} = k \cdot t_{\text{element}} \cdot 10^3 \]
\[
= 1.78 \cdot 400 \cdot 10^3 \]
\[
= 7.12 \cdot 10^5 \frac{kN}{m/m} \]

**Shear stiffness in the horizontal connection, columns, \( k_x \)**

For the shear stiffness of the horizontal connection with normal forces an important assumption has been made. Looking at the expression for the part of the stiffness associated with normal force, \( \mu \sigma_n \), it is clear that its value, along with the value of \( \sigma_n \), is different for different places in the structure. Ideally, one would make an intricate iterative calculation to determine the stiffnesses of all different connections. This however would be very time consuming and improve the results of the calculation only slightly. Therefore the assumption was made that through the entire structure, for all load cases, \( \sigma_n = 5 \frac{N}{\text{mm}^2} \) is present in the columns. This overestimates the stiffness in the higher parts of the structure, but underestimates it in the lower parts. Because the stiffness in the lower parts of the structure is more important than the stiffness in the top, this assumption is most likely
safe to make. However, in practice, such an assumption would have to be checked. The calculation reads:

\[
v_{Rdi} = \mu \sigma_n + \mu \rho f_{yd} \leq 0.5 \nu f_{cd}
\]

\[
= 0.5 \cdot 5 + 0.5 \cdot 0.005 \cdot 435 \leq 0.5 \cdot 0.48 \cdot 33.33
\]

\[
= 4.28 \leq 8
\]

\[
k = \frac{v_{Rdi}}{\sigma_n} = \frac{4.28}{1}
\]

\[
k = \frac{4.28}{N/mm^2}
\]

\[
k_{AxisVM} = k \cdot t_{element} \cdot 10^3
\]

\[
= 4.28 \cdot 400 \cdot 10^3
\]

\[
= 1.71 \cdot 10^6 kN/m/m
\]

For both shear stiffnesses, the same slip of \(\delta_x = 1\) mm was used. The value of this slip was based on an article by van Keulen [35], which has been described in the literature study of this thesis. It gives different slip values for the different modes contributing to the connection stiffness. For the \(\mu \sigma_n\) a slip lower than \(\delta_x = 1\) mm could be used in most cases, but because the two different stiffness values are present on the same element, giving them the same slip value makes sense and can be considered conservative.

**Shear stiffness in the corner connection, compression,** \(k_y\) and \(k_z\)

The calculation of the shear stiffness in the corner connection is very similar to the calculation of the shear stiffness in the horizontal connection. For the corners in compression, the same value of \(\sigma_n = 5 \frac{N}{mm^2}\) is used throughout the structure and in all load cases. This gives

\[
v_{Rdi} = \mu \sigma_n + \mu \rho f_{yd} \leq 0.5 \nu f_{cd}
\]

\[
= 0.5 \cdot 5 + 0.5 \cdot 0.005 \cdot 435 \leq 0.5 \cdot 0.48 \cdot 33.33
\]

\[
= 3.59 \leq 8
\]

\[
k = \frac{v_{Rdi}}{\sigma_n} = \frac{3.59}{1}
\]

\[
k = \frac{3.59}{N/mm^2}
\]

\[
k_y = k_z = k \cdot A
\]

\[
k_y = k_z = 3.59 \cdot 400^2
\]

\[
k_y = k_z = 1.75 \cdot 10^5 \frac{N}{mm} = kN/m
\]

**Shear stiffness in the corner connection, tension,** \(k_y\) and \(k_z\)

For the corner connections in tension, the calculation is similar to the previous one, with the only difference being that \(\sigma_n = 0\). This leads to:

\[
v_{Rdi} = \mu \rho f_{yd} \leq 0.5 \nu f_{cd}
\]

\[
= 0.5 \cdot 0.005 \cdot 435 \leq 0.5 \cdot 0.48 \cdot 33.33
\]

\[
= 1.09 \leq 8
\]

\[
k = \frac{v_{Rdi}}{\sigma_n} = \frac{1.09}{1}
\]

\[
k = \frac{1.09}{N/mm^2}
\]

\[
k_y = k_z = k \cdot A
\]

\[
k_y = k_z = 1.09 \cdot 400^2
\]

\[
k_y = k_z = 1.75 \cdot 10^5 \frac{N}{mm} = kN/m
\]

**Table of all used stiffnesses in the connections**

In the following table all stiffnesses used for both concrete classes and all connections will be given as an overview.
of this section.

<table>
<thead>
<tr>
<th>Connection</th>
<th>C50/60</th>
<th>C80/95</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal stiffness of the horizontal connection, cracked concrete, $k_y$</td>
<td>$2.47 \cdot 10^6$</td>
<td>$2.8 \cdot 10^6$</td>
</tr>
<tr>
<td>Normal stiffness of the horizontal connection, uncracked concrete, $k_y$</td>
<td>$4.93 \cdot 10^6$</td>
<td>$5.6 \cdot 10^6$</td>
</tr>
<tr>
<td>Normal stiffness of the corner connection, compression, $k_z$</td>
<td>$1.97 \cdot 10^6$</td>
<td>$2.24 \cdot 10^6$</td>
</tr>
<tr>
<td>Normal stiffness of the corner connection, tension, $k_z$</td>
<td>$3.15 \cdot 10^6$</td>
<td>not present in structure</td>
</tr>
<tr>
<td>Shear stiffness of the horizontal connection, lintels, $k_x$</td>
<td>$7.12 \cdot 10^6$</td>
<td>$7.12 \cdot 10^6$</td>
</tr>
<tr>
<td>Shear stiffness of the horizontal connection, columns, $k_x$</td>
<td>$1.71 \cdot 10^6$</td>
<td>$7.12 \cdot 10^6$</td>
</tr>
<tr>
<td>Shear stiffness of the corner connection, compression, $k_z$ and $k_y$</td>
<td>$5.75 \cdot 10^6$</td>
<td>$5.75 \cdot 10^6$</td>
</tr>
<tr>
<td>Shear stiffness of the corner connection, tension, $k_x$ and $k_y$</td>
<td>$1.75 \cdot 10^6$</td>
<td>$1.75 \cdot 10^6$</td>
</tr>
</tbody>
</table>

6.3 Model Elements

In this section the elements that make up the FEM-model in AxisVM will be discussed. In the last section
the description of the structure was given, but to model a building certain assumptions and choices have to be made. The right balance has to be found between a very detailed calculation and one that does not take to long to compute. An important choice that was made early on, was to model the structure in a 3-dimensional model. The reason behind this is that the program, AxisVM, is very capable of doing 3-dimensional models and that the computational power available was not a restriction. Therefore it would be of no use to model the 3-dimensional structure into a 2-dimensional one, making assumptions that could make interpretation of the model results harder later on.

6.3.1 Prefabricated Concrete Shear Walls

The prefabricated wall elements in the building are modeled with the standard 3-dimensional domain element, the shell. The shell elements in AxisVM can experience both in plane and out of plane effects and are given a thickness and a structural material. Normal rectangular shells, for instance the elements in the core, are the easiest to create. For the creation of the elements with holes in them, present in the tube of the structure, there are two possibilities. Either create a rectangular domain and create a hole with the Holes option, or creating the elements with multiple shells, manufacturing the holes by leaving out those parts. Although early on the first option was chosen, it was found later on that the second option was better because it gave the opportunity to give different parts of the element different Young’s moduli and different stiffnesses for the connections. There is no difference between the two options in structural results, because AxisVM will consider two domains without an edge hinge between them to be a cast-in-place connection. The build up of a tube element and its surrounding elements can be seen in Figure 50.

As mentioned in the section on connections, the Young’s modulus of each element is estimated to be different for parts that are most likely cracked and parts that are most likely uncracked. The division of the element into smaller shell elements gives the freedom to give all the lintels the cracked Young’s modulus $E_{cracked} = \frac{E}{17}$ and all other shell elements the uncracked Young’s modulus $E_{uncracked} = \frac{E}{17}$.

6.3.2 Corner connections

The way corner connections are modeled in AxisVM in this phase of the thesis is different from the one used in the parameter study. Still a halfway interlocking connection (IHC) is modeled, but not as a smeared stiffness over the entire height of the element, but instead as a single spring at the interface location. This method requires some more complex shaped elements near the corners and the addition of infinitely stiff elements to transfer the forces from the single point of the spring into those elements. The advantage of this method is that the stiffness of the mortar interface and the stiffness of the shear key of the element are in the correct spot in the model, rather than both smeared over the entire height of the element. A picture of the corner connection in AxisVM can be found in Figure 51. The unit of a linear stiffness in a spring is $[kN/m]$. In the modeling of the springs in AxisVM, a choice was made for the global axes for reference. This means that the stiffness in the $z$-direction is the normal stiffness and the stiffnesses in $x$- and $y$-direction are the shear stiffnesses. They are given the correct value as determined in the previous section. The rotational stiffnesses are all equal to zero: $k_{zz} = k_{yy} = k_{xx} = 0$

6.3.3 Other Connections

The other connections in the model are modeled by using an edge hinge. These elements connect two shell elements together over the entire length they are connected and use a unit of $[kN/m/m]$. The stiffnesses in edge hinges are given by a local reference, instead of a global one. The orientation of the axes can be seen in Figure 52. This means that in the horizontal connections, the shear stiffness is entered in the $k_x$ direction and the normal stiffness in the $k_y$ direction. For the $k_z$, an arbitrary value of $1.0 \cdot 10^7$ is used to ensure the stability of...
Figure 50: One shear wall element in the tube and its neighboring elements. One actual element is made up of smaller elements to be able to give them different Young’s moduli.

Figure 51: A picture explaining the way the corner connections are modeled in AxisVM for this part of the thesis, with a single spring and infinitely stiff elements.

The model calculation. The rotational stiffnesses are again all equal to zero: $k_{xx} = k_{yy} = k_{zz} = 0$. As has been explained in the earlier section on connections, the horizontal connection stiffness is different in different places in the structure. They depend on whether they are in a lintel or a column and are within an area with C50/60 as a concrete class or C80/95. The values for these different cases can be found in the table in the previous section. The vertical connections are non-structural and are given a value of $K_x = K_y = K_z = K_{xx} = K_{yy} = K_{zz} = 0$. These values have to be included in an edge hinge because the standard connection assumed between two
6.3.4 Floors

Often in models of tube-in-tube structures the floors are not modeled at all. The designer has to make sure that the correct amount of lateral load is placed on the correct stabilizing element in another way, while at the same time preventing the outer tube and core having different lateral deflections. The reason why the floors are often left out is that they are not designed to help resist the lateral loading of a structure like this. If they were to be included in the lateral load resist part of the structure, they would have to be connected with some moment resistance in the connection. This would most likely cause cracking in the concrete near these connections. To get the best of both worlds, one would like to have the effect of in plane stiffness of the floors to prevent different lateral deflections between core and tube and to transfer the correct amount of lateral loading to the stabilizing elements. But now without the out of plane stiffness which is likely to cause cracking in the floor and does not contribute in practice. However it is not possible to use membrane elements (which only have an in plane stiffness) in the same model as shell elements in AxisVM. In order to get the same results, a shell with a low out of plane stiffness is used with the boundary conditions preventing any moment from occurring by using edge hinges around the entire perimeter of the floor. The first try of this solution however produced some problems. These can be seen in Figure 53. Although the boundary conditions were made to prevent high moments from occurring in the floors, there still are moments present. They occurred due to the difference in vertical deflection $e_Z$ between the core and the tube of the structure. In practice, this difference in settlement is nullified by the use of hydraulic jacks or other in work solutions. These kind of solutions however cannot be (easily) modeled in AxisVM. So however these moments would probably be not present in the practice, the fact that they are in the model results poses a problem. It cannot be ruled out that the floors do not provide an important element in the lateral stiffness of the structure. Therefore, a solution in the model was used to prevent these moments from occurring there. By linking the corners of the core to the corresponding corners
of the tube by a hinge in the floor, the moments were reduced to be almost zero. By using an edge hinge with $k_x = k_y = k_z = 1.0 \cdot 10^7 \text{kN}/m/m$ the shear and normal forces could still be transferred in the floor, without any moments occurring there. An image of this solution can be found in Figure 54. In the figure it can also be seen how the floor is detailed near the corners of the tube and the core of the structure. In both cases the floor does not extend all the way to the corner. This was chosen to make sure that the floors did not influence the corner connection in an unexpected way.

6.3.5 Foundation

To get a fair comparison later in this thesis, the values of the foundation of the model of ten Hagen [29] were used. This model is a simplification of the original design, that used diaphragm walls and a $1m$ thick foundation slab. Instead, he modeled beams underneath all of the elements connected to the foundation level of the structure with a dimension of $1m$ thick and $2m$ deep. These beams have in turn be given a line foundation with the following values: $K_x = K_y = 1.0 \cdot 10^7 \text{kN}/m$ and $K_z = 6.97 \cdot 10^5 \text{kN}/m$, all in a global coordinate system. All the rotational stiffnesses are equal to zero: $K_{xx} = K_{yy} = K_{zz} = 0$. The same approach has been used for the foundations in the model of the prefabricated tube-in-tube structure.

6.4 Loading

Another important part of the structural model is the loading on the different parts of the structure. These loads have different origins, such as self weight, live floor load or wind loading. In this section all the different loads and how they are used in the model will be explained.

6.4.1 Dead load

The dead load of the structure can be divided into three parts:

- Self weight of the prefabricated concrete wall elements
- Self weight of the floors and accompanying installations
- Self weight of the façade and balconies

The self weight of the prefabricated concrete wall elements in the tube and core of the building is the most simple load case in the AxisVM model. Elements can be given a **Self Weight** value which automatically applied at the right position in the model and in the right direction, which is negative in the $z$-direction. In the materials tab the concrete can be given a density $\rho$, which is used to determine the size of the loads. For all concrete including reinforcement the value used is $\rho = 2500 \text{ kg/m}^3$.

The next dead load discussed will be the self weight of the floor and accompanying installations such as ventilation and heating ducts. The weight of these floors is estimated to be the weight of a monolith floor of $300mm$ thickness, although a weight saving option like a Bubbledeck floor will be used. The 25-30%
weight reduction of such a floor type is balanced out by the before mentioned installations. The weight per 1m² floor area can now be calculated using a density for the floor and the floor thickness estimated:

\[ q_{sw, floor} = \rho \cdot t_{floor} = 24 \text{kN/m}^3 \cdot 0.3 \text{m} = 7.2 \text{kN/m} \]

The way this load is applied is different from the self weight of the structural wall elements. As mentioned in an earlier section, the floors in the model are only there to distribute the horizontal forces from the wind load to the correct lateral stability system. In the model, the floors should encounter no bending moments. In addition to the boundary conditions discussed in the earlier section, there can be no outside force on the floors perpendicular to the floors themselves. Therefore the loads from the floors are directly applied to the vertical elements bringing these forces to the foundations: the tube and core wall elements. Here, the forces are applied as a combination of mostly line loads and some point loads. In Figure 55 a floor plan of the structure can be seen, with color coded areas and corresponding line and point loads where the loads on these parts are applied. The assumption made is that the floor loads will be transferred to the foundation by the closest vertical load bearing element. For instance: a red area is 14.4m × 4.375m. The load on the red line will be

\[ q_{redline} = A_{red} \cdot q_{sw, floor} = (14.4 \times 4.375) \times 7.2 = 31.5 \text{kN/m} \]

Although this is not the exact reality of how the loads are applied, this is an estimation which is close. This situation, where the area loads are converted into line and point loads is present on each level of the structure.

The last part of the self weight of the structure is the self weight of the façade. Because a façade design is not part of this thesis, a rough estimation suffices. For a curtain wall façade, the self weight is estimated as

\[ q_{sw, facade} = 1.5 \text{kN/m} \]

This load is applied as a line load on each level around the circumference of the structure. The line load can be calculated as followed:

\[ q_{façade} = q_{sw, facade} \times h_{element} = 1.5 \times 3.75 = 5.625 \text{kN/m} \]

In addition to this façade load, the dead load of the balconies was also added to the load case. This load is added as a line load located on the support of the balcony. Only a vertical load has been used and not a moment. The impact of the moment on the behavior of the entire structure was estimated to be negligible. The dead load used for

![Figure 55: A floor plan of the structure with color coded areas and corresponding line and point loads where the loads on these parts are applied](image-url)
the balconies is: \( q_{\text{balcony}} = \rho \cdot t_{\text{floor}} \cdot w_{\text{balcony}} = 24 \cdot 0.3 \cdot 2.0 = 14.4 \text{kN/m} \)

### 6.4.2 Live load

The live load of the structure can be divided into two parts:

- Live load on the floors (including balconies) of the structure
- Wind loading

For the live floor load on the floors of the structure, the function of the building has to be known. Because the tube-in-tube structure was deemed most fit for an office function, this function was chosen in an earlier section. NEN-EN-1991 1-1 gives a value of \( 2.0 - 3.0 \frac{\text{kN}}{\text{m}^2} \) for an office function. To be on the conservative side and to be able to change functions later in the lifetime of the structure, \( 3.0 \frac{\text{kN}}{\text{m}^2} \) was chosen here. As with the dead load of the floors, these loads are not applied to the floors themselves, but in a similar fashion applied to the vertical load bearing walls of the tube and core using Figure 55.

Probably the most important load on a high rise structure is the wind loading, because the lateral deflection is often a governing factor in structures with a high slenderness. The determination of the wind load on a structure is usually done by using NEN-EN 1991-1-4 which is the Dutch version of the Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions [3]. Because the original Zalmhaventoren was going to be built in Rotterdam, the Dutch wind Area II of the national annex will be used in an urban environment. For structures this tall however, using just the NEN-code will not suffice. On the one hand because the values for the wind thrust per \( \text{m}^2 \) \( (q_p) \) are only given for heights up to 200m, on the other hand because the force coefficient \( c_f \) is better defined in the so called ”Hoogbouwconvenant” [5], a Dutch annex for high rise structures. The combination of both will give the correct wind load on the structure.

For the force on a structure, the following equation is used in the Eurocode:

\[
F_w = c_s c_d \cdot c_f \cdot q_p \cdot A
\]

in which \( F_w \) is the wind force on the structure \([\text{N}]\), \( c_s c_d \) the structure factor \([-\]), \( c_f \) the force coefficient of the structural part\([-\]), \( q_p(z) \) the wind thrust per \( \text{m}^2 \) dependent on the height \( z \) \([\text{kN}/\text{m}^2]\) and \( A \) the affected area. For application in the model a line load is used. To get to the right unit \( q_{\text{wind}}(z) = \frac{F_w}{1 \text{m}} \) will be used \([\text{kN}/\text{m}]\).

The structure factor \( c_s c_d \) for a structure as tall as this one is a hard coefficient to calculate. It depends on the dimensions of the structure \( c_s \) and the dynamic factor \( c_d \) to account for the changing winds on structures. The value calculated by ten Hagen [29] will be used because the structure has the same dimensions and both structures are prefabricated concrete structures. He calculated it to be \( c_s c_d = 1.11 \).

The value for \( c_f \) is derived from the information in the ”Hoogbouwconvenant”. It specifies in a table the factor for rectangular structures with a slenderness that is less than 7 and different width to depth ratios. Because the structure in this thesis has a slenderness of \( \frac{29.5 \text{m}}{30 \text{m}} = 0.98 \) the values in the table can be used. The ratio between the width and depth of this structure is very close to 1, because the floor plan has the dimensions 30m \( \times \) 29.5m. This results in a value of: \( c_f = 1.47 \).

The considered area \( A \) is dependent on how the wind loads are applied on the structure in the model. Because they are placed as line loads on each floor, the considered area is an area of 3.05m(2.05m) \( \times \) 1m. So for the entire structure: \( A = 3.05 \text{m}^2 \). For the value of the wind thrust per \( \text{m}^2 \), \( q_p(z) \), it is important to note that for structures as tall as this one, the Eurocode asks for a different value on different heights of the structure. If the height of the structure is more than twice its width or depth the figure in Figure 56 applies. From the

\[ h > 2b \]

Figure 56: A changing \( q_p \) has to be used for different height according to the Eurocode [3]

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bottom to the top there are three areas. The first has a height of $b$, the width of the structure. Here, the value for $q_p$ is set at $q_p(b)$. In this case, for a structure in wind Area II and in an urban environment (which is the case for Rotterdam), the value is $q_p(30) = 1.03 \text{kN/m}$. In the next area, the engineer has to divide the area into multiple strips which have the value of $q_p(z\text{strip})$. The size of these strips is not specified in the code and for this structure strips of 3 floors are used for this area of the tower. In the last area of the tower, with a height of $b$, the following $q_p$ has to be used: $q_p(h)$. The value for this structure has to come from the Hoogbouwconvenant, because the structure is taller (if only just) than 200 m. If linear interpolation is used between $q_p(200) = 1.92$ and $q_p(225) = 1.97$, the following value is found: $q_p(201.3) = 1.923$. The Hoogbouwconvenant reads that all values above 150 m can be considered to be outside an urban area, because there is barely any effect of the city on the wind conditions that high off the ground. As an example, the line load $q_{wind}(201.3)$ will be calculated.

$$F_w = c_s c_d \cdot c_f \cdot q_p \cdot A$$

$$= 1.11 \cdot 1.47 \cdot 1.923 \cdot 3.05$$

$$= 9.57 \text{kN}$$

$$q_{wind}(201.3) = \frac{9.57}{1m} = 9.57 \frac{\text{kN}}{m}$$

All this leads to the wind forces applied in the AxisVM model as can be seen in Figure 57. The found values are used for the forces in both the $x$-direction and the $y$-direction. The difference in width of the building $b$ between 30 m and 29.5 m has a too small influence if line loads per floor are used.

### 6.4.3 Load cases

To determine the different load cases that have to be looked into, first some building characteristics have to be looked into. The first one is the expected life-time of the structure. The standard life-time of a structure in the Netherlands is 50 years and there is no reason to differ from this standard. The second building aspect is the consequence class. Consequence classes are used to give structures which have dire consequences if they would fail structurally a higher safety. This makes sense from the point of view that: $\text{Risk} = \text{Consequence} \times \text{Likelihood}$. To reduce the total risk of a structural failure, the likelihood is lowered by the use of consequence classes. For regular office structures, the consequence class used is the average consequence class, class 2. However, NEN 1991 1-7 table A.1 says that this consequence class is only applicable or structures with less than 15 floors. Because the design of the building for this thesis surpasses that number of floors, the highest consequence class, class 3, will be used.

In the Dutch national annex of the Eurocode a table (NEN-EN-1990 A1 + A1/C2 NB.5) [2] describes the load cases that have to be used for the consequence class in the Dutch building setting. This table can be found in Figure 58.

In another table from the same annex the values for $\psi_0$ can be found. The two values that have to be used in this case are:
Precast Concrete in Framed Tube High-Rise Structures

Figure 58: Table NB.5 from NEN-EN-1990 A1 + A1/C2 describing the load cases that have to be used for structures in the Netherlands

- Imposed loads on buildings: Category B: Offices $\psi_0 = 0.5$
- Wind loads $\psi_0 = 0$

Now all the load cases can be determined. Both Serviceable Limit State (SLS) as well as Ultimate Limit State (ULS) will have to be used in different parts of the result analysis. Because the structure is different in the $x$ and $y$-direction multiple load cases are needed for analysis in different directions. In addition to the $x$ and $y$-direction, also the load cases for the $-x$ and $-y$-direction have to be looked into, because the structure is not symmetric at the bottom of the structure. And because the dead load could be either favorable or unfavorable both options have to be looked into. This leads to the following 13 load cases:

- **LC1**: SLS in the $x$-direction: $1.0 \times (I) + 1.0 \times (II) + 1.0 \times (III)$
- **LC2**: SLS in the $-x$-direction: $1.0 \times (I) + 1.0 \times (II) - 1.0 \times (III)$
- **LC3**: SLS in the $y$-direction: $1.0 \times (I) + 1.0 \times (II) + 1.0 \times (IV)$
- **LC4**: SLS in the $-y$-direction: $1.0 \times (I) + 1.0 \times (II) - 1.0 \times (IV)$
- **LC5**: ULS without wind: $1.5 \times (I) + 1.65 \times 0.5 \times (II)$
- **LC6**: ULS in the $x$-direction and unfavorable dead load: $1.3 \times (I) + 1.65 \times 0.5 \times (II) + 1.65 \times (III)$
- **LC7**: ULS in the $x$-direction and favorable dead load: $0.9 \times (I) + 1.65 \times 0.5 \times (II) + 1.65 \times (III)$
- **LC8**: ULS in the $-x$-direction and unfavorable dead load: $1.3 \times (I) + 1.65 \times 0.5 \times (II) - 1.65 \times (III)$
- **LC9**: ULS in the $-x$-direction and favorable dead load: $0.9 \times (I) + 1.65 \times 0.5 \times (II) - 1.65 \times (III)$
- **LC10**: ULS in the $y$-direction and unfavorable dead load: $1.3 \times (I) + 1.65 \times 0.5 \times (II) + 1.65 \times (IV)$
- **LC11**: ULS in the $y$-direction and favorable dead load: $0.9 \times (I) + 1.65 \times 0.5 \times (II) + 1.65 \times (IV)$
- **LC12**: ULS in the $-y$-direction and unfavorable dead load: $1.3 \times (I) + 1.65 \times 0.5 \times (II) - 1.65 \times (IV)$
- **LC13**: ULS in the $-y$-direction and favorable dead load: $0.9 \times (I) + 1.65 \times 0.5 \times (II) - 1.65 \times (IV)$

in which:

- (I) is the dead load on the structure (elements, floors and façade)
- (II) is the live load on the floors of the structure
- (III) is the wind load in the $x$-direction
- (IV) is the wind load in the $y$-direction
6.5 Meshing

The mesh used in the main model for the design building differs from the meshes used in the parameter research in several ways. The first difference is that the mesh has roughly the same size for every element in the structure, whether it is near the top, middle or bottom of the structure. This is opposed to the parameter research, where the shear lag was the most important result and therefore the mesh near the base of the structure was deliberately made smaller. In the main model, all parts of the building could contain the governing element with respect to compression, tension or shear stresses. So the choice was made to give the entire model the same mesh size. Secondly, a triangular mesh was used instead of a quadrangular one. This choice was made when the earlier models with a quadrangular mesh became numerically unstable.

The mesh size of the triangular mesh used throughout the structure depends partly on the part of the structure which is meshed. For the tube and core, the longest side of a triangle in the mesh is 1.0m. For the floors this parameter of the mesh generation is 2.0m. This measure was taken when the model became too large for the program to handle. Because the floors are not modeled realistically and will not be checked for stresses the mesh was made less dense in this place, instead of through the entire model. The mesh parameter is however not the only thing that influences the actual size of the mesh elements. In parts of the structure where the elements themselves are smaller than 1m in width or height, or near complicated elements like the corner connections the mesh becomes finer than the mesh parameter of 1m. This effect can be seen in Figure 59.

![Figure 59: Part of the mesh of the tube of the structure. The mesh elements themselves become smaller than 1m in small elements and near complicated model elements, such as the corner connection](image-url)
7 Results from the FEM-analysis

In this section the results from the FEM-model in AxisVM are discussed. The model is used to check the most important characteristics of a prefabricated tube-in-tube high-rise structure, such as deflection at the top of the structure, the reaction forces on the foundations and the shear stresses in the connections. In addition to these checks, also an indication will be given for where the highest stresses in the concrete are present and two governing elements will be checked. Lastly, the iterative process that was used to determine which corner connections are in tension is discussed.

For the results of the top deflection, the model will be compared with a cast-in-place model that was created. This cast-in-place model has exactly the same parameters as the prefabricated model, with respect to dimensions, elements, floors and loading. The only difference is the removal of all the edge hinges in between the elements in the tube and the core. If no edge hinges are modeled, AxisVM will assume a cast-in-place connection. The comparison between the two models will give insight in the behavior of prefabricated concrete high rise structures with the used stability system.

7.1 Top deflection

Often the governing factor in high-rise design is the deflection at the top of the structure, because high-rise structures are mostly slender and the wind loads are very high. The maximum allowable deflection of the top of the structure is given by the following expression:

$$w_{max} = 0.002 \cdot H = 0.002 \cdot 201.3m = 402.6mm$$

This value is the same for both the $x$ and the $y$-direction in either direction of those axes. Because this is a deformation calculation, the load cases that should be used are in the Serviceable Limit State (SLS). In these load cases no load factors are used. These correspond to: LC 1, LC 2, LC 3 and LC 4, which feature wind loads in the $x$, $-x$, $y$ and $-y$ direction respectively. The results of these load cases on the deflections can be found in Figure 60.

As can be seen in the figure, all deflections are within the limit set of 402.6mm. The highest deflection is found in load case LC 4 and is 327.075mm. This means that the structure does satisfy the requirements set for high-rise structures. Between the two directions in which the wind load was researched it can be seen that the deflections in the $y$-direction are slightly higher than in the $x$-direction. This difference is around 13%. This can be explained by the floor plan of the structure. In the $x$-direction, the deflection is smaller due to the slightly smaller loaded area (the structure is 29.5m wide instead of 30m) and the longer lateral resisting elements. The tube walls are 30m long instead of 29.5m and the core walls are 14.4m long instead of 12m.
The results of the prefabricated tube-in-tube structure can be compared to the results from a similar structure that is built in a cast-in-place way. In order to be able to do this comparison all edge hinges in the core and tube of the structure were removed. The results of this comparison can be found in the table below:

<table>
<thead>
<tr>
<th>Load Case</th>
<th>cast-in-place</th>
<th>Prefabricated</th>
<th>Additional deflection %</th>
</tr>
</thead>
<tbody>
<tr>
<td>LC 1</td>
<td>260.6mm</td>
<td>281.5mm</td>
<td>8.0%</td>
</tr>
<tr>
<td>LC 2</td>
<td>258.7mm</td>
<td>279.4mm</td>
<td>8.0%</td>
</tr>
<tr>
<td>LC 3</td>
<td>281.0mm</td>
<td>305.3mm</td>
<td>8.6%</td>
</tr>
<tr>
<td>LC 4</td>
<td>301.8mm</td>
<td>327.1mm</td>
<td>8.4%</td>
</tr>
</tbody>
</table>

The additional deflection is fairly uniform for all load cases and is around 8%. This increase in top deflection is one of the downsides of making a structure in prefabricated concrete. And because the top deflection is often governing in high-rise construction this could lead to extra measures that have to be taken in order to satisfy the requirements of the building codes. However, an increase of 8% is not enough to rule out prefabricated construction of tube-in-tube structures. It is just one of the disadvantages of the system that has to be balanced by other, advantageous, aspects of the system.

### 7.2 Reaction forces

The second result from the AxisVM analysis that will be looked into is the reaction force in the foundation. The highest reaction force that was found in the model was for the load case LC 5. This is a load case without any wind loads, but with a high loading factor on the dead load of the structure. This might appear to be strange, for most of the governing load cases are a combination of the high lateral loads combined with the forces in the $z$-direction. The results for this load case can be found in Figure 61.

In this image it can be seen that the highest reaction force on the foundations can be found in the core of the structure and has a value of 12153 kN/m. The reason why this maximum value can be found in the core and in this particular load case can be explained by the large part of the vertical loads that are transferred to the foundations by the core. The reason why this highest value cannot be found in any of the corners of the core, but instead is near the middle of one of the walls, is that the doorways into the core are close to those corners. These holes in the core make the walls less stiff near the corners which means that larger parts of the loads will be taken up by the middle of the wall. Whether this maximum value exceeds the limit of the connection will not be part of this thesis, for the foundation of the structure is not looked into in great detail.

### 7.3 Shear stress connections

In the section on connections the shear stiffness of the horizontal connections has been determined. Part of this calculation was the determination of the design shear resistance $v_{Rdi}$ at the interface between two elements. In this section it will be checked whether the results for the stresses at these interfaces stay within this shear
resistance limit. If it does exceed this value the connection will have to be made stronger. The results from the model that have to be used to determine this are the shear forces in the edge hinges $n_x$, which are the elements that were used to model the connections. Because these values are given in kN/m, it has to be rewritten to the same unit as $v_{Rd, N/mm^2}$. This can be done by dividing the found value by the area of the connection per meter ($A_{con}$) and then converting the kN/m to N/mm² by dividing that value by 1000. This gives: $\tau_{con} = \frac{n_x}{A_{con}} \cdot 0.001$.

Because there are two connection stiffnesses, one for the connections at the lintels and one for the connections at the columns and they both have a different design shear resistance $v_{Rdi}$, just looking at the highest value from a particular load case is not enough to determine whether the design shear resistance is met. Therefore, the shear stresses in the edge hinges are displayed from the top view in the structure. From this view, the two different zones can be distinguished and the maximum values for both can be found. Because the values for $n_x$ were very high in the columns near the lobby, it was decided that the bottom 2 levels of the tube are going to be cast in-situ rather than made of prefabricated elements.

The governing load case for the column and lintel maximum shear stiffness are not in the same load case. For the columns, the governing load case is load case LC 12. The location of this maximum value in the plan can be seen in Figure 62. The height of the governing location is between the 3rd and 4th floor, the lowest horizontal prefabricated connection.

The governing load case for the lintels can be found in load case LC 6. It is important to note that this is not the highest value for this load case, but the highest value of $n_x$ located near a lintel. The results can be found in Figure 63. Several things can be seen in the two figures. First off all, the highest shear stresses occur in the web walls of the structure, which are the top and bottom wall for LC6 and left and right wall for LC12. Secondly, the sign of the shear force is not important and is only dependent on the local axes in the edge hinges. An interesting pattern arises from the two different shear stiffnesses in the same horizontal connection. If the shear connection stiffness would be the same for the entire connection, the highest shear stresses would occur where the windows are, because the absence of material there leads to the highest shear deformation. However, in this case, the stiffness is significantly higher in the opposite parts, where the columns are. This leads to the shear stress pattern in the web walls as can be seen in Figure 62 and Figure 63. The highest shear stress occurs in the columns with the higher shear stiffness, but closest to the windows where the shear deformation is highest. The discrete change of stiffness leads to a jump in the stress diagram of the shear stress in the connections. This clearly is the result of the assumption made for the connection stiffnesses and does not reflect the real stress distribution. To prove this, Figure 64 was created. Here, three different shear stiffness distributions of the connections are given for a perforated shear wall with the resulting shear stress distribution in the connections. The uniform distribution shows that the shear is in the parts near the windows. The two stepped distribution shows a similar pattern to the one found in the main model and the third image shows a distribution close to the real distribution, which is the most advantageous distribution. From this we can conclude that if the used
The results of the AxisVM model for load case LC 6 for the $n_x$ in the horizontal connections and the location of the governing connection near the lintels.

two-step stiffness distribution satisfies the set requirements, the real distribution will as well.

The highest shear force found in Figure 62 is: $n_{x,max,\text{strong}} = 1174 \, kN/m$. Now, the shear stress can be determined by:

$$\tau_{\text{con,\text{strong}}} = \frac{n_x}{A_{\text{con}}} \cdot 0.001 = \frac{1174}{0.4} \cdot 0.001 = 2.94 \, N/mm^2$$

This value is lower than the $v_{Rdi,\text{strong}}$:

$$2.94 \, N/mm^2 \leq v_{Rdi,\text{strong}} = 4.28 \, N/mm^2$$

This can also be done for the connections with the lower connection stiffness:

$$\tau_{\text{con,\text{weak}}} = \frac{n_{x,max,\text{strong}}}{A_{\text{con}}} \cdot 0.001 = \frac{471}{0.4} \cdot 0.001 = 1.18 \, N/mm^2 \leq 1.78 \, N/mm^2 = v_{Rdi,\text{weak}}$$

This means that for the entire structure and for all load cases the shear strength of the connections is not reached and the connections are strong enough.
7.4 Element stresses

Another aspect that has to be checked are the normal stresses in the concrete. There are many different regions in the structure in which this has to be checked for both compression and tensile stresses. Checking all of these elements would be impossible to do within the scope of this thesis. However, an indication of the highest stresses and their locations can be found. With this information a closer look into important or governing elements can be done.

For an indication of the governing locations for stress in the elements, the maximum allowable stresses in the concrete have to be determined. For the design compression stress, the characteristic cylinder strength has to be divided by the material factor. This gives for the main two used concrete classes:

\[
\begin{align*}
f_{cd,C50/60} &= \frac{f_{ck,C50/60}}{\gamma} = \frac{50}{1.5} = 33.33 \frac{N}{mm^2} \\
f_{cd,C80/95} &= \frac{f_{ck,C80/95}}{\gamma} = \frac{50}{1.5} = 53.33 \frac{N}{mm^2}
\end{align*}
\]

However, a simple check whether this limit is reached in a specific direction for certain load cases is not enough to determine whether the concrete is strong enough. The reason behind this is that an element has to resist not just normal stresses, but also shear and moment stresses and a combination of all those factors at the same time. The results for these kind of stresses can however be indicative to find governing elements. For tension stresses it is impossible to determine such a stress limit which may not be reached, for there is always the possibility of using reinforcement to cope with tension stresses. The check for tension stresses will be done by estimating the amount of reinforcement needed and seeing whether the reinforcement percentage would not be too high.

**Compression stresses**

For the compression stresses in the model one could imagine where the highest stresses might appear. Near the bottom of the structure the weight of the structure above it becomes higher. In addition to this compression stress, on the leeward side of the structure there are heightened stresses due to the bending of the tube, leading to a higher compression there and a lower compression force on the windward side of the structure.

Therefore, load case LC 10 was picked to find the governing elements in the structure with respect to compression stress. This load case is described by ULS in the y-direction and unfavorable dead load. This load case was found to be exemplary for the results in other, similar load cases. The results will be shown in the AxisVM parameter of \( n_{yD} \), which is a value that is used to determine the reinforcement in the local y-direction (which is the global z-direction for all core and tube elements). The first location which will be discussed is the core of the structure. The results can be seen in Figure 65.

As was expected, the highest values for \( n_{yD} \) can be found on the leeward side of the structure near the...
foundations. Ignoring some very local peak stresses, the values of the elements here are near \( n_yD = 18000 \text{kN/m} \).

To get an indication of the stress in \([\frac{N}{mm^2}]\) the value found has to undergo the following transformation:

\[
\sigma_n = \frac{n_yD}{t_{wall}} = \frac{18000}{0.4} = 45 \frac{N}{mm^2}
\]

Fortunately, this value is below the \( f_{cd,C80/95} = 53.33 \frac{N}{mm^2} \). If it were over this value, the chances of being able to reinforce this element properly would be small.

To check whether the value for \( n_yD \) reaches higher values in the tube of the structure another analysis was performed. The results of which can be found in Figure 66.

In this figure, the entire leeward side of the structure is shown on the left side. From this overview it can again be seen that the highest design normal stresses in the local \( y \)-direction can be found near the bottom of the structure. Therefore a closer look at the first few floors can be seen on the right of the image. The highest values for \( n_yD \) are present in the lobby of the structure. Because these elements are thicker than most elements in the tube and the design of the lobby is less important for this thesis, these values will be ignored. The governing element with a 400\( mm \) thickness is the column right above the lobby, which is indicated by the black box in the figure. In this column it can be seen that not only the normal stresses from the weight of the structure and the bending of the entire tube are parts of the \( n_yD \), but also the shearing of the frames. Especially the shearing of the frames near the balconies, where the tube is weaker than in other areas of the tube with regular windows, seems to have a real effect on the value for \( n_yD \). In Figure 67 a simplified frame is shown which is loaded from the side to see the effect of such a force on the normal stresses in a frame loaded in shear. The same effect does also explain the peak areas of stress in the larger model. All in all, the column indicated by the black box in Figure 66 seems to be a good element to look further into in an upcoming section.

**Column calculation**

The column that will get an estimated reinforcement design is 3.05\( m \) long, 0.9\( m \) wide and 0.4\( m \) thick. The concrete class of the column is C80/95. On the top and bottom of the column, lintels are present. These transfer forces into the column and might give some support in the plane of the tube wall. However, this support is not used in the following calculations for this will be a simple calculation that is made for an estimation only. The forces in the column will be determined using the following section equations:

\[
N = \int \sigma_{yy}dA
\]

\[
M_y = \int y\sigma_{yy}dA
\]

\[
V = \frac{dM_y}{dy}
\]
Using 5 sections and 8 measuring points per section, the following values were found for the column: In figure 68

\[
\begin{align*}
n_{Ed} &= \frac{N_{Ed}}{b \cdot h \cdot f_{cd}} = \frac{20280}{400 \cdot 900 \cdot 53.33} = 1.0563 \\
m_{Ed} &= \frac{M_d}{b \cdot h^2 \cdot f_{cd}} = \frac{1300 \cdot 10^6}{400 \cdot 900^2 \cdot 53.33} = 0.0752
\end{align*}
\]

Using the interaction diagram in Figure 69. It can be found that the following holds: \( \Psi = 0.2 \). From the diagram it can also be found what parts of the reinforcement will be yielding. In this case, only the reinforcement in the most compressed side of the column will be yielding.

Now the reinforcement percentage can be determined via: \( \rho = \frac{\Psi f_{cd}}{f_{yd}} = \frac{0.2 \cdot 53.33}{435} = 0.02452 = 2.452\% \). This will mean an area of the reinforcement of: \( 0.02452 \cdot 900 \times 400 = 8827 \text{mm}^2 \). Using the fairly large 032mm reinforcement bars this can be satisfied with 11 bars: \( 11 \times 0.25 \times \pi \cdot 32^2 = 8846 \text{mm}^2 \). This reinforcement will be necessary in both sides of the column, because the wind can come from an opposite direction in another load case. The total reinforcement rate will then be: \( \rho_{tot} = \frac{4}{11} = \frac{17694}{400 \times 900} = 0.0491 = 4.91\% \). The recommended value in the Eurocode is \( \rho_{tot,max} = 4\% \) so this is over the recommended limit. However the Eurocode states that higher percentages may be used if it can be shown that the integrity of the concrete is not affected. Furthermore, the Dutch national annex states that reinforcement percentages up to \( \rho_{tot,max} = 8\% \) are allowed.
7 RESULTS FROM THE FEM-ANALYSIS

Figure 69: The interaction diagram in which the value for $n_{Ed}$ and $m_{Ed}$ lead to a value of $\Psi$, courtesy of [38]

in column were no lapping of the reinforcement is present. It can be concluded that the found reinforcement percentage provides no problems for the feasibility of the structure.

**Tension stresses**

Although tension stresses that occur in the structure can be solved by adding additional reinforcement, it should be checked whether the reinforcement percentages needed throughout the structure can be deemed realistic. This will be checked in this subsection. The parameter which is used to check the need for reinforcement is the AxisVM output parameter $n_{xD}$, which is the design reinforcement for axial forces in the local $x$-direction. The local $x$-direction of the lintels, which are the governing elements in this check, is horizontally and the parallel to the direction of the wall the element is in. In Figure 70 the results for this parameter in load case 12 are shown. This load case causes some of the largest problems with tension in the tube.

The two details on the right of Figure 70 show best where the highest values for $n_{xD}$ occur. Near the lobby these values are very high. The discontinuation of the columns in the front wall of the lobby gives tension and compression stresses that are very high compared to the rest of the structure. Just as with the high compression stresses in the same area, these elements will not be looked into or a reinforcement design made. The more recurring elements are more interesting and tell more about the workings of the prefabricated tube-in-tube structures. Therefore, in the other detail, the floors 16 to 22 are shown. This detail is made of one of the walls that functions as a web-wall with the wind direction of load case 12. The shearing associated with the web wall has its influence on the $n_{xD}$, just as it had its effects on the maximum compression stress. There are peak values of $n_{xD}$ in the lintels near the corners of the frames. These values are significantly higher than the peak values of $n_{xD}$ in the regular window lintels.

**Lintel calculation**

To determine the reinforcement percentage that would be needed in these lintels a smaller detail is shown in Figure 71. This is the frame of the small balcony on the 20th floor of the structure. This lintel is 3.0m long, 0.6m high and has the same thickness as the rest of the tube: 0.4m. The concrete class on this level is C50/60. Using some assumptions some reinforcement parameters will be found. Multiple sources [37] were used and combined to produce these calculations. If these found values are reasonable for reinforcement in such an element in such a structure, the lintels can be deemed structurally feasible.

Using only the highest value in the exact corner of the frame would not give a good estimate of the actual
Figure 70: The results for the design reinforcement for axial forces in the local \( x \)-direction in the tube for the AxisVM model of load case LC 12. On the left an overview of the entire structure, on the right two details of important areas for the analysis.

Figure 71: The results for the design reinforcement for axial forces in the local \( x \)-direction the frame of the small balcony of the 20th floor

reinforcement needed. To design the lintel properly with normal forces, shear forces and bending in mind the lintel should all be used. To determine the N-, M- and V-line of the lintel the following equations were used:

\[
N = \int \sigma_{xx} dA
\]

\[
M_x = \int x \sigma_{xx} dA
\]
7 RESULTS FROM THE FEM-ANALYSIS

\[ V = \frac{dM_x}{dx} \]

In figure 72 the lintel and the N-, M- and V-line are shown. The results for these lines are estimated by using

5 sections and 7 measuring points on those. This gives a close enough estimate to determine the reinforcement needed in the lintel.

Firstly, the N-line will be discussed. Large parts of the lintel are in tension. The highest found value is \( N_{\text{max}} = 201 \text{kN} \). If this force is divided by the area of the lintel a stress due to this highest normal force can be found:

\[
\sigma_{\text{normal}} = \frac{201 \cdot 10^3}{400 \cdot 600} = +0.83 \text{N/mm}^2.
\]

This value is than the characteristic tensile strength of C50/60 concrete \( f_{c,k,C50/60} = 2.9 \text{N/mm}^2 \) and no reinforcement has to be designed for solely the purposes of the normal stress.

When the M-line is reviewed, it can be seen that there is a positive moment on the left side of the lintel, an almost equal sized negative moment on the right side of the lintel and an absence of high moments in the middle of the lintel. The governing moment is on the left side of the lintel and equals: \( M_{x,\text{max}} = 519 \text{kNm} \). To determine the reinforcement needed for this moment, first the effective depth of the cross-section will have to be determined. As an estimate, longitudinal bars of Ø25 mm will be used and stirrups of Ø16 mm. Combined with a minimum cover \( c = 25 \text{mm} \) the effective height can be determined via

\[
d = h - \left( \frac{3}{2} \cdot \phi + \phi_{\text{stirrup}} \right) + c = 600 - \left( \frac{3}{2} \cdot 25 + 16 + 25 \right) = 521.5 \text{mm}
\]

With the following value and a design table the reinforcement can be determined:

\[
\frac{M_{\text{ed}}}{f_{\text{cd}} \cdot b \cdot d^2} = \frac{519646}{33.33 \cdot 400 \cdot 521.5^2} = 143.31
\]

Which leads to an interpolated value in the design table of \( \rho = 1.1954\% \). The total area of the reinforcement in the (in this case) top of the beam can be calculated with: \( A_{s,\text{top}} = \rho \cdot b \cdot d = 0.01954 \cdot 400 \cdot 521.5 = 2493.6 \text{mm}^2 \). Using Ø25 mm bars this can be achieved with: \( 6 \times 0.25 \cdot \pi \cdot 25^2 = 2945.2 \text{mm}^2 \). Giving the beam a reinforcement percentage of \( \rho_{\text{top}} = 1.41\% \). Because in other load cases the reversal of the wind direction will switch the positive and negative moment, the same reinforcement will be placed in the bottom of this section. The total reinforcement percentage will then be: \( \rho_{\text{tot}} = \rho_{\text{top}} + \rho_{\text{bot}} = 1.41 + 1.41 = 2.82\% \).
The next step is to determine the shear reinforcement, if needed, in the lintel. In 72 it can be seen that the shear force is fairly uniform over most of the lintel. The section which will be looked into is the same as in the determination of the moment reinforcement, on the left side of the lintel, with \( V = 372kN \).

To determine whether shear reinforcement is actually needed an equation can be found in the Eurocode [4], which reads:

\[
V_{Rd,c} = [C_{Rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{2/3} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d
\]

in which \( V_{Rd,c} \) is the shear resistance \([N]\), \( C_{Rd,c} = 0.18 \), \( k = 1 + \sqrt{\frac{200}{d}} \), \( \rho_1 = \frac{A_{sl}}{A_{c}} \), \( A_{sl} \) the area of the tensile reinforcement \([mm^2]\), \( b_w \) the smallest width of the lintel \([mm]\), \( \sigma_{cp} = \frac{N_{Ed}}{A_{c}} \cdot \frac{N}{mm^2} \), with \( N_{Ed} \) the normal force in the section \([kN]\) and \( A_{c} \) the area of the concrete cross section \(mm^2\).

\[
\begin{align*}
C_{Rd,c} & = 0.18 \\
f_{ck} & = 50 \frac{N}{mm^2} \\
k & = 1 + \sqrt{\frac{200}{521.5}} = 1.69 \\
\rho_1 & = \frac{400 \times 521.5}{5 \times 2.04 \times 521.5} = 0.0114 \\
\sigma_{cp} & = \frac{184 \times 10^3}{400 \times 600} = 0.76 \frac{N}{mm^2} \\
k_1 & = 0.15
\end{align*}
\]

With these values the shear resistance without shear reinforcement can be found:

\[
V_{Rd,c} = [C_{Rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{2/3} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d = [0.12 \cdot 1.619 \cdot (100 \cdot 0.0114 \cdot 521.5)^{2/3} + 0.76 \cdot 400 \cdot 521.5] = 179793N
\]

Because this value is lower than the shear force in the section \((180kN < 374kN)\) shear reinforcement will be needed.

To determine the shear reinforcement, the following equation will be used:

\[
\frac{A_{sw}}{s} = \frac{V_{Ed}}{0.9 \cdot d \cdot f_{yd}}
\]

in which \( A_{sw} \) is the area of the shear reinforcement \([mm^2]\), \( s \) the distance between the shear stirrups and \( V_{Ed} \) the present shear force. If \( s \) is set to 1000\( mm \), the area needed per \( 1m \) lintel can be determined by:

\[
\frac{0.9 \cdot 521.5 \cdot 435}{0.12 \cdot 1.619 \cdot (100 \cdot 0.0114 \cdot 521.5)^{2/3} + 0.76 \cdot 400 \cdot 521.5} = 1831mm^2/m.
\]

Using stirrups of \( Ø16mm \) this can be achieved with a center to center distance of the stirrups of \( 200mm \): \( 5 \times 2 \times 0.25 \cdot \pi \cdot 16^2 = 2010.5mm^2 /m \). The factor 5 is for the amount of stirrups per \( m \) \((1000 \cdot 200)\) and the factor 2 is because of the two passes of one stirrup, on both sides of the beam.

The next step is to check whether the compression stress in the compression diagonals between the reinforcement is reached. The equation for this stress is:

\[
V_{Rd,2} = 0.5 \cdot v \cdot f_{cd} \cdot b_w \cdot 0.9 \cdot d
\]

assuming the angle \( \alpha \) of the diagonal is \( 45^\circ \) and using \( v = 0.7 - (\frac{f_{cd}}{200}) = 0.7 - (\frac{200}{200}) = 0.45 \). The equation now reads:

\[
V_{Rd,2} = 0.5 \cdot v \cdot f_{cd} \cdot b_w \cdot 0.9 \cdot d = 0.5 \cdot 0.45 \cdot 33.33 \cdot 400 \cdot 0.9 \cdot 521.5 = 1408kN
\]

This is the resistance of the concrete compression diagonals. The actual stress can be determined with the shear force and the angle of the diagonal: \( V_{D,max} = \frac{V_{Ed}}{\sin(\alpha)} = \frac{374kN}{\sin(45)} = 528.9kN < V_{Rd,2} = 1408kN \). This means the stress in the compression diagonal is not reached and the reinforcement as is will be sufficient.

### 7.5 Tension stress in corner connections

In the section on connections the normal stiffness for corner connections has been determined for connections that are in compression and that are in tension. To figure out which elements are in compression and which are in tension an iterative process has to be followed. At the beginning of this process all corner connections are given a stiffness associated with compression in the connection. After an analysis is completed all connections that experience a tension force in the modeled springs in any of the load cases are given the lower normal stiffness associated with a tensile force in the connection. Hereafter another analysis is done until no connections have to be changed after an analysis step.

In practice the connections that experienced tension forces are close to the top of the structure. In the upper
few levels of both the tube and the core of the structure the vertical loads due to the self weight and live load on
the floors is relatively low. In combination with wind loads this can than lead to a tensile force in the springs.
Especially the load cases with the favorable dead load (LC 7, LC 9, LC 11, LC 13) were prone to include these
forces. After a few iterations however, a result was found. The top 5 levels of the structure are given a lower
stiffness in the corners due to tensile stresses. Figure 73 shows the results of the last analysis step in load case
LC 9.

Figure 73: The results of the last analysis step for checking in which corner connections a tensile force is present,
for LC 9

7.6 Possible removal of the balconies

In some of the previous sections, the high stress concentrations were focused near the frames at the balconies.
Clearly these are the least stiff points on each floor. In an earlier stage models were made to see how large the
negative influence is of the frames at the balconies on the overall deflection of the structure. These models were
made in a stage before some of the important changes of the model had been done, such as the implementation
of the lobby and the lowering of the corner connections in stiffness. However, these models can still give an
indication about the influence of the balconies on the overall deflection of the structure.

In order to study this behavior, three models were made. One model was the standard model with 400\text{mm}
thick walls and balconies. In the other two models the balconies were removed and more regular windows
placed where the balconies had been. These two models had different wall thicknesses, 400\text{mm} and 300\text{mm}
respectively. The results of the deflection can be found in the following table:

<table>
<thead>
<tr>
<th>Model</th>
<th>400\text{mm} with balconies</th>
<th>400\text{mm} without balconies</th>
<th>300\text{mm} without balconies</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection in the $x$-direction</td>
<td>255.6\text{mm}</td>
<td>190.0\text{mm}</td>
<td>251.8\text{mm}</td>
</tr>
<tr>
<td>Deflection in the $y$-direction</td>
<td>288.7\text{mm}</td>
<td>263.8\text{mm}</td>
<td>269.894\text{mm}</td>
</tr>
</tbody>
</table>

From this table, it can be clearly seen that there is a negative effect of the frames around the balconies on the
total deflection of the structure. In the $x$-direction the deflection is increased by 34.5\% if balconies are present.
The difference in the $y$-direction is even larger, at 41.7\%. Even when the thickness of the elements is reduced
by 25\% from 400\text{mm} to 300\text{mm} for the model without balconies, it outperforms the one with balconies. The
overall deflection is higher for the model with balconies by 1.5\% in the $x$-direction and 7\% in the $y$-direction.
In practice, this information would be discussed with all important parties in the design phase of structure. A
significant reduction of the element thickness would mean a significant decrease in price of the structure. Most
project developers would be interested in such a saving. However, it may also be very important to a developer
to keep the balconies for it generates value for the users of the structure. Also the architect might not want to
do any concessions to his or her design. This discussion can have different outcomes. For this thesis however,
in which making a comparable model to the one of ten Hagen is most important, just noting this effect of the
frames around the balconies on the overall deflection is enough and should be kept in mind while writing the
conclusions.
8 Additional Calculations

In this section some additional calculations are presented which are not the results from the main AxisVM model.

8.1 Top acceleration

In tall structures not only the total deflection under static loading is important, but also the dynamic behavior of a structure. The accelerations associated with this dynamic behavior can cause nausea and a sense of insecurity for the users of a building. Therefore it is important to determine that the maximum acceleration in the top of the structure stays below a threshold that is determined in the Hoogbouwconvenant [5], a Dutch annex to the Eurocode with regard to high-rise structures. To determine this a rather intricate calculation is necessary. This section is based on the Eurocode 1: Actions on structures – Part 1-4: General Actions, Wind Actions [3], the Dutch national annex to this Eurocode, the Hoogbouwconvenant and chapter 6 of a reader from the TU Eindhoven [15].

The peak accelerations of a structure are determined by the following equation:

\[ a = k_p c_f p I_v(z) v^2_m(z) R K_e K_r \mu_e \]

in which \( a \) is the acceleration at the specified height \([m/s^2]\), \( k_p \) the peak factor or ratio between the maximum value and the standard deviation, \( c_f \) the earlier determined force coefficient of the building, \( \rho \) the density of air, \( I_v(z) \) the turbulence intensity, \( v_m \) the average wind speed with a one year return period, \( R \) the resonance response factor, \( K_e \) and \( K_r \) factors that indicate the loading shape coefficients and \( \mu_e \) the mass of the building per unit area. A lot of these factors have their own equations that have to be calculated before this equation can be used.

Before these values are determined a reference height \( z \) has to be chosen for the height where the answer of the equation relates to. Here, \( z = H = 201.3m \) is used. For \( k_p \) the following equation is given: \( k_p = \sqrt{2 \ln(vT)} + \frac{0.6}{\sqrt{2 \ln(vT)}} \geq 3. \) The \( v \) is the frequency of the wind gust, here taken as the frequency of the building (\( n_1 \)). The frequency of a concrete structure can be estimated as: \( n_1 = \frac{42}{H} = \frac{42}{201.3} = 0.2086Hz. \) \( T \) is the duration of a 10-minute reference storm in seconds: \( T = 600s. \) This leads to:

\[ k_p = \sqrt{2 \ln(vT)} + \frac{0.6}{\sqrt{2 \ln(vT)}} = \sqrt{2 \ln(0.2086 \cdot 600)} + \frac{0.6}{\sqrt{2 \ln(0.2086 \cdot 600)}} = 3.3 \]

The force coefficient \( c_f \) has been determined in the section on wind loading and reads:

\[ c_f = 1.47 \]

\( \rho \) is the density of the air around the building and can be taken as:

\[ \rho = 1.25 \frac{kg}{m^3} \]

To determine the turbulence factor at \( z = 201.3 \) the following equation is used: \( I_v(z) = \frac{k_1}{c_0(z) \ln \left( \frac{z}{z_0} \right)} \). The recommended turbulence factor \( k_1 = 1.0 \), according to the Eurocode. The factor \( c_0 \) depends on the height differences in the terrain, or orography, and is also \( c_0 = 1.0 \) for a structure in Rotterdam. \( z_0 \) is the roughness length and is dependent on the structures and terrain around a building. For an urban environment such as Rotterdam (category IV) this length is \( z_0 = 1.0m \). Because all these factors are equal to 1.0, the equation for \( I_v(z) \) is simplified and will read:

\[ I_v(201.3) = \frac{1}{\ln(201.3)} = 0.1885 \]

For the \( v_m(z) \) the following equation is used: \( v_m(z) = v_b k_r \ln \left( \frac{z}{z_0} \right) \). The basic wind speed \( v_b \) is determined by the following equation: \( v_b = v_b 0.05 \cdot \text{dir}_c \cdot \text{season}_c \cdot \text{prob}_c \). In early design stages however, the last three terms can be set to a unit value and only \( v_b 0.05 \) has to be used. For wind area II in the Netherlands this value is \( v_b 0.05 = 27m/s = v_b \). \( k_r \) is a terrain factor and uses the roughness length earlier used in the determination of \( I_v \). \( k_r = 0.19 \left( \frac{z_0}{0.05} \right)^{0.07} = 0.19(10)^{0.07} = 0.2343 \). Now the equation for \( v_m \) can be filled in:

\[ v_m(201.3) = v_b k_r \ln \left( \frac{z}{z_0} \right) = 27 \cdot 0.2343 \cdot \ln \left( \frac{201.3}{1.0} \right) = 33.6m/s \]

For the resonance response factor \( R \), first the \( \mu_e \) has to be determined. This value indicates the mass of the building per unit value and is determined by \( \rho d \) in which \( d = 30m \) which is the width of the building and \( \rho d \) is the density of the building.
is the volumetric mass of the building in kg/m$^3$. This value can be determined by calculating the mass of the concrete per floor and dividing that by the volume of one floor. $\rho_{\text{concrete}} = 2500\text{kg/m}^3$ will be used.

If a massive floor of 300mm is assumed, the weight of one floor is: $W_{\text{floor}} = 30\times 29.5\text{m} \times 0.3\text{m} \times 2500\text{kg/m}^3 = 663750\text{kg}$. For the walls a similar calculation can be used, but for the tube walls it has to be taken into account that the walls are perforated with windows. The windows make up around 30% of the area of the elements, so a factor of 0.7 will be added to the calculation of the weight of the wall tubes. The doorways in the core are considered to be negligible. This leads to: $W_{\text{walls, tube}} = (2 \times 30m + 2 \times 29.5m) \times 0.4m \times 3.05m \times 2500\text{kg/m}^3 = 254065\text{kg}$ and $W_{\text{walls, core}} = (2 \times 14.4m + 2 \times 12m) \times 0.4m \times 3.05m \times 2500\text{kg/m}^3 = 161040\text{kg}$ Now $p_0$ is:

$$p_0 = \frac{W_{\text{floor}} + W_{\text{walls, tube}} + W_{\text{walls, core}}}{V_{\text{floor}}} = \frac{663750 + 254065 + 161040}{60m \times 29.5m \times 3.05m} = 399.7\text{kg/m}^3$$

Which with a width of 30m leads to:

$$\mu_c = p_0d = 399.7 \times 30 = 11990\text{kg/m}^2$$

Now for the resonance response factor $R$, the equation is: $R^2 = \frac{\pi^2}{20} S_L(z,n_1) K_s(n_1)$. The damping $\delta$ is made up of two parts: $\delta = \delta_s + \delta_a$. $\delta_s = 0.1$ for concrete structures. The equation for the other part, the aerodynamic damping is: $\delta_a = \frac{c_p \rho_b v^2(z)}{2 \mu_c} = \frac{1.47 \times 1.25 \times 3.36}{2 \times 0.2086 \times 11990} = 0.0123$. Which leads to: $\delta = \delta_s + \delta_a = 0.1 + 0.0123 = 0.1123$.

The next part of the equation is the non-dimensional spectral density function $S_L(z,n_1)$, which requires a non-dimensional frequency $f_L(z,n_1)$, which in turn needs the average size of the wind gust $L(z) = L_t \left(\frac{z}{z_0}\right)^a$ in which $L_t = 300m$ is the reference length, $x_t = 200m$ is a reference height and $a = 0.67$ for terrain type IV. $L(201.3) = 300 \left(\frac{200}{201.3}\right)^{0.67} = 301.3$

This leads to: $f_L = \frac{n_1 L(z)}{v_m(z)} = \frac{0.2086 \times 301.3}{33.6} = 1.8706$. Which then can be used in this equation: $S_L = \frac{6.8 f_L}{(1 + 10.2 f_L)^2} = \frac{6.8 \cdot 1.8706}{(1 + 10.2 \cdot 1.8706)^2} = 0.0857$.

Now the size reduction factor $K_s(n_1) = \frac{1}{1+\sqrt{(G_x \phi_x)^2 + (G_z \phi_z)^2 + (G_G \phi_G)^2}}$ has to be determined. In which $\phi_x = \frac{11.5 M_{n_1}}{v_m(z)} = 14.37$ and $\phi_z = \frac{11.5 M_{n_1}}{v_m(z)} = 2.14$. $G_x = \frac{16}{25}$ and $G_z = \frac{5}{4}$ are mode shape coefficients and have these values for the loading over the height has a parabolic shape and over the width is uniform. This leads to:

$$K_s = \frac{1}{1+\sqrt{(\frac{16}{25} \times 14.37)^2 + (\frac{5}{4} \times 2.14)^2 + \left(\frac{5}{4} \times \frac{11.5}{25} \times 14.37 \times \frac{5}{4} \times 2.14\right)^2}} = 0.1682$$

After all this work, the resonance response factor can be determined:

$$R^2 = \frac{\pi^2}{20} S_L(z,n_1) K_s(n_1) = \frac{\pi^2}{20} \cdot 0.0857 \cdot 0.1682 = 0.633$$

$$R = \sqrt{R^2} = \sqrt{0.624} = 0.796$$

The last two factors needed, $K_x$ and $K_z$ are also linked to the shape of the loading and become:

$$K_x = \frac{5}{3}, K_z = 1.0$$

Now all terms are known, the peak accelerations at the top of the structure can be determined using the earlier shown equation:

$$a = k_p c_f p I_v(z) v_m(z)^2 R K_s K_z \mu_c = 3.3 \times 1.47 \times 1.25 \times 0.1885 \times 33.6^2 \times 0.796 \times \frac{5}{3} \times 0.11990 = 0.1428 \text{m/s}^2$$

To check whether this is within the set limits, the Hoogbouwconvenant is used. In there is a graph which shows the maximum allowable acceleration accompanying certain frequencies. For higher frequencies higher accelerations are allowable than for lower frequencies. In Figure 74 this graph is shown and the values for $a$ and $n_1$ used to determine whether the results from the calculation meet the set requirements. It can be seen that the value calculated stays just below the set limit for line 2. Line 2 is the limit for residential structures. For office structures line 1 can be used and then the acceleration is well within the limits set for the peak acceleration at the top of the structure.

### 8.2 Floors

Although there where floors modeled in the main model in AxisVM, these floors did not represent the reality very well. The floors in that model were only added to function as elements that transferred forces in plane. To check whether the floors are capable of withstanding the forces out of plane, a separate model of one floor has been made. In this model, only the out of plane loads are added.

As has been described in an earlier section, the floors used in the structure are two way spanning floors which contain weight saving elements. No additional columns are used in the entire floor plan and the floors are all connected as a hinge to tube and core elements. In the section on loading, it was assumed that the floors weighted $0.3m \times 2400\text{kg/m}^3 = 720\text{kg/m}^2$. In this section a solution that factors in these boundary conditions
Precast Concrete in Framed Tube High-Rise Structures

Figure 74: Figure 6 of the Hoogbouwconvenant, presenting a graph for the maximum allowable acceleration in structure. Line 1 gives the limit for residential structures, line 2 the limit for office structures. The blue lines indicate the structure in this thesis.

will be found. AxisVM, the FEM-program used throughout this thesis, has an option to calculate the weight saving floor system called Cobiax, which consists of either round or flattened weight saving elements. Here, the choice was made for the round weight saving elements.

The floor that was chosen is 350\(\text{mm}\) thick and has round weight saving elements with a diameter of 225\(\text{mm}\), that are placed every 250\(\text{mm}\) in both directions. The concrete class chosen (C25/30) is fairly low, because these floors are (partly) cast in situ. In Figure 75, a part of the floor and its dimensions are shown. The weight saving elements are also visible in blue. The program automatically removes those elements near the supports, where the shear forces are highest and the weight saving elements would cause problems. The weight of the floor per \(\text{m}^2\) is: 0.35\(\text{m} \cdot 2500\text{kg/m}^3 = 875 - 238.5 = 636.5\text{kg/m}^2 \leq 720\text{kg/m}^2\). There are two loads in the floor model.

First of all there is the self weight, for which there is an option in the program itself. The second load is the variable floor load that was also used in the main model. The value of the variable load is 3.0\(\text{kN/m}^2\). This load was placed as a distributed surface load on all the floor domains. There are two load cases needed for the analysis of the floors:

- Serviceability Limit State (SLS): 1.0\(\times\)Selfweight + 1.0\(\times\)Variable floor load
- Ultimate Limit State (ULS): 1.3\(\times\)Selfweight + 1.65\(\times\)Variable floor load

For the floor domains in the model shell elements were used. These elements are capable of dealing with forces both in and out of plane. For the supports, line supports were used that go around the entire perimeter of the floors. The support stiffness used is \(R_x = R_y = R_z = 1.0 \cdot 10^7\). No rotational stiffness was given to these line supports. Although no deformation limits for floors are in the Eurocode, the old Dutch code NEN6702 did have some recommendations for it. If the most strict value is taken, the maximum allowable deflection of a floor is: \(w_{\text{max}} = 0.002 \cdot l\). In Figure 76 the deflections in the \(z\)-direction under influence of the SLS loads can be seen. The highest deflections can be seen in the part of the floor where the span is 8.75\(m\). The deflection is 6.312\(\text{mm} \leq 0.002 \cdot 8750 = 17.5\text{mm}\). In the part of the floor where the span is 7.8\(m\), the maximum deflection is 3.906\(\text{mm} \leq 0.002 \cdot 7800 = 15.6\text{mm}\). So clearly, the deflections in the floor at not a problem.

To check whether the strength of the floors is sufficient, the reinforcement needed in the floors was looked into. If the reinforcement needed is not a very high percentage, the floor can be considered viable. Precise detailing and calculating crack widths is too detailed for this thesis. In Figure 77 the results from the AxisVM calculation can be found. For the bottom of the slab the reinforcement needs for both the \(x\)-direction and \(y\)-direction are shown, for the top layer only in one direction. It can be seen that for the bottom layer the maximum needed deflection per \(m\) is 799\(\text{mm}^2/m\) for the \(x\)-direction and 942\(\text{mm}^2/m\) for the \(y\)-direction. Both these needs can be fulfilled by for instance 1 bar \(\text{Ø}18\text{mm}\) every 250\(\text{mm}\). This works well because every 250\(\text{mm}\) a weight saving element is used. This would lead to: \(4 \times \pi \cdot r^2 = 4 \times \pi \cdot 9^2 = 1018\text{mm}^2/m\). In a slab of 350\(\text{mm}\) thick this adds:
Figure 75: An image of part of the floors in AxisVM. In blue the weight saving elements can be seen which are discontinued near the supports.

Figure 76: The deflections of the floor in the z-direction in the serviceability limit state (SLS).
In combination with the corners this leads to the total lettable area: 

\[ A = \text{length of the areas between these corners} = 30 - 5 \cdot 25 = 161 \text{m}^2 \]

This gives: 7.8% to the reinforcement ratio. As can be seen from the results for the top layer, the need for reinforcement is concentrated in the corners of the floors. The highest value is a multiple of the results from the bottom layer, reaching requirements of around 4000\( \text{mm}^2/\text{m} \). If these are added, the reinforcement ratio becomes: 

\[ \frac{1038}{350 \times 1000} = 1.18 + 0.29 = 1.47\% \]

This is still a value which is reasonable in a floor like this one, especially if this is only very locally needed. Therefore, this floor can be considered viable.

Figure 77: The reinforcement needs for the bottom (in the \( x \)-direction (left) and in the \( y \)-direction (middle)) and the top layer (right) of the slab in the ultimate limit state (ULS)

### 8.3 Net Lettable Area

In this section, the area that can be rented out to companies will be calculated and compared to the original design of the Zalmhaven tower. This will be done by determining net Net Lettable Area for one floor in the structure. The net lettable area is given by the following equation:

\[
NLA = (1 - \frac{A_{lettable}}{A_{total}}) \times 100\%
\]

Some assumptions have been made to serve as a starting point in the calculation:

- Although the original building was a residential structure, both structures will be considered as an office building
- Only the 7.2\( \text{m} \) closest to the window can be let as an office
- All corridors are 1.5\( \text{m} \) wide and do not count as lettable area
- The outer façade of the original structure is 0.25\( \text{m} \) thick
- Both structures have the same dimensions with respect to the lines through the center of the walls

With these assumptions, a calculation can be made. The total area of one floor is given by: 

\[ A_{total} = 30\text{m} \times 29.5\text{m} = 885\text{m}^2 \]

To calculate the area that can be rented out to companies, the remaining distance between the façade and the corridor has to be calculated for both the directions with respect to the core. This gives: 7.8\% to the reinforcement ratio. As can be seen from the results for the top layer, the need for reinforcement is concentrated in the corners of the floors. The highest value is a multiple of the results from the bottom layer, reaching requirements of around 4000\( \text{mm}^2/\text{m} \). If these are added, the reinforcement ratio becomes: 

\[ \frac{1038}{350 \times 1000} = 1.18 + 0.29 = 1.47\% \]

This is still a value which is reasonable in a floor like this one, especially if this is only very locally needed. Therefore, this floor can be considered viable.

To calculate the area that can be let, the remaining distance between the façade and the corridor has to be calculated for both the directions with respect to the core. This gives:

\[ A_{corners} = 4 \times 5.9 \text{m} \times 8.5 \text{m} = 161.66\text{m}^2 \]

The length of the areas between these corners is given by: 30 - (2 \times 0.5 \cdot 0.4) - (2 \times 5.9) = 17.8\text{m} and 29.5 - (2 \times 0.5 \cdot 0.4) - (2 \times 6.85) = 15.4\text{m}. Which leads to the rest of the lettable area using: 

\[ A_{rest} = (2 \times 6.85 \times 17.8) + (2 \times 5.9 \times 15.4 = 425.58\text{m}^2) \]

In combination with the corners this leads to the total lettable area: 

\[ A_{lettable} = A_{corners} + A_{rest} = 161.66\text{m}^2 + 425.58\text{m}^2 = 587.24\text{m}^2 \]

The net lettable area can then be determined:

\[ NLA = (1 - \frac{A_{lettable}}{A_{total}}) \times 100\% = (1 - \frac{587.24}{885}) \times 100\% = 66.34\% \]

In a similar calculation, \( A_{lettable} \) can be determined for the original Zalmhaven tower. Apart from the small differences coming from different wall thicknesses in the design, there is a more significant difference. The shear walls that serve as a lateral stability system cannot be let as office area. These are the red zones in Figure 78. This gives: 

\[ A_{lettable} = 594.2875\text{m}^2 - (6 \times 6.875\text{m} \times 0.4\text{m}) - (4 \times 5.975\text{m} \times 0.5\text{m}) = 565.8375\text{m}^2 \]

With this, the net lettable area of the original Zalmhaven tower can be determined:

\[ NLA_{zalm} = (1 - \frac{A_{lettable}}{A_{total}}) \times 100\% = (1 - \frac{565.8375}{885}) \times 100\% = 63.94\% \]
The difference between the original Zalmhaven tower and the design building can now be determined as a percentage: \( \frac{\text{NLA}_{\text{new}}}{\text{NLA}_{\text{old}}} = \frac{66.34}{63.94} = 1.0375 = +3.75\% \). This increase in lettable area is one of the advantages of the tube-in-tube structure. The percentages for the net lettable area themselves are fairly low. This is mainly caused by the rather large and non-square core of the original Zalmhaven tower. For an office structure that would be designed as such from the start, a higher net lettable area could be achieved by making the core and structure square and reducing the size of the core by placing the vertical transportation needs of the building efficiently.

Figure 78: The floor plan of the redesigned Zalmhaventoren (left) and the original Zalmhaventoren (right). In blue the area lettable as office space is indicated. The corridors are orange and the red parts indicate the unlettable area due to the shear walls.
9 Comparison with prefabricated shear wall Zalmhaventoren

In this section the structural design of the redesigned prefabricated tube-in-tube Zalmhaventoren will be compared with the redesigned prefabricated shear wall Zalmhaventoren from the work of ten Hagen [29]. First, the differences in structural design between the two buildings will be explained. Thereafter, the assumptions used leading to the AxisVM models will be shown. Thirdly, the comparable results between the two structures will be given. Lastly, conclusions of this comparison will be given.

9.1 Structural design

Because the design by ten Hagen is based closely on the original Zalmhaventoren, the comparison between that structure and the tube-in-tube version is almost the same as the earlier section where the Zalmhaventoren was redesigned from the original plan to a tube-in-tube structure. Therefore, in Figure 79 a figure from an earlier section on this topic is repeated.

The most important difference between the two designs is the stability system. The structure by ten Hagen consists of the original shear walls, two 500mm thick in one direction and three 400mm thick in the other direction. Combined with two load bearing façades for the vertical forces this can be considered the main load bearing system. The tube-in-tube structure consists of a prefabricated core and a prefabricated tube around the perimeter of the structure. The difference between the design by ten Hagen and the original Zalmhaventoren is in the floor plan. Where the original (and tube-in-tube) design has a floor plan of 30m × 29.5m, the redesign by ten Hagen has a floor plan of 30m × 30m. This difference has some influence on the exact location and size of the windows and balconies, but the influence is deemed negligible, especially in comparison with the differences that result from the structurally important corner connections in the tube-in-tube variant. Although both structures have a core of 14.4m × 12m, there is a difference within the cores themselves. In the shear wall variant, one of the shear walls splits the core into two equal sections of 7.2m × 12m. This split is not found in the tube-in-tube design.

More consistency can be found in the floor height and total height of the structure. For both designs there are 66 floors of 3.05m, which leads to a total height of $H = 66 \times 3.05m = 201.3m$.

Because of the lack of shear walls in the tube-in-tube version of the structure, the floor system is different from the shear wall design. The simple one-way spanning floors that span from shear wall to shear wall are replaced by two way spanning floors.

Both structures have a lobby that is as high as two floors at the bottom of the structure. Once again, the differences between the two prefabricated structures are the same as the difference between the original and the redesign and a figure is repeated from earlier in the thesis. The connections in the two structures are fairly similar between the two structures. All horizontal connections are unprofiled reinforced connections, which get their stiffness from a combination of factors such as normal stress and reinforcement. The vertical connections between elements are non-structural and do not transfer any forces in the structure. However, the connection between two perpendicular walls is different between the two structures compared in this section. In the shear wall design, the walls cross each other with a staggered connection. In the tube-in-tube design perpendicular walls do not cross each other, but rather meet in a corner. Here a interlocking halfway connection is used.
The Zalmhaventoren designed by ten Hagen is designed purely as a residential tower. This implies that its loads are designed with only this function in mind. For a live load, a value of \(1.75 \text{kN/m}^2\) is used. Because of sound insulation and additional separation walls a high dead load for the floors is used, namely a total of \(9.75 \text{kN/m}^2\). The corresponding values for the tube-in-tube structure are \(3.0 \text{kN/m}^2\) and \(8.0 \text{kN/m}^2\) respectively.

For the redesign as a prefabricated shear wall structure, all elements were designed to be concrete class C90/105. To reach a more financially reasonable structure in the tube-in-tube redesign, lower concrete classes were chosen. C90/105 is only used in the lobby tube, C80/95 on the remainder of the bottom 17 floors and C50/60 throughout the rest of the structure.

### 9.2 Assumptions and the AxisVM model

In addition to the differences made in the previous section, some assumptions have been made before both designs were made in an AxisVM model. An assumption that was made for both models is that the foundations can be modeled as a huge beam under each element that is in connection with the floor. This beam has a dimension of \(1.0m \times 2.0m\) and is supported by a linear spring element in the model. This differs from the original foundation plate and diaphragm walls of the original design of the Zalmhaventoren, but this model is good enough for this comparison.

An assumption that was only done for the tube-in-tube structure is that the floors only transfer normal and shear forces between the core and tube, but no moments. The effects of this on the floors in the model have been explained before. The floors in the shear-wall structure are placed as line loads parallel to the direction of the wind on the shear walls themselves. In the tube in tube structure the line loads are perpendicular to the wind direction and placed in the perimeter of the structure.

Another load placement aspect is the vertical load placement. In the tube-in-tube model, these loads act on the vertical load bearing elements. The same occurs in the shear wall model, but there is one important difference: Because the two load bearing façades in the shear wall model are not included in the model, the parts of the loads that would go into these elements are not included in the model as well. This leads to a total vertical reaction force that does not contain all vertical loads in the structure. For deflection purposes this is probably no problem, but a comparison of reaction forces between models would not be correct because of this difference.

The Young’s modulus of the concrete was estimated in the same way in both models. For both models the difference between cracked and uncracked concrete. For \(E_{\text{uncracked}} = \frac{E}{1 - v^2}\) and for \(E_{\text{cracked}} = \frac{E}{1 - v^2}\).

The connections in both models were made with the same type of elements in the model. For the connections between parallel elements edge hinges were used, for the connections between perpendicular walls, spring ele-
ments were used. In both cases these spring elements are connected to infinitely stiff elements to negate the peak stresses.

In Figure 81 the two models can be seen as they are modeled in AxisVM. The differences are very clear. The shear wall model consists mainly of those shear walls. The floors are not in the model, nor are any of the façades. These elements are modeled in the tube-in-tube model in AxisVM.

In the following table some parameters in the two models will be displayed. It is important to note that differences may occur just because of parameters connected to the design of the structure, such as thickness of the element or chosen reinforcement. The most important comparison is for the order of magnitude, not the precise value:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Shear wall model</th>
<th>Tube-in-tube model</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Overall dimensions</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Floor plan</td>
<td>$30m \times 30m$</td>
<td>$30m \times 29.5m$</td>
</tr>
<tr>
<td>Core size</td>
<td>$14.4m \times 12m$</td>
<td>$14.4m \times 12m$</td>
</tr>
<tr>
<td>Floor height</td>
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<td>3.05m</td>
</tr>
<tr>
<td>Total height</td>
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<td>201.3m</td>
</tr>
<tr>
<td><strong>Elements</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Element thickness</td>
<td>400mm and 500mm</td>
<td>400mm</td>
</tr>
<tr>
<td>Concrete class</td>
<td>C90/105</td>
<td>C50/60, C80/95 and C90/105</td>
</tr>
<tr>
<td><strong>Loads</strong></td>
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<td></td>
</tr>
<tr>
<td>Floor dead load</td>
<td>$9.75 \frac{kN}{m^2}$</td>
<td>$8.0 \frac{kN}{m^2}$</td>
</tr>
<tr>
<td>Floor live load</td>
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<td>$3.0 \frac{kN}{m^2}$</td>
</tr>
<tr>
<td>Wind load</td>
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<td>$1.036 \frac{kN}{m^2} &lt; q_p &lt; 1.923 \frac{kN}{m^2}$</td>
</tr>
<tr>
<td><strong>Foundation</strong></td>
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<td></td>
</tr>
<tr>
<td>Translation stiffness</td>
<td>$K_x = K_y = 1.0 \cdot 10^5 kN/m$</td>
<td>$K_x = K_y = 1.0 \cdot 10^5 kN/m$</td>
</tr>
<tr>
<td></td>
<td>$K_z = 6.97 \cdot 10^5 kN/m$</td>
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</tr>
<tr>
<td>Rotational stiffness</td>
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</tr>
<tr>
<td><strong>Connections</strong></td>
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</tr>
<tr>
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<td>$k_x = k_y = k_z = k_{xx} = k_{yy} = k_{zz} = 0$</td>
</tr>
<tr>
<td>(400mm, uncracked)</td>
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<td></td>
</tr>
<tr>
<td>Horizontal connections</td>
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<td></td>
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<tr>
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<td>$k_{xx} = k_{yy} = k_{zz} = 0$</td>
</tr>
<tr>
<td>Spring stiffness</td>
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</tr>
<tr>
<td></td>
<td>$K_z = 2.93 \cdot 10^6 kN/m$</td>
<td>$K_z = 2.93 \cdot 10^6 kN/m$</td>
</tr>
</tbody>
</table>

### 9.3 Results

As we have seen in the previous two sections, differences between the two models are quite large. This stems from the different lateral stability system and different approaches to the modeling of those systems. This means that there can only be few quantitative results that give a good comparison between the two models. Qualitatively, more aspects of the two structures can be compared. Such a comparison is more subjective and drawing hard conclusions from such a comparison is not always possible.

One of the few results that remains comparable is the deflection of the top of the structure. For both models the same load case in the SLS is used for these calculations. The results of these calculations can be found in the following table:
In this table it can be seen that all values for the deflection remain under the maximum allowable deflection of \( w_{\text{max}} = 403\text{mm} \). Also for the negative wind directions that are not included in the work of ten Hagen, it is very unlikely that this limit will be reached. The direction in which the structure deflects the most is different in both models. Where the shear wall structure is most stiff in the \( y \)-direction, the tube-in-tube structure is most stiff in the \( x \)-direction. The explanation for the tube-in-tube structure was given before and was related to the length of the lateral resisting elements in both directions. For the shear wall structure a similar explanation can be given. In the \( x \)-direction there are only two shear walls and in the \( y \)-direction there are three shear walls. The difference in thickness, 500\( \text{mm} \) and 400\( \text{mm} \) respectively, does not eliminate this difference.

Two other quantitative results are from the section on additional calculations. First, the acceleration at the top of the structure. In this thesis it was determined that an acceleration of \( a = 0.14 \frac{w}{s^2} \) was present at a frequency of 0.21\( H\text{z} \). This value was within the limits for both residential and office structures. Ten Hagen calculated the acceleration of the prefabricated shear wall structure to be \( a = 0.08 \frac{w}{s^2} \) at a frequency of 0.17\( H\text{z} \). These values also are within the limits set in the Eurocode for both building functions.

Secondly, the net lettable area (NLA) was given. The result of this calculation was that the NLA of the tube-in-tube structure was 75% higher for office use. For residential use such a parameter is harder to determine, because the walls in between residences play a roll in the calculation and are dependent on the amount of residences per floor, amongst other things.

One of the main differences between the two structures is the freedom in the floor plan of the structure. In the tube-in-tube structure there are no shear walls present and this can be considered an advantage over the shear wall structure. The possibility of having large open spaces, the ability to change the mapping of each floor for its specified user and the choice of changing its function later in the lifetime of the structure are all good for the overall value of the structure. The lack of these possibilities in the shear wall structure is probably the biggest downside to this option. This difference is also clear in the lobby of the structure, where openness of the floor plan is good for the transport of people and goods. However, the openness of the floor plan in the tube-in-tube structure also has its downsides, especially in the lobby area. Because a lot of the lateral stability is in the perimeter of the structure, the openness of the façade on the bottom floor is harder to retain. In the redesigned tube-in-tube structure already some extra columns were added, but the high stresses remained. The openness of the floor plan is combined with the loss of openness of the façade. What is more important for the design of the structure is not a question for which an objective answer can be found.

Because the perimeter of the structure is important to the stability of the structure for the tube-in-tube structure, the size of the windows and balcony frames is limited by this function as well. This is why an extra column was added in the center of the large balcony frame in the redesigned tube-in-tube structure. Also the thickness of the perimeter elements is larger, especially in comparison with the two non load bearing façades in the original shear wall design. The importance of the perimeter to the stability also prevents the corner windows that were in the original design. Concrete had to be added to tie the different façade walls together.

In one of the previous sections it has been described that the reinforcement percentage in one of the columns was over the recommended maximum value for the tube-in-tube design. Although this proved no problem if the integrity of the concrete could be assured or the Dutch national annex to the Eurocode was used, Ten Hagen found no such results when he designed the reinforcement of two elements (a lintel and a wall). This can be considered an small advantage for the shear wall structure. However, the usage of concrete class C90/105 throughout the entire structure might have something to do with this. In the tube-in-tube design care was taken not to use high concrete classes in areas where they were not needed. This can be considered more economical than the shear wall design.

### 9.4 Conclusion

In this section the results of the comparison will be summarized in a table and some conclusions drawn from that information. The results used are from the models that were created. Possible improvements, such as a better fitting lobby structure for the tube-in-tube structure or different concrete classes in different areas for the shear wall structure will not be present in the table.
Because both structural systems satisfy (or could likely be changed to satisfy) the structural properties of the structure, less important and more soft aspects of the structure have to be compared.

From the results in the table it is clear that both designs have their advantages and disadvantages. If the buildings are compared for just residential use, one could weigh those to determine which is better suited. Is the advantage of an open floor space more important than the openness of the façade? Does one prefer an open lobby façade or an open lobby floor plan?

However if the structures are compared over its entire lifetime and for different functions, it seems that the tube-in-tube structure has something that the shear wall structure lacks: freedom to change. The open floor plan of the floors give the freedom to fit any amount of residences on one floor, from just one to possibly eight smaller units. And not only residences are possible, but also other functions such as offices or education. If the floors contain enough services for these kind of functions changing to other function later on in the lifetime of the structure is possible.

Of course there are other important comparisons that are not looked into in this thesis. For instance whether one of the two models is easier or faster to build. But probably the most important lacking comparison is the difference in cost between the two structures. A good estimate of the cost of the structure however, is outside the scope of this thesis, as it was outside of the scope of the thesis by ten Hagen. This could be an interesting subject for someone else’s thesis.

<table>
<thead>
<tr>
<th>Aspect</th>
<th>Shear wall design</th>
<th>Tube-in-tube design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top deflection</td>
<td>Within limits</td>
<td>Within limits</td>
</tr>
<tr>
<td>Acceleration</td>
<td>Within limits</td>
<td>Within limits</td>
</tr>
<tr>
<td>NLA for offices</td>
<td>63.94%</td>
<td>66.34%</td>
</tr>
<tr>
<td>Reinforcement in elements</td>
<td>No problems found</td>
<td>One column with a $\rho &gt; \rho_{max}$</td>
</tr>
<tr>
<td>Concrete classes</td>
<td>High concrete class C90/105 used throughout</td>
<td>Concrete class C50/60 most used, only higher where needed</td>
</tr>
<tr>
<td>Lobby floor plan</td>
<td>Many columns under shear walls</td>
<td>Open floor plan</td>
</tr>
<tr>
<td>Lobby façade</td>
<td>Open façades used</td>
<td>Extra columns needed</td>
</tr>
<tr>
<td>Corner windows</td>
<td>Applied in structure</td>
<td>Impossible for this stability system</td>
</tr>
<tr>
<td>Balcony frames</td>
<td>Implemented without problems</td>
<td>Extra column in center large balcony</td>
</tr>
<tr>
<td>Changing function of the structure</td>
<td>Only residential function possible</td>
<td>Designed for more uses</td>
</tr>
<tr>
<td>Openness of the floor plan</td>
<td>Many fixed shear walls</td>
<td>Completely open floor plan</td>
</tr>
</tbody>
</table>

Because both structural systems satisfy (or could likely be changed to satisfy) the structural properties of the structure, less important and more soft aspects of the structure have to be compared.

From the results in the table it is clear that both designs have their advantages and disadvantages. If the buildings are compared for just residential use, one could weigh those to determine which is better suited. Is the advantage of an open floor space more important than the openness of the façade? Does one prefer an open lobby façade or an open lobby floor plan?

However if the structures are compared over its entire lifetime and for different functions, it seems that the tube-in-tube structure has something that the shear wall structure lacks: freedom to change. The open floor plan of the floors give the freedom to fit any amount of residences on one floor, from just one to possibly eight smaller units. And not only residences are possible, but also other functions such as offices or education. If the floors contain enough services for these kind of functions changing to other function later on in the lifetime of the structure is possible.

Of course there are other important comparisons that are not looked into in this thesis. For instance whether one of the two models is easier or faster to build. But probably the most important lacking comparison is the difference in cost between the two structures. A good estimate of the cost of the structure however, is outside the scope of this thesis, as it was outside of the scope of the thesis by ten Hagen. This could be an interesting subject for someone else’s thesis.
10 Conclusions

In this section the conclusions from the thesis will be reviewed. Just as the research questions, the conclusions will be divided into three subjects. The first two subjects were treated during the parameter research, the last one was treated during the case study on the Zalmhaventoren.

Shear lag effect of non-rectangular framed tube structures

The first overarching research question was: How do framed tube structures without a rectangular floor plan behave with respect to shear lag? This research question was than divided into more specific research questions. They will be presented and answered below.

• What is the effect on the shear lag if more sides are added to a square floor plan, approaching a circle?

  The effect of adding more sides and corners to a structure with the same circumference has an overall positive effect on its structural behavior. The deflection is slightly lower and for the shear lag there is a noticeable positive effect. This is due to the shorter elements, where the shear lag cannot build up, and the higher number of corners over which the stress peaks are divided.

• Which shear lag behavior can be found in structures with inside corners?

  In an investigation into the differences in shear lag behavior for inside and outside corners no significant differences between the two corners were found. The shear lag factor $P$ was slightly higher for inside corners, but for the other shear lag factor, $P^*$, the found values were slightly lower. It can be concluded that inside corners do not lead to more structural problems caused by shear lag than outside corners.

• What is the influence of corners with an angle different than 90 degrees?

  Multiple different structures with corner angles more acute and more obtuse that 90 degrees were researched, but no differences were found between these different corners angles. It can be concluded that these kind of corner will not cause problems in prefabricated framed tube structures.

Structural behavior of prefabricated framed tube structures

During the parameter research some rectangular structures were also researched to answer some other research questions involving prefabricated framed tube structures. These and their accompanying answers can be found below:

• What is the influence of the aspect ratio (ratio between width and depth of the building) on the shear lag factor in a prefabricated concrete framed tube structure?

  The effect of changing the aspect ratio of a rectangular structure has a small and negligible effect on the shear lag factor. The best ratio with respect to the shear lag is $A_r = 1$, but the effects of changing this factor up to 2.5 or down to 0.4 are only 12% and 5% respectively.

• What is the influence of the slenderness on the shear lag factor in a prefabricated concrete framed tube structure?

  A clear relation between the slenderness and shear lag factor was found. For structures with a low slenderness, the shear lag factors $P$ and $P^*$ are significantly higher. This is due to the fact that less slender structures rely more on their shear stiffness against lateral loads, where slender structures rely more on their bending stiffness to deal with lateral loads. More shear leads to more shear lag.

• Does changing the non-structural joints in a masonry configuration to structural joints for a framed tube structure have a positive influence on the structural behavior?

  The influence of adding structural joints in a prefabricated structure with a masonry configuration that used 50% overlap on the shear lag is non-existent and negligible for the top deflection. This influence might be higher for masonry configurations with a smaller overlap of the elements.

• What is the influence of the corner stiffness on the structural behavior of a prefabricated concrete framed tube structure?

  A better connection between the web and flange walls of the structure as a results of a better corner connection stiffness is positive for the structural behavior. The vertical reaction forces in the flange walls are higher and this leads to a smaller top deflection. However, increasing the stiffness from the estimated value used has a far smaller positive effect than the adverse effect decreasing causes.
Case study Zalmhaventoren: Potential of fully prefabricated tube-in-tube structures

The last part of this thesis was focused on the following research question:

*Is it possible to build a structurally feasible prefabricated tube-in-tube high-rise structure in the confines of the Dutch building market of approximately 200m high and how does it compare to other prefabricated solutions?*

One of the problems encountered with this research question is the Dutch building market. From literature research it was found that building structures in the Netherlands of approximately 200m is most likely not in the best economic interests of the developer. Companies sometimes give enough value to being in a tall or even the tallest tower which might make it worth developing a structure with such a height.

However, the literature research did provide a building which could be designed as a prefabricated tube-in-tube structure with the Zalmhaventoren. An added advantage was that this structure was also already redesigned to a prefabricated shear wall structure in another Master’s Thesis, providing a model to compare the redesigned structure to.

A lot of the parameters of the original Zalmhaventoren were unchanged in the redesign of the Zalmhaventoren. The structure has a floor plan of 30m × 29.5m and has 66 floors with a floor height of 3.05m, giving the structure a total height of 201.3m. The structure was checked on several structural results, such as static top deflection, shear stress in the interfaces between the elements and its dynamic behavior. All these results proved to be within the limits. When detailing two governing elements, the reinforcement of one column was found to be relatively high (exceeding the recommended value in the Eurocode). However, this value was still within the limits of the Dutch national annex. The high reinforcement percentage value is in part because of the positioning of this column close to the lobby, which was kept as a rather open structure to be able to compare the results to the design with the prefabricated shear walls.

Besides the main model, two other tests were done to research the effect of certain aspects on the most important result: the top deflection of the tower. The first one was the difference between a cast-in-place model and the prefabricated one. The increase in top deflection for the prefabricated structure was around 8%. Not negligible, but not high enough to stop considering prefabricated tube-in-tube structures from being viable. The second check was whether removing the balconies, and with them the frames around the balconies, and replacing them with ordinary windows had a positive effect on the top deflection. It was found that implementing this change with the same element thickness reduced the top deflection by 34%-42%. Even when the thickness of all the elements was reduced from 400mm to 300mm, the top deflection of a structure without the balconies was better than for the model with balconies and a element thickness of 400mm.

In the comparison with the structure from the work of ten Hagen, it became clear that both lateral stability resisting systems could work for the Zalmhaventoren. Both had advantages and disadvantages that could be weighted either way by different developers if the structure was designed purely as a residential building. However, if the tower was compared as being a multipurpose building, the open floor plan of the tube-in-tube structure provides more flexibility at completion, but especially over the entire lifetime of the structure.

Ultimately this research gives insight into the structural behavior of the prefabricated tube-in-tube structure and proved that the possibilities of the prefabricated tube-in-tube structure lie even beyond the 200m, especially if the technologies associated with this stability system keep improving over the coming years.
11 Recommendations

In this thesis the subject of prefabricated precast concrete framed tube and tube-in-tube structures has been addressed. To do this research simplifications and assumptions were made to find the answers to the research questions set at the beginning of the process. Researching this subject also led to even more questions with respect to specific parts or elements of the researched structures. In this section recommendations for further research will be given. It contains both new research subjects and tips for improving the process.

- Creating the models in AxisVM took a lot of time during this research. Also, calculating the larger models took over an hour for certain cases. The results these models give are detailed, but might be too detailed for designers who are just looking to answer whether the top deflection is within limits or what concrete class might be needed. A parametric model could serve as a welcome tool for designers in the early stages of design. Such a parametric model could be created and verified in the future.

- For shear lag in framed tube structures a lot of different parameters have been investigated in the parameter research. However, all the models used were symmetrical. Because asymmetrical structures are not that uncommon, the shear lag effect in structures which are not symmetrical could be researched.

- The horizontal connections in the perforated tube of the structure have a shear stiffness that, among other things, depends on the normal stress in the connection. Because the normal stress distribution is far from uniform, with high peaks near the columns and minimum values near the windows, the connections are hard to model realistically. Research into this type of prefabricated connections with different shear stiffnesses within the same element could result in less conservative calculation methods.

- Because the scope of this thesis was focused more on the entire structure and its behavior than on the smaller elements it consists of, the detail of the design structure is fairly low. For only a few elements some global calculations were made for the reinforcement. More research could be done on the reinforcement of the elements, the detailing of the connections and the way the floors work together with the lateral resisting elements.

- In the comparison between the two lateral load resisting systems the two systems were compared for several aspects such as structural behavior and functionality for different functions. It however excluded some other important aspects, such as the construction of the structure and probably most important, the difference in building costs. Research into these aspects is a possibility.

- The focus of this thesis was on the prefabricated concrete tube-in-tube structure within the Dutch building market. Because of the high-rise culture in the Netherlands and the daylight requirements, structures much higher than the designed 200m are not possible. However, the possibilities of the structural system itself can be over 200m if larger floor plans, thicker elements and higher concrete classes are used. More research could be done on the possibilities of the structural system outside the Netherlands.
References


A Literature Report

In this appendix a part of the literature report will be presented. It contains background information for the first three chapters of the main thesis and gives a broader view into the context of this thesis than could be contained in those three chapters.

A.1 High-rise in prefabricated concrete

A.1.1 Common connections between shear wall elements

The main difference between a cast-in-place structure and a structure consisting of prefabricated concrete elements is the discontinuation of the concrete in the latter one. Because of this discontinuation, joints appear in the structure. In this section, a review will be given of the types of joints that are used in practice. A distinction between horizontal and vertical, as well as structural and non-structural will be given. This section is based on work of Falger [10] and de Boer [8].

First, the vertical joints will be discussed. For vertical joints, there are two main types, structural joints and non-structural joints. These two types are dependent on the configuration of the elements of the wall these elements are in. In a vertically stacked configuration, continuous vertical joints appear from the bottom to the top of the wall. When this wall is loaded laterally, shear stresses have to be taken up by these vertical joints. So for this configuration structural joints have to be present. In a masonry configuration however, there are no continuous vertical joints and the shear stresses are taken up by the elements themselves. Here, non-structural joints, which are easier and cheaper to construct, will suffice.

The first vertical joint is a so-called wet connection. This means that after both elements are installed, concrete is poured between them to get a stiff connection between the elements. To achieve collaboration between the two elements both elements have protruding stirrups at the end of the element. After the elements are placed, an extra reinforcement bar will be placed through these stirrups to tie the reinforcement together. After the reinforcement is in place, a formwork is placed around the joint and filled with concrete. This connection is called a smooth wet connection and can be seen in the figure.

The second vertical joint is very similar to the first one. It also has the protruding stirrups and the extra reinforcement bar. This is also a wet connection so concrete is poured to fill the gap between the elements to achieve collaboration. The only difference is the shape of the ends of the elements. These elements do not have a smooth surface, but are profiled. In this way, compression cords can form between the “teeth” of the elements. This leads to a higher strength than the smooth wet connection. This joint is called a profiled wet connection.

Another variation of this joint is one using flexible loops instead of protruding reinforcement stirrups. Creating these loops is an easier process during fabrication of the elements, but suffers a stiffness decrease compared to the joints with the protruding stirrups.

In contrast with the wet joints, there are the so-called dry joints. These joints are constructed without concrete or mortar, which means that they do not have to cure and are at their full strength immediately after construction. Two slightly different dry connections will be discussed. The first one is a connection with a steel plate cast...
A profiled wet connection between two shear wall elements in prefabricated concrete (Courtesy of [8])

into the element. To connect this element to the next one, the elements have to be welded together using a steel coupling plate after placement. The placement of these steel plate matters for the stiffness of the joint as a whole. For instance, if a joint is heavily loaded by shear forces placement of the steel plate in the middle of the element is preferred to prevent problems due to eccentricities. All steel plates are anchored in the element by reinforcement bars. The second dry connection that will be covered in this section is one with a cast in UNP channel profile. A UNP profile is cast at the end of each element, with the web facing the connection. When the webs of the two elements are placed against each other during construction, coupling plates on both sides of the elements will be used to weld the flanges of the two profiles together. Instead of a UNP-profile, a steel tube can also be cast in the element. This does not change the principle of the connection. Using a UNP-profile or steel tube generally creates a stiffer joint than a normal cast in steel plate, but the stiffness reached in comparison to the wet connection remains low. Furthermore, welding at the construction site can be influenced by the weather conditions and take up some valuable time in the construction process. And if welding on both sides of the elements is necessary, a scaffolding on the outside of the structure is required.

For structures with a masonry configuration it is unnecessary to have a stiff vertical joint. The elements above and below the element will take up the shear forces when a element ends. However, because of building physics requirements for water and air tightness and fire safety reasons the 20 to 30mm between elements has to be filled with a non-structural material which is naturally not reinforced.

For horizontal joints there are not many different types. They basically all use the same design, which can be
changed to fit the requirements by using a little more or a little less reinforcement or using mortar of a higher or lower quality. The design consists of protruding reinforcement bars at the top of each element and recesses in the bottom of each element. During installation, the protruding bars are placed in the recesses of the next element. Hereafter, the recesses and the approximately 20mm of space between the elements are filled with mortar to connect the elements together and obtain some axial and shear stiffness in the horizontal joint.

![Figure 85: Horizontal joints in a prefabricated shear wall structure in concrete (Based on an image courtesy of [8])](image)

A.1.2 Suitable floor systems

All structures, so also prefabricated high-rise structures, need floors in them. Many different floor systems are available in the market for all different kind of applications and conditions. For prefabricated high-rise, some floor systems are more suitable than others. Because the main advantage of high-rise in prefabricated concrete is the increased building speed, the used floor system is also preferably a quick one. Another point of interest in high-rise is the ability of a floor to transfer the lateral loads on the building in its own plane. In this section three different (semi-)prefabricated floor systems which are used in practice will be discussed, including their advantages and disadvantages.

The first floor system is a well known one: the hollow core slab. Hollow core slabs are prefabricated concrete slabs with circular voids in the direction of their span. These voids reduce the weight of the slab significantly without reducing its bending stiffness too much. Weight reduction in a high-rise structure is very important, because all the weight saved on one floor really adds up in the lower regions of the construction. Because all hollow core slabs are prestressed they are able to span spans over 10m. In framed tube high-rise, they usually span from façade to façade or façade to core.

The connection at the façade elements is usually detailed using a corbel attached to the wall element. The slabs are placed on this corbel and divert the vertical forces through them. To connect the façade and the floors some of the recesses of the hollow core slabs are exposed and reinforcement is placed the recesses. After all the elements are in place concrete is poured to tie the floors and façade together. Usually a top layer is poured onto the hollow core slabs to create a even surface and to increase the diaphragm action. Especially in residential buildings, a top layer could also be added to increase the weight of the floor for sound insulation. Another way to achieve a good sound insulation is to use massive prefabricated concrete slabs instead of the hollow core slabs. The structural system remains the same.

The main advantage of using prefabricated slabs is that a quick erection is indeed possible. No temporary studs are needed to construct the floors. If the element is placed only the connections need to be finished before the floor is at its full strength. The reduced weight is also an important advantage of hollow core slabs, especially in high-rise structures. Disadvantages can also be found. A hollow core slab is a typical one way spanning floor system. For structures with irregular floor plans this can mean that this floor system is less suited. The corbel
needed at the supports is another disadvantage because some users do not like the them in their rooms and sometimes architects want to avoid them at all costs.

The prefabricated concrete planks (in Dutch: breedplaatvloeren) is another option for a floor system in a prefabricated high-rise building. This floor system consists of prefabricated prestressed concrete elements of around 50mm thick with some steel elements sticking out at the top. Reinforcement is connected to these elements and then concrete is poured onto the prefabricated slab to create a floor which can carry not only its own dead load but also the variable loads in a structure. During the concrete casting, temporary studs are needed and they remain necessary until the concrete has cured. The necessity of the studs can be considered a disadvantage of this system because setting up and removing the studs can be time consuming. The reinforcement and supports of prefabricated concrete planks can be designed to be either one-way or two-way spanning. Either way they are usually placed partly on the prefabricated façade wall and connected to these walls by reinforcement stirrups and a poured concrete connection. Concrete planks have the advantages belonging to semi-prefabricated
systems like this one. The transport weight of the elements per square meter is low and the vertical transport of these elements can be faster than that of heavier elements. It is also possible to cast all ducts and other services in the layer on top of the prefabricated plank, something which is impossible with hollow core slabs for instance. But of course there are also disadvantages to this system. The most important one is the necessity of temporary studs which can negatively influence the cycle time of a floor construction. The last floor system that will be discussed in this section is the Bubbledeck floor. This floor has a lot of similarities with the prefabricated concrete planks, because it also is a semi-prefabricated floor system. The floors are transported to the building site as prefabricated elements with a thickness of around 50mm. On top of these elements, between two layers of a reinforcement grid, plastic balls are placed. These reduce the weight of the floor by creating voids in the floor without diminishing the strength and stiffness of the floor too much. Again, concrete is poured on the prefabricated elements to obtain a structural floor that is able to withstand the loads that are present in a building. During the casting of the concrete temporary studs are necessary until the concrete is cured and reaches its full capacity. Bubbledeck floors are usually applied as two way spanning floors and can be used for floor plans in all different shapes and are able to create some fairly large cantilevers if properly reinforced. The connection to the façade elements is fairly similar to the one of prefabricated concrete planks. The main disadvantage of the Bubbledeck floors is the same as for the concrete planks, the temporary studs can increase the building time of the structure. This floor system however, does have some advantages as well. Besides the weight reduction and the lightweight transport that were already discussed at the hollow core slabs and prefabricated concrete planks respectively, the Bubbledeck floor stands out because of its ability to be used in less commonly shaped floor plans. The two way spanning properties could also lead to less columns or columns with smaller dimensions.

A.2 Relevant Research

In this section previous research in this research field will be described, its conclusions given and the relevance of these conclusions for this thesis provided. A lot, but not all of the research found was done as Master’s theses at Delft University of Technology in the last decade. There are several reasons for this. First of all is prefabricated high-rise a fairly new research field, without many decades of research to go through. Secondly, the Netherlands are at a forefront in respect to the use of prefabricated concrete, high-rise structures in particular. It is therefore no surprise that a lot of the research is done in the Netherlands as well. Stuifd, a Dutch study association that promotes the development of theoretical and practical knowledge with respect to concrete, teamed up several times with a student of the TU Delft to provide more insight in certain areas of research.

A.2.1 Ontwerpregels voor betonnen gevelbuisconstructies, S.J.M.G.Faessen, 2000

This research [9] was conducted as a Master’s thesis at the Delft University of Technology. The Dutch title translates to Design Rules for Concrete Framed Tubes.

Description of the research

This research describes the framed tube structure as a viable stability system for multi story structures. The goal of this research was to make design rules for framed tube construction in both cast-in-place concrete and prefabricated concrete structures. These rules could be used by designers to determine in an early stage the top deflection and the force distribution within the structure by only using some key parameters such as building height, width and depth in combination with the sizes of the columns and beams. The focus in this report will be mainly on the prefabricated concrete parts of this thesis.

The design rules were created by a parameter study that was performed using Finite Element Method (FEM) called ANSYS. Some parameters had a quite narrow bandwidth as can be seen in the table below:

<table>
<thead>
<tr>
<th>Parameter name</th>
<th>Minimum value</th>
<th>Maximum value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building Height</td>
<td>15m</td>
<td>150m</td>
</tr>
<tr>
<td>Web façade Width</td>
<td>14m</td>
<td>18m</td>
</tr>
<tr>
<td>Flange façade Width</td>
<td>36m</td>
<td>80m</td>
</tr>
<tr>
<td>Beam Height</td>
<td>0.8m</td>
<td>1.5m</td>
</tr>
<tr>
<td>Column Width</td>
<td>0.4m</td>
<td>1.3m</td>
</tr>
</tbody>
</table>

In which the web façade width is the length parallel to the main wind direction and the flange façade width the length perpendicular to the main wind direction. The floor height was kept constant at 3.6m.

Conclusion of the research

The outcome of this research are design rules for framed tube construction in (cast-in-place and) prefabricated concrete. The design rules can be used to find the first and second order deflection of the structure, as well as the reaction forces in the corner columns, the normal and shear forces in the corner columns and the maximum bending moments. Some of these calculations require quite some steps and can be considered fairly intricate. Although design rules like these still take some time from an engineer to work out, way less time is required in
comparison to creating a FEM model. And given that older design rules were found to be inaccurate in this research, the formulation of these design rules was a fine outcome for this thesis. Using these design rules other conclusions were found as well. The main conclusion was that deflection is governing in concrete framed tube high-rise structures. Some other conclusions are listed below:

- With respect to the difference between framed tube structures that are cast in situ and the ones that are made out of prefabricated concrete, using the design rules it was calculated that prefabricated structures have a top deflection 2 to 3 times higher than a cast-in-place structure. Also the reaction forces in the corner column are higher, 1.2 to 1.6 times compared to the cast-in-place variant.
- Increasing the height of the beams or increasing the width of the columns has a positive influence on the top deflection, reducing it considerably
- The closer the length of the web façade and flange façade are together the lower the top deflection is. This can be explained by the fact that the increased wind loads of a longer flange width of the building cannot be taken up by the slightly higher flange façade capacity in the structure
- In respect to the building height, \( u_{bending} \) increases to the power 4 and \( u_{shear} \) increases quadratic
- The maximum number of floors that can be reached in prefabricated concrete framed tube construction is around 30, but only if the beams are very high and the columns very wide. For lower values of these dimensions 10 floors is the maximum

Relevance to this thesis
This research is very relevant with regard to the topic but at the same time less relevant because it has become somewhat dated since it was conducted 13 years ago. Especially its somewhat harsh conclusions related to prefabricated framed tube constructions have been made implausible by newer research and currently constructed buildings, which can be found in the remainder of this report. Furthermore, the research focuses on a specific shape and fairly specific dimensions, something which is looked into further by Prakoso in the next section.

A.2.2 Ontwerpregels voor betonnen gevelbuisconstructies, P.S.Prakoso, 2001
This research [22] was conducted as a Master’s thesis at the Delft University of Technology. The Dutch title translates to Design Rules for Concrete Framed Tubes.

Description of the research
This research report has got the same name as the research by Faessen described above. It can be seen as a continuation and an improvement over the work of Faessen. In this thesis it is stated that the design rules created by Faessen, with its small web façade width, have been made with a very European mindset in respect to the floor plans. In other areas of the world the requirements for daylight in work spaces is less strict and the web façade width can be higher. Also the parameter for the height of the building is slightly increased. The following parameters were used for this research:

<table>
<thead>
<tr>
<th>Parameter name</th>
<th>Minimum value</th>
<th>Maximum value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building Height</td>
<td>18m</td>
<td>180m</td>
</tr>
<tr>
<td>Web façade Width</td>
<td>18m</td>
<td>36m</td>
</tr>
<tr>
<td>Flange façade Width</td>
<td>36m</td>
<td>36m</td>
</tr>
<tr>
<td>Beam Height</td>
<td>0.8m</td>
<td>1.3m</td>
</tr>
<tr>
<td>Column Width</td>
<td>0.4m</td>
<td>1.5m</td>
</tr>
</tbody>
</table>

The differentiation in the dimensions of the columns and beams is the same as in the research by Faessen. The same Finite Element Method program was also used, being ANSYS. Again, cast-in-place and prefabricated framed tube structures were looked into.

Conclusion of the research
Prakoso managed to improve upon the design rules that were made by Faessen. Creating design rules that fit not only the buildings researched in this report but only all the buildings researched by Faessen. The intricacy of these design rules went up even further in comparison to the rules they were based upon. By doing so the effect of the width of flange façade was captured. For the design rules and the exact reason behind some of these changes one should read the full report by Prakoso.

Some additional conclusions were found based on these design rules in a similar way as they were found by Faessen. Some of these conclusions, with respect to the influence of the column and beam dimensions and the increase of \( u_{bending} \) and \( u_{shear} \) over the building height were even exactly the same. Some new and updated conclusions were found as well. The first one is the influence of the web façade length on the structure. By increasing this parameter both the deflection and the forces in the structure decrease making it able to build
higher structures using the same system. An updated conclusion is one with respect to the possibilities of prefabricated framed tube construction. For every increase of about 7m to the length of the web façade between 5 and 10 additional floors can be constructed. Because of this, this research states that the maximum number of floors that can be built using this structural system is not limited to 30, but can be as high as 50 floors. Also the maximum number of floors possible in a construction with small column and beam dimensions is doubled from 10 to 20 floors.

**Relevance to this thesis**
Although this thesis is very similar to the thesis of Faessen, the relevance for this thesis is perceived higher. Especially the conclusions in respect to the possibilities of prefabricated framed tube high-rise seem more realistic and positive for prefabricated structures. Because the improved design rules fit the structures from both reports it should be fine to use the new design rules where early estimates for deflections or internal forces are necessary.

### A.2.3 Logistiek bij Hoogbouw, J.Pronk, 2002

This research [23] was conducted as a Master’s thesis at the Delft University of Technology. The Dutch title translates to *Logistics for high-rise.*

**Description of the research**
This thesis is a research done into the logistics of high-rise from a business administration point of view. It was conducted as combination of a literature study and a study case.

The literature study focused on several aspects of construction. First of all the building world with all its participants was dissected to find all the players and their interests. Secondly vertical transport was looked into, because the higher you build a structure the more important and critical the vertical transport becomes. The third and last part of the literature research was into all the physical flows on a construction site, from materials and waste to the workers.

The second part was a study case for the Millennium Tower in Amsterdam. This is an office building of 24 floors and a height of 97.5m. Its construction started in 2002 and it was finished in 2004. For this building two different construction plans were made, including a planning, the amount of labor needed, costs and other aspects of construction. The two different plans were one using traditional building methods and one using prefabricated concrete. Hereafter the construction plans were compared to each other.

**Conclusion of the research**
While performing the literature study and the study case of the Millennium Tower in Amsterdam, Pronk got the following conclusions.

The most important aspect in high-rise construction is vertical transport. This is always the critical path and therefore problems with regard to vertical transport always leads to delays. Not only the vertical transport of materials is an important factor in the building process. With increasing height of the building the vertical transport of workers becomes more time consuming and critical to construction.

Using prefabrication the building time can be reduced to a great degree. The materials for prefabricated building can be more expensive, but due to the reduced building time loans can be shorter and income can be generated earlier, which could make up for the higher material costs.

![Figure 88: Millennium Tower in Amsterdam (Courtesy of blogspot.com)](image-url)
Comparing the traditional and the prefabricated concrete construction the ratio between the costs of materials and the costs of labor changes drastically. For traditional construction this ratio is around 2:1, for prefabricated construction closer to 20:1. This leads to the conclusion that coordination of labor in traditional construction is a focus point. For prefabricated structures, good teamwork for the few workers is deemed more important. Because the building speed is higher in prefabricated construction, loss of building time is a bigger problem. Because prefabricated construction is fairly wind dependent a bad year with respect to wind can have really terrible effects on a planning. Because there is no time that concrete has to heal before moving on in prefabricated construction working around the clock or in the weekends is a possibility to make up for lost time. This however does have financial implications.

The last conclusion from this report is that especially in a high-rise project it is very important that all disciplines are talking with each other in early stages of design to make a high level of prefabrication possible. A Design & Build style contract would be a good way to ensure this collaboration.

**Relevance to this thesis**

The report of Pronk is a good insight in the construction process of a high-rise building of 10 years ago. Its conclusion about vertical transport being critical is even more relevant to structures higher than the one looked into in the study case (97.5m) and therefore is very relevant to this thesis. All but one of the other conclusions, in which the wind dependency of prefabricated construction is pointed out, seem to be highly in favor of this building method. Praising its shorter building time, less labor on site and earlier rent incomes, Pronk thinks building high-rise structures using prefabricated elements is the future. The author of this report agrees on this point, although building higher than 200m has its own problems, as will be described in the section on van der Meij [32].

A.2.4 Geprefabriceerde betonnen stabiliteitsconstructies met open verticale voegen in metselwerkverband, M.M.J.Falger, 2002

This research [10] was conducted as a Master's thesis at the Delft University of Technology combined with a research team of the Stufib Study Association. The Dutch title translates to Prefabricated Concrete Stability Structures with Open Vertical Joints in Masonry Configuration.

**Description of the research**

The research of Falger starts with a thorough literature study of several aspects of prefabricated concrete stability systems. Aspects such as framed tube structures, traditional vertical joints, progressive collapse and earlier research into masonry configurations are all described. For the earlier research into masonry configuration the two that were perceived as best were the Master's theses by J.G.A. Snelders, TU Eindhoven, 1994 and J.W. van Dorst, TU Delft, 1995. Some of the conclusions Falger deemed relevant for his further research are the following:

- The better the non-structural vertical joints are distributed over the width of the wall, the closer the force distribution is to that of a cast-in-place wall
- The positive effects of a masonry configuration is lost if the overlap between elements in different layers is less than 25% of the element width
- To make sure the elements work together the width of the smallest element cannot be smaller than the height of the elements.
- In the mid-90’s there was no calculation method available that was both accurate and not too time consuming

After the literature study Falger proceeds to conduct a parameter study using a Finite Element Method model. The FEM program of his choice was ATENA. The set-up of the the parameter study is the following. Four wall types are defined, as can be seen in the first figure in this section. Type A is a wall without any windows or other openings in it. Type B is a wall with only one opening in the center of the wall which could be used as a passage for a corridor. Type C and type D both represent a façade wall with windows. The difference between these two types is the amount of windows and how close to the end of the wall they are situated. The size of the walls is the same for all types. The total height of the wall is 24 times the floor height of 3.6m and becomes: $H = 24 \times 3.6 = 86.4m$. The width of the wall is 14.4m, giving the wall a slenderness ratio of $\frac{H}{W} = \frac{86.4}{14.4} = 6$. For all wall types, Falger calculated 6 different configurations of elements and their joints to get a sense of the behavior in these different cases. These configurations can be placed in one of 3 different groups. The first group consists of cast-in-place walls, the second group of walls with a vertical element configuration and the third of walls with a masonry type configuration. The 6 different configurations are:

1. cast-in-place wall
2. Vertical configuration with a smooth wet reinforced structural vertical joint
3. Vertical configuration with a profiled wet reinforced structural vertical joint
4. Vertical configuration with a structural vertical joint made of a welded cast in plate
5. Vertical configuration with a structural vertical joint made of a welded cast in UNP-profile

For all 4 different wall types and 6 different configurations a model in ATHENA was created. Each of these models were given the same horizontal loading which in practice would be present as the result of a wind loading on the structure. These 24 FEM results, combined with two hand calculations for a cast-in-place wall of type A and type D were compared to each other and lead to the conclusions covered in the next section.

**Conclusion of the research**

By using this many comparable models, Falger made it easy to define a lot of conclusions. With respect to the non-structural joints and the masonry configuration he defined the following:

- Although there is no stiffness in the vertical joints, the deviations in deflection behavior between a masonry configuration and a cast-in-place wall are very low. The deflection increase ranges from a minimum of 5.3% for the solid wall (Type A) to a maximum of 8.0% for wall type B. This behavior is comparable to the stiffest joint in the vertical configuration (Profiled wet reinforced joint).
- Because of the small differences, it would be acceptable to use a calculation for a cast-in-place construction in the design phase when designing a prefabricated structure with a masonry configuration.
- The shear stresses that are usually taken up by the vertical joints, are now taken up by the elements below and above the non-structural vertical joint. The increase of these shear stresses is fairly high percentage wise (up to 84%), but remain low in absolute value (less than $1 \frac{N}{mm^2}$).
• Although the deflections of a masonry structure are comparable to a cast-in-place structure, the internal forces are not. Especially for wall types C and D the moments and the shear forces can be up to 45% higher. These values are comparable with the results from the wall with the structural vertical joints.

Because Falger had 4 different joint types in the second type of wall configurations, a lot of data concerning the differences between these joints was obtained. He used this information to define the following conclusions:

• Using prefabricated elements in a stability structure will always effect the deflections of a building. A clear relation was obtained between the stiffness of the vertical joints and the deflection of the building. A decreasing joint stiffness leads to an increasing wall deflection. This relation is non-linear.

• A structure’s reaction to a change in joint stiffness is more clear if the stiffness of the joint is low.

• Although the difference in joint stiffness between the stiffest connection (profiled wet reinforced joint) and the least stiff connection (welded plate connection) is a factor 6.4, the difference between the resulting deflections is fairly low (only 14% in comparison with the cast-in-place variant)

• If a joint is present in a column in wall type type C or D the forces and moments in these columns are lower (up to 40%) than is to be expected from a cast-in-place calculation. The forces in the columns next to these split columns are increased by similar percentages. Obviously the stiffer elements in the structure collect a higher amount of the loads.

All in all, Falger concludes that stacking prefabricated concrete elements in a masonry configuration is a very viable alternative to the traditional vertical element configuration and its structural joints.

Relevance to this thesis
This thesis by Falger seems to be of high quality all through the report. Both the literature study and the FEM-model study can be considered thorough and yield some helpful conclusions for the remainder of this research. The geometric conclusions, with respect to elements sizes and overlap, found in the work done in the mid-90’s by Snelders and van Dorst are very clear and should be used throughout the remainder of this research.

The conclusions that are found about the behavior of masonry type configurations in stabilizing walls are also interesting. Because their behavior is arguably the best of all prefabricated options, combined with the fact that less work on site is needed because of the non-structural joints, this configuration should be the primary option in any design conducted in this thesis.

If for some reason a vertical configuration is required, the conclusions based on the study into different kind of joints is helpful to see the implication of design choices for different joint types. Especially the influence of the stiffness of the vertical joint is well described.

A.2.5 Verband in Hoogbouw, K.de Boer, 2004
This research [8] was conducted as a Master’s thesis at the Delft University of Technology. The Dutch subtitle (Uitvoeringsaspecten van in verband gestapelde geprefabriceerde betonnen wanden) gives a better indication of the thesis subject and translates to Construction aspects of prefabricated concrete walls in masonry configurations.

Description of the research
This research was conducted as a follow up of the thesis in the last section by Falger. Falger proved that the structural properties of concrete elements in masonry configurations were very good. This thesis asks whether the construction aspects do also have a positive effect in comparison to elements in a regular configuration and structural vertical joints. To determine whether this is the case, a reference project was used (Prinsenhof, The Hague). This reference project was worked out with the different kind of configurations and vertical joints to give an indication of the differences in cycle time and costs. The information needed to create such a study case was obtained by literature research and a survey among several professionals in different industries connected to the construction of these prefabricated structures. Three different configurations were worked out. The first one is the traditional vertical configuration with structural vertical joints. 4 different joint types were distinguished within the configuration.

• Wet reinforced connections

• Looped connections

• Welded plate connections

• Profiled connections
The second configuration is the masonry configuration which does not require a structural joint. For the last configuration a combination of the other two is used. The elements are stacked vertically but do not use vertical structural joints but instead rely on the staggered connection in the corners (see the section on Tolsma [30] for a further explanation) and the dowel action of the edge beam (as will be explained in the section on Pieterse [20]).

**Conclusion of the research** The main conclusion of this report is that building a framed tube structure in prefabricated concrete in masonry configuration has got financial advantages over the configuration that leads to structural vertical joints. These advantages follow from both the simplicity during the production of the element and the simplicity on the construction site.

Factors that could increase the costs of this method were also researched. From this it was concluded that the need for extra molds does not have a negative effect because in a high-rise structure repetition of more uncommon elements is still fairly high.

Some points of interest were presented next to the conclusions. For instance, the influences on the building time of the masonry configuration is low. Only in cases were an extra scaffolding is needed to construct the vertical joints from the outside time can be won in comparison to the non-structural joints. The reason behind this is that the construction of the joints is not in the critical path of the construction process. Another point of interest is that it could be aesthetically unwanted to create a masonry configuration and that therefore another choice could be made.

**Relevance to this thesis**
The relevance of this research for this thesis is a confirmation of the conclusions from the research of Falger. Using a masonry configuration instead of a vertical configuration was already proven to be the structurally better choice. After this research concluded that also for construction purposes this is the best choice in prefabricated concrete framed tube construction a choice can be made for a direction of the designs within this thesis. A masonry type configuration is preferred in this thesis for both structural and construction reasons.

A.2.6 Deuvelwerking van randbalken in prefabbouw, E.A.Pieterse, 2006

This research [20] was conducted as a Master’s thesis at the Delft University of Technology combined with a research team of the Stufib Study Association. The Dutch title translates to *Dowel action of edge beams in prefabricated construction*.

**Description of the research**
Although the research of Falger proved that a masonry configuration was a good alternative in stability walls, in practice a preference for vertical element configuration remained. In the research of Falger the effect of the floors on the stiffness of the structure was not investigated. In a vertical configuration however, the effects of the floor and edge beam could be beneficial.

For the investigation of this behavior first a literature study was performed. This resulted in some data that could serve as comparison material for the rest of the project. The next step was the creation of five Finite Element Method models and test them until one was deemed best. This model is shown in the image in this section. After that the parameters of the modeled were defined. The parameters that are variable in the model are:

- Dowel Height
- Dowel Width
- Concrete Quality
To calibrate the FEM-model a few real life tests were performed in the lab of the TU Delft. For both strength and to a lesser extend stiffness the validity of the model was proven using the tests. With an improved model the parameter study could be performed leading to some conclusions that will be discussed in the next section. After the validation and the parameter test the dowel action was implemented into a macro model of a 10 story high wall, also leading to some conclusions.

**Conclusion of the research**

The main conclusion of this thesis is that there is a great potential using a concrete dowel in these kind of structures. Furthermore, some conclusions were found in the parameter study. A linear relation was found between the overall strength and stiffness of the dowel and the dowel width, the dowel height and the concrete quality. In the macro model it was found that the 10-story wall could resist the wind loading that was introduced in the model. The highest forces in the dowel were close to $100 \text{kN}$, while the maximum allowable forces were closer to $175 \text{kN}$. The deflection of the wall was also compared to a wall constructed with welded plates as vertical structural joints. The deflection at the top of the concrete dowel model was 50% higher for the new model using dowel action.

**Relevance to this thesis**

The relevance to this thesis is highly influenced by the last sentence of the last section, in which the increased deflection compared to a model with welded plates is discussed. An increased deflection of 50% for a wall of only 10 stories high can be considered a lot if one were to realize that in serious high-rise construction, such as this theses, around 50 stories are necessary. Because the top deflection is often governing in high-rise construction an increase of 50% can be enough to make a project (financially) impossible. Furthermore, if the stiffness of the construction is reduced significantly, even higher differences between a welded plate construction and a dowel action construction can be expected. Because the masonry configuration is even stiffer than a connection with welded plates (according to Falger [10]) this system cannot be considered as the best option for a prefabricated high-rise structure of approximately 200m, even though it might have a great potential for lower buildings.

### A.2.7 Precast concrete cores in high-rise buildings, K.V. Tolsma, 2010

This research [30] was conducted as a Master’s thesis at the Delft University of Technology.

**Description of the research**

In the introduction of the thesis by Tolsma it is explained that there has never been any research into prefabricated concrete cores in buildings. This research is therefore conducted to check whether this is a feasible option. As most researchers do, first a literature study was done by Tolsma. Although there was no previous research into prefabricated concrete cores, information about prefabricated concrete connections in general, as well as the structural mechanics of a cantilevered tube structure was researched.

For the main part of this study, Tolsma designed 3 different corner connections. The corner connections will have a great influence on how much the flanges and the webs of a tube structure work together. The factor describing this connection is called $K$. All these connections can be seen in a figure in this section. The first connection is an interlocking connection, with the interlocking point halfway up the element. The name of this design in this thesis is the Interlocking Halfway Connection (IHC).

The second corner connection design is very similar to the first one, except for the fact that the interlocking
Figure 93: The Interlocking Halfway Connection (IHC), the Interlocking Above Ceiling Connection (IACC) and the Staggered Connection (SC) as designed by Tolsma (Courtesy of [30])

point is at a higher point in the element. The advantage of this design is that the joint can be hidden in the ceiling of a floor. This connection is called: Interlocking Above Ceiling Connection (IACC). Connections like this have been used in the Strijkijzer in the Hague which will be discussed in a later section.

The third and final connection that is analyzed in the research is the Staggered Connection (SC). This connection is created by changing each level which wall will form the corner between two sides of the core. This system has been used for corner connections in the Maastoren, which can also be found in a later section of the report.

All of the corner connection designs were first put into the Finite Element Method ATHENA in a 2 dimensional calculation. The reason why firstly a 2 dimensional calculation was performed is based on the processing speed of computers at the time of making this research. Creating a 40 story 3D model with a small mesh sizes around each connection to simulate its behavior correctly is simply not possible. Instead, the stiffness results of the 2D-model were imported into the 3 dimensional model as a smeared stiffness between the perpendicular core walls. In this way the stiffness of the corner connections on the overall stability of a complete core could be calculated.

Conclusion of the research

The results for the different connections under an increasing load are plotted in the graph in a figure this section. In this graph multiple values can be found. One important value is the $F_r$, the load at which the concrete ruptures. Before this value is reached, the loading in the shear key is only vertical. After rupturing of the concrete the shear key will rotate and the horizontal reinforcement of the shear key will be used to take up the horizontal forces introduced by the rotation. Before the concrete ruptures the stiffness behavior of the connection is linear and the slope is equal to the important factor $K$. As can be seen in the graph, the slope of the different connections is almost the same for all connections. The value of $F_r$ however, is not similar for all connections. Especially the IACC has a low $F_r$. Because in practice the value of $F_r$ should never be reached, this connection is the worst one of the three designed.

For the global 3 dimensional model, the value of $K_{\text{discrete}}$ from the 2 dimensional model has to be converted into a value of $K_{\text{smeared}}$ in the global model. To convert this value it has to be divided by the connection height.

Figure 94: The behavior of the three corner connections under increasing load, designed by Tolsma (Courtesy of [30])
and the wall thickness. Because the height of the connection of the SC is twice as high as the height of the IHC and IACC connections the $K_{\text{smear}}$ of the SC is the lowest of the three.

The overall results for the different corner connections are displayed in the second graph of this section. The three lines in this graph do not represent the three different connections however. Because of the low value of $F_r$ for the IACC the shear force in the global model exceeded this value. Therefore rupture in the concrete would appear, which is unacceptable in a cyclic loaded structure as a stabilizing core. The third line in the graph represents a cast-in-place core calculated in ATHENA, to compare the results of the prefabricated cores to. As can be seen in the graph, the differences between the two different connections, as well as the differences between the prefabricated concrete cores and the cast-in-place one are small. The core with a IHC connection just has a increase in top deflection of 3.3%. For the core using the SC this value is only 5.9%. From these values it can be concluded that prefabricated concrete cores are a viable alternative to be used in high-rise construction. Also it can be concluded that the behavior of the IHC is stiffer than the behavior of the SC, although the differences are not very high.

**Relevance to this thesis**

The information from this research can be used in the later stages of this thesis. There are two main points by Tolsma that can be used going forward. The first one is the possibility of designing a prefabricated concrete core in a high-rise structure. If at some point a tube-in-tube structure requires a core which takes up part of the lateral forces it would make sense to create this core in prefabricated concrete, just as the rest of the structure. The second contribution of Tolsma is the description of the behavior of different corner connections in prefabricated concrete. Although this research was done with primarily concrete cores in mind, the research done on corner connections can also be used in framed tubes in prefabricated concrete. Having some good data on these kind of corner connections could than prove to be valuable while creating a model.

**A.2.8 Application of Higher Strength Concrete in Tubular Structures, H. Balbaid, 2011**

This research [7] was conducted as a Master’s thesis at the Delft University of Technology.

**Description of the research**

Balbaid gives himself a few objectives at the start of his thesis. He wants to research the possibility of the use of of HSC and UHSC in tubular structures, sum up the advantages and disadvantages of using these higher strength concrete types, compare structures their OC counterparts also from a financial point of view and research the possibility of extra floors while maintaining the same geometry as in an OC construction. To meet these objectives, Balbaid took the following steps.

In the literature study of his report Balbaid describes the framed tube structure and its behaviors like shear lag and the combination of bending and shear deflection. Also some information on the differences between Ordinary Concrete (OC), High Strength Concrete (HSC) and Ultra High Strength Concrete (UHSC) are described. These differences lie mostly in the different material properties as modulus of elasticity.

Hereafter the design of the building which will be used in a parameter study is described. A typical floor plan is 14.4m wide and has a lenght of 36m. Each floor has a height of 3.6m and the amount of floors is changed.
throughout the research as one of the key parameters. The floors are made of hollow core slabs which span from façade to façade. These slabs have a height of 320\text{mm} or 400\text{mm} depending on the loads and have a fire proofing of at least 120 minutes. The façade elements all have a thickness of 350\text{mm}.

The 3 dimensional models are created in a Finite Element Method (FEM) program called Nemetschek SCIA Engineer. Two different geometries for the façade elements were created. One has a column width of 1600\text{mm}, the other one of column width of only 800\text{mm}. The geometry with the wider columns obviously has a higher stiffness and higher structures are possible using these dimensions. For each of the three concrete types (OC, HSC and UHSC) around three models were made, each with a different amount of floors. For these models the top deflections were checked and compared to the maximum allowable deflections.

In the following step all the elements of the structure are checked whether they can withstand the forces and moments due to the wind loading. In spots where the elements cannot withstand the forces, or are have a lot of leftover capacity, changes can be made to the structure increase the efficiency. This has been done in the optimization section in of the thesis. The figure in this section shows one of the proposed optimization designs. It uses Fiber Reinforced High Strength Concrete (FRHSC) in the bottom 12 levels and ordinary concrete in the remaining 18 levels. In this way, the stronger and stiffer concrete is used only in the part of the building where it is needed most. This is only one of the proposals. Others include reducing the beam height on the higher

![Figure 96: One of the optimization designs proposed in Balbaid (Courtesy of [7])](image)

floors and using steel profiles in the highly loaded corner columns.

Lastly a financial calculation has been made to compare an OC structure with structures in HSC and UHSC. The calculation is mainly focused on three aspects: The first one is the higher prize of the stronger concrete mixes, the second one is the low percentage of the concrete costs compared to the total building costs and lastly the ability to construct more floor in higher strength concretes.

**Conclusion of the research**

The objectives of Balbaid were described in the previous section. In this section the conclusions of this thesis will be described.

Firstly, the possibility of application of HSC and UHSC in tubular structures is discussed, in combination with the possibility of achieving a greater building height with HSC and UHSC. In the table the results from the FEM models is displayed. From these results it was concluded that constructing tubular structures in higher strength concretes is possible. Moreover, using higher strength concretes allows for taller structures with the same element sizes. For the geometry with the larger column width the increase in amount of floors possible is 5 for both the step between OC and HSC and the step between HSC and UHSC. For the geometry with the thinner columns this is also 5 floors per step, but the overall possible height is 10 floors lower in for each concrete class.

Most of the advantages described by Balbaid of using higher strength concrete in these kind of structures are pretty straight forward. Less lateral displacement, less required reinforcement and a more slender design are all to be expected using a material with higher material properties. The only advantage which stands out is the reduction of the shear lag effect for higher strength concretes. Unfortunately, this effect is very small (8% in the best case) and can probably be neglected. The disadvantages of using higher strength concretes are according to Balbaid the higher costs, the increased chance of instability when constructing slender elements and the little gain that can be found in elements loaded primarily by bending moments.
The conclusion of the financial comparison with an OC structure based on the aspects mentioned in the previous section is that constructing a building in HSC can actually reduce the costs per floor because the concrete costs are only a small part of the total budget and more floors are possible in HSC. For UHSC however, this effect is not demonstrated. Although even more floors are possible in UHSC, the increased price (12 times as expensive as OC) of the concrete cannot make using UHSC throughout the building profitable.

But arguably the most important conclusion is one that is not based on one of the objectives set in the start of the thesis. During the section about optimization a lot of tweaks were made to an existing structure to get the most out of the higher strength materials. This is not how the design of a structure in HSC or UHSC should be. A design in either one of these materials should be designed as a building in this material from the start to get the most out of these higher material properties.

Relevance to this thesis

The research of Balbaid is not the most important one discussed in this section. It however does prove that HSC and UHSC can be used in framed tube structures, and even could be financially efficient to do so (for HSC). Unfortunately in this report the facades are all constructed using cast-in-place construction, making it less relevant to this thesis. The last conclusion mentioned in previous section is probably the most useful one: If you want to make a structure in HSC or UHSC, design a structure that fully makes use of the better material properties.

A.2.9 Een bouwmethode voor geprefabriceerde betonnen hoogbouw (> 200m), M. van der Meij, 2012

This research [32] was conducted as a Master’s thesis at the Delft University of Technology. The Dutch title translates to A Building Method for Construction in Prefabricated Concrete (> 200m).

Description of the research

The research by van der Meij consists of three parts. The first one being a literature research that analyzes the then current situation of the building process for high-rise structures and the one in prefabricated concrete in particular. The boundary conditions of different vertical transport systems are also described in this study.

The second part of the thesis consist of a so called theoretical framework. Within this section the distinctive aspects of building structures over 200m in prefabricated concrete are described. Another part of this section concerns itself with describing the quantitative relations between system properties of the building process in direct relation to the height of the structure.

The first of these system properties described within the theoretical framework is the vertical transport. Logically, the amount of time it takes to lift elements to the floor at which they are installed increases when a building gets higher. Van der Meij describes that this does not influence the cycle time for building below 100m. For taller structures however, especially above 200m, the cycle time will be increased by the vertical
transport time of the elements. Therefore, the efficiency of the vertical transport becomes more important in
taller structures. An analysis was made for three prefabricated concrete structures to determine what optimization
of the vertical transport efficiency could do. These structures were the Erasmus MC tower in Rotterdam,
Het Strijkijzer in the Hague and the Millennium Tower in Amsterdam. This analysis proved that increasing the
efficiency could reduce the total vertical transport time by 50%, although this could be limited by other factors
such as structurally optimized element sizes and maximum transport sizes for transport to the building site.
Another aspect which is described is the possibility to eliminate the influence of the vertical transport time
on the cycle time by splitting the vertical and horizontal transportation. Tower cranes are an example of a
non-split system. Hoisting sheds are capable of performing vertical and horizontal transport simultaneously
and can be considered a split system.
Another important system property with a direct relation to the height of the structure is the wind. A param-eter study has been done to describe the effect of the wind on elements hoisted by a tower crane. The parameters
used were the length of the cable, the ratio between the element area and its weight and the ratio between an
elements length and its weight. The conclusions of this study can be found in the next section.
Another part of the wind investigation was the effect of larger heights on the amount of delay days caused by
difficult wind conditions. Because the wind speeds increase at higher altitudes, the amount of delay is expected
to increase. Using some intricate methods to combine the weather models provided by the KNMI (Royal Nether-
lands Meteorological Institute) and the Dutch regulations on wind stoppage an analysis was made to determine
the expected number of delay days for constructions higher than 200m. The third and final part of this thesis
is a case study, comparing two different vertical transportation systems with each other. The building on which
this case study is performed is based on the Erasmus MC tower in Rotterdam. Because this structure is only
120m high, a fictitious building is used that has a height of 200m, but has identical dimensions as the Erasmus
MC tower for width, depth and floor height.
The two different transport systems are:

- A hoisting shed with vertical transport guidance
- Two tower cranes

The use of two tower cranes in the second system is explained by arguing that the hoisting shed also has two
 cranes, one for vertical and one for horizontal transport. The case study focuses mostly on two aspects. The
first one is the influence on the construction time of the building. The information found in the theoretical
framework part of the thesis is used intensively. The second aspect of this case study is a comparison in costs
between the two different transportation systems. The results will be described in the next section.

Conclusion of the research
The theoretical framework of the research by van der Meij resulted in a few conclusions. For instance, the
parameter research into to influence of the wind resulted in the following:

- If the ratio between the weight and the area of the element increases, the deflection is reduced
- If the ratio between the length and the weight of the element increases, the chance an element will rotate
  increases
- The deflection of elements increases linear with the cable length, the rotation is not influenced by the
cable length.

Another conclusion in regard to the wind, is that due to the increased wind speeds at higher altitudes the delay
days for a 200m building are a factor 2.0 higher than for standard, medium rise structures. The arguably most important conclusion of the theoretical framework is that vertical transport for structures above 200m can influence the cycle time. To cope with this two options are available, either increasing the
efficiency of the vertical transport or, if all else fails, plan with a longer cycle time.
The main conclusion of the case study is that the higher initiation costs of a hoisting shed (around 750.000
euros) compared to two tower cranes can be compensated by the following aspects:

- The robustness of this building method is higher
- The erection of the concrete carcass is 89 days shorter
- The start of the finishing work can begin 32 workable days earlier
- The building time is 9 weeks shorter, reducing the site costs with around 350.000 euros

The main conclusion of the whole thesis though is another one. The choice for a transport system is important
during all phases of the building process, including initiation, design phase and realization. To make an informed
choice it is important to have enough knowledge about the different systems to understand the possibilities and
limitations of the transport systems.

**Relevance to this thesis**
The report of van der Meij focuses on the transport systems of prefabricated concrete high-rise structures. The design of such a system does not belong to the scope of this thesis. However, knowledge about the different systems and their possibilities and limitations is, as can be read in the main conclusion, important in all phases of the building process. Because a high-rise design will be made in this thesis, this information about transport systems can be used to make certain design choices.

### A.2.10 The Zalmhaven Tower, S.ten Hagen, 2012

This research [29] was conducted as a Master’s thesis at the Delft University of Technology.

**Description of the research**
The main research question ten Hagen asks in his thesis is whether it is structurally and logistically feasible to construct a 202.5m prefabricated concrete high-rise building in Rotterdam. To answer this question, ten Hagen alters a design of a planned high-rise in Rotterdam, the Zalmhaven Tower. His research starts, as do almost all theses, with a literature study. In this study he focuses on five different aspects with regard to high-rise in prefabricated concrete.

The first aspect is the wind loading, which is a major factor in constructing any high-rise. Furthermore, a building of more than 200m has never been constructed in the Netherlands and no regulations for these kind of heights exist in the old Dutch codes. The Eurocode does provide regulations that can be used. The second aspect looked into is the connections between the elements. Connections have a large influence on the overall behavior of the building and plays an important role in the construction process as well. The element configuration is the third aspect focused on in the literature report. As has been described in previous sections, the choice for a masonry configuration has a lot of influence on the entire structural concept. Material properties and especially the use of HSC and UHSC are described in the fourth part of the study. The conclusion of Balbaid was found to be as described in the previous section, where only HSC can be financially feasible to use in high-rise structures because of the better material properties for a fairly reasonable prize. The fifth and final aspect focused on in the literature report is progressive collapse. The structural reaction to unforeseen forces is described and a look into the risk analysis of the original Zalmhaven Tower is looked into. Ten Hagen writes about the system consequence classes in relation to high-rise structures and compares in situ structures to prefabricated ones. After this study the direction of the thesis is towards a structural design of the Zalmhaven Tower in prefabricated concrete elements. Several steps are taken to work towards this goal. The first one is the description of the designed Zalmhaven Tower using cast-in-place concrete. In the second step different structural concepts and element configurations that are possible in prefabricated concrete are designed and a choice is made for one that has the most potential without further investigation. A detailed description of different hoisting sheds, their advantages and disadvantages and their influence on different concepts is part of this decision. The next sections

![Figure 98: Structural layout of a floor of the Zalmhaven tower (Courtesy of [29])](image)

are about the loads on the structure and the selection of a fitting Finite Element Method (FEM) program for analysis of prefabricated concrete high-rise structures. The two sections after that are about the modeling of
the structure in AXISVM (the FEM-program) and the influence of the horizontal and vertical connections on both the structural behavior and the construction methods of the building.

One of the longer and more important sections is about the results of the FEM analysis. In this section the results of the analysis are given, such as the distribution of forces, the deformations and effects due to second order influences and shear lag. Using these results the reinforcement of some key elements is designed and the

Figure 99: Deflection results of the FEM analysis (Courtesy of [29])

Young’s modulus is verified.

The two final sections focus on the construction of the building. One section is about the dimensional control in prefabricated concrete high-rise structures like these. What kind of accuracy is required for the elements sizes and which tolerances should be factored into the design. The other section is about the realization of the project and focuses on the vertical and horizontal cycle time using a hoisting shed as described in an earlier section. With this information a total construction time is given which is compared with an estimated construction time in cast-in-place concrete.

Conclusion of the research

The main conclusion ten Hagen finds is that a creating a prefabricated concrete high-rise structure of 200m is structurally and logistically feasible in the Netherlands. Some of the conclusions with regard to structural behavior are listed below:

- If large elements are used in a masonry configuration, the stiffness is only marginally smaller (4%) than a cast in situ variant
- The horizontal connections behave as stiff as the surrounding concrete due to the high normal forces.
- The shear stiffness of the horizontal connections and the stiffness of the vertical connections in a masonry configuration both do not influence the behavior of the structure.
- The distribution of shear forces is vastly different between a cast-in-place wall and one with a masonry configuration

Conclusions that are of a non-structural nature can be divided into two groups. The first one is about the construction phase and the influence of large heights on cycle times. Also the advantages of a hoisting shed are described. The second group of conclusions sums up the advantages of building in prefabricated concrete in comparison to cast-in-place construction. Factory based production, less labor on site and a reduced construction time are all mentioned.

Relevance to this thesis

The relevance of ten Hagen’s research to this thesis is very clear. Ten Hagen designs a structure of over 200m in prefabricated concrete, which is also one of the goals of this thesis. Although a different stability system is used, using shear walls instead of the façade, similarities between the two structures can be expected. Some of the
conclusions and recommendations can surely be used and difficulties that ten Hagen ran into can be avoided.
The research question of ten Hagen consists of two parts. The structural and the logistical feasibility of a prefabricated high-rise structure of 200m were described. Because the focus of this thesis weights more heavily on the structural part, the logistical aspects and conclusions are less important. The conclusions with regard to structural behavior are of more use. The ones about connection stiffness of both vertical and horizontal joints and their influence on the total behavior of the structure give a lot of insight in prefabricated concrete stability systems in general.

A.2.11 Towards optimal design of high-rise building tube systems, M. Shin et al., 2010

This research [25] was conducted by M. Shin (Ulsan National Institute of Science and Technology, Korea), T.H.K. Kang (University of Oklahoma, USA) and B. Pimentel (Rosenwasser/Grossman Consulting Engineers, New York, USA) and was published online in Wiley Online Library in 2010.

Description of the research

The goal of this research is to get a sense of what parameters in a concrete frame wall tube building influence its behavior most. In the introduction previous research is described and the phenomenon shear lag explained. A distinction can be made between positive and negative shear lag. Positive shear lag is shear lag that results in higher forces in the columns near the corners of the building than is to be expected from the mechanics of a clamped tube, negative shear lag has the opposite effect. The introduction concludes by stating what the writers thought lacked in earlier research and why they made this publication

In the second section the building is described on which the parameter study will take place. It is a 55 floor hotel building which is planned in New York City. The height of the structure is approximately 174m and the floor plan is around 33m in the east-west direction and 17.7m or 15.8m in the north-south direction for the lower levels and upper levels respectively. To accommodate the change in floor plan a full story belt wall was designed at mid-height in the structure. A mechanical floor was created at this level because due to all the structural elements this floor could not function as a hotel.

The stability of the structure consists of a tube structure. The sides of the tube consist of shear walls in the north-south direction and of a column and beam structure in the east-west direction. Some structural cores are incorporated in the structure as well. The entire structure is made of reinforced concrete. A 3D-model and typical floor plans for the lower and upper levels are in the figure in this section. The design variables used in the parametric study are the following: The depth of the column ($h_c$), the depth of the beam ($h_b$), the column width ($b_c$) and the beam width ($b_b$) of the members on the north and south façade. Changing these parameters several properties of these elements change, such as the axial ($EA$), flexural ($EI$) and shear ($GA$) rigidity. These changes lead to differences in behavior which can than be examined.

Conclusion of the research

From the parameter study on the case study building a lot of different graphs were made and data collected. From this data a lot of conclusions can be found. Two of the graphs are shown in an image in this section. From these two graphs the influence of the column depth and the beam depth on the axial forces in the columns in south façade on the 30rd floor can be seen if the structure is loaded by a wind force in the north-south direction. In both graphs the influence of the positive shear lag, with higher axial forces near the corners of the building, can be seen. For the column depth the relation is simply that a column with a higher depth attracts
a higher force. This also holds for the middle columns by an increasing beam depth. For the outside columns however, a deeper beam results in lower axial forces. The effect of the shear lag can be considered diminished for an increased beam depth.

These kind of graphs and found relations are made for all kind of internal forces and moments, internal stresses and overall deflections. The most important conclusions found are listed below:

- The overall deflection of a building similar to the case study building can be effectively reduced by increasing the stiffness of the flange members. Increasing column depth and the column width are most effective ways to do this.

- The contribution of the flange frame in resisting the overturning moment can be enhanced in the best way by increasing the column depth. The second best way to do this depends whether it is at a point in the building above or below the belt structure, in which case increasing the beam depth or increasing the column width is the best option respectively.

- The contribution of the flange members in a tube structure would be relatively small in a low rise buildings because in these kind of buildings the shear resistance is governing.

- The shear lag in the web shear walls is in a system similar to the case study building small.

- The degree of shear lag is approximately linearly proportional to variation in column depth, but non-linearly to the beam depth. An increase in beam depth is the best way to diminish the shear lag effects.
Increasing the column width works adversely for reducing shear lag.

The overall conclusion is that the tube action of a high-rise framed tube structure can best be improved by increasing the column depth and shear lag reduced by increasing the beam depth. Increasing the column width is recommended only for reducing the overall deflections of a building. Lastly, the beam width has the least influence on the lateral force resistance of the four examined parameters.

Relevance to this thesis
This publication has several aspects that are interesting for the further writing of this thesis. The conclusions with respect to the increasing of element parameters and their effects are really helpful in designing a framed tube structure. Another aspect that was learned from this publication is the existence of negative shear lag in the highest floors of framed tube structures. Although this is not likely to be governing for any building, knowing of its existence will help interpret results of FEM-analysis better. The last aspect that helps further research is the design of a belt wall structure to cope with the change of floor plans at mid-height of the building. It proves that even relatively small changes (0.9m on each side of the floor plan) can have serious consequences for the main load bearing structure and the behavior of the structure as a whole.

A.2.12 Work of Kung-Kun Lee et al., 2001/2002
This section is about two works by Kung-Kun Lee and his co-writers. Because the two publications have a lot of similarities and were both published within two years their contents will be discussed in one section. The two publications are:

- Simple Analysis of Framed-tube Structures with Multiple Internal Tubes, Kang-Kun Lee (Hanyang University, Seoul, Korea), Yew-Chaye Loo and Hong Guan (both Griffith University, Gold Coast Campus, Queensland, Australia)
  Journal of Structural Engineering No.4, April, 2001 [18]

- Prediction of Shear-lag effects in Framed-tube Structures with Internal Tubes, Kang-Kun Lee, Li-Hyung Lee and Eun-Jin Lee (all Hanyang University, Seoul, Korea)
  Structural Design of Tall Buildings 11, 2002 [17]

Description of the research
The start of these publications is dedicated to explaining why the authors deemed the research they did was necessary. They describe early studies done on shear lag in general, research focused on (positive and negative) shear lag in steel box girder and research on framed tube structures by Coull and Ahmed (1978) and Kwan (1994). Those last two methods for determining the shear lag lack two important aspects. First of all, the negative shear lag is not taken into account. Secondly, there is no possibility to determine the effect of one or more internal cores on the shear lag of the total structure.

Before explaining their new method of solving these problems, shear lag in tube(s) in tube structures is explained. From primary bending theory one expects that in a bending tube the stresses in the flanges are uniform and the stresses in the webs are linear. This is not the case in framed tube structures. Either the forces in the corner columns are higher than what could be expected by the linear gradient of the stress distribution, called positive shear lag, or the forces in the corner columns is lower, called negative shear lag. At the same time, this influences the forces in the columns near the center of the webs, positive shear lag reducing these forces and negative shear forces increasing them. The linear gradient of the stress distribution in the webs, along with the uniform stress distribution in the flanges is lost if shear lag is present. The method by Lee and his co-authors is based on the minimum potential energy principle in combination with the variational approach. Lee [18] further explains his method as: "The structures are modeled as a box assemblage of the orthotropic plate panels, each composed of horizontal beams and vertical columns. In the proposed method, Reissner’s function is modified to account for the independent distribution of the vertical displacement in the flange frame panels of each tub, thereby taking the net shear lag into consideration. By simplifying the assumptions in relation to the patterns of the vertical displacement distribution in the tubes, the complex 3D structural analysis is reduced to the mere solution of a single second-order linear differential equation". Using this approach, the total deflection and the stress distributions of flange and web panels of external and internal tubes can be calculated.

As a test case, the method is used on 3 buildings with a similar floor plan and a different amount of floors (30, 50 and 70). For these structures a calculation also has been made with a 3D frame analysis program (ETABS) for the correct answer and with two other approximation methods from earlier research (Coull & Ahmed, Kwan) to compare this new method to. The results of this test case can be found in the next section.

In the publication Prediction of Shear-lag effects in Framed-tube Structures with Internal Tubes [17] Lee and his co-authors try to find which stiffness parameter in a framed tube structure has the largest influence on the shear lag. Three stiffness parameters are investigated:

1. Axial Stiffness of the columns
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2. Bending stiffness of the beams

3. Bending stiffness of the columns

Each of these parameters has been separately multiplied by a factor 0.5 and a factor 1.5 to measure their influence. This was performed on three different structures. A structure without internal cores, a structure with one internal core and a structure with two internal cores. In total 21 structures have been calculated in a 3D frame analysis program. The results of this analysis were then translated into a factor $p$, called the shear lag factor. This factor is the ratio between the axial force in the corner column and the axial force in the center column of the exterior flange. The conclusions can be found in the next section.

**Conclusion of the research**

The results of the comparison between the 3D frame model and the three approximation methods can be seen in the table in this section. The comparison is made based on the highest axial force in the structure. The proposed model is the model proposed by Lee and his co-authors. In the table it can be seen that the error of the method by Lee is significantly smaller than the methods by Coull & Ahmed and Kwan. For higher structures the error decreases, leading to an error of only 1% for the structure of 70 floors. More results from this comparison can be found in the three graphs in this section. In these graphs, the previously explained shear lag factor $p$ is plotted over the height of the structure for the same four methods. The closer the results of the method are to the results calculated by the 3D frame analysis, the better its results are. Again, the results by Lee are closest to these values. Furthermore, the accuracy increases for higher structures, just as in the results in the table.

Another aspect of shear lag can be seen in the graphs for the three structures. Positive shear lag mainly occurs...
in the lower parts of a framed tube structure. Above a quarter of the height of a framed tube the shear lag factor \( p \) becomes negative, indicating a negative shear lag. Using these results in the table and the graphs it is concluded that the approximation method by Lee is one that is accurate enough to use in early design stages. Because the method takes up way less time than a 3D frame analysis, the method could be well used in practice.

The results of the parameter analysis to determine which stiffness has the most influence on the shear lag effect did also lead to some conclusions. From the analysis it turned out that influencing the axial stiffness of the columns has the most effective in reducing the shear lag in a framed tube structure. Changing this parameter increased the so called stiffness parameter \( S_f \). Which can be determined with:

\[
S_f = \frac{12h}{A_c d^2} \left( \frac{h}{I_c} + \frac{d}{I_b} \right)^{-1}
\]

In this equation \( h \) is the structural height, \( d \) the span between the columns, \( A_c \) the cross-sectional area of the column and \( I_c \) and \( I_b \) the second moment of area for the column and the beam respectively. Increasing this factor \( S_f \) reduced the shear lag factor \( p \). And although increasing \( I_c \) and \( I_b \) also has an influence on the value of \( S_f \), the reduction of \( A_c \) is has more influence.

From this it can be concluded that decreasing the axial stiffness of the columns has the most influence on the reduction of the shear lag effect.

Relevance to this thesis

For this thesis, the work of Lee is very relevant. A lot of insight on the shear lag principle is obtained. Especially the focus on the positive and negative shear lag over the height of a structure was very insightful. In contrast to a lot of other literature which mention shear lag only for a pure framed tube structure, Lee includes the possibility of one or more internal cores in the calculation.

Although the best way to get accurate results is still to make a 3 dimensional model of the structure in a Finite Element Method (FEM) program, this approximation method can be a quick way to find some answers about the behavior of a framed tube or tube-in-tube structure without a lot of modeling in a FEM-program.

The work of Lee is about a monolith concrete structure. These conclusions may not hold for prefabricated concrete structures and should therefore be used with care when translating them into this thesis.


This research [26] was conducted by Y.Singh and A.K.Nagpal, both affiliated with the Indian Institute of Technology, Hauz Khas, New Delhi. The research was published in the Journal of Structural Engineering in 1994.

Description of the research

Exploring negative shear lag further, this research was referenced by the previous three articles and explains negative shear lag in buildings. Singh and Nagpal do so by first explaining why they decided to write the article. It is explained that although the negative shear lag effect has been noticed in framed-tube structure research for around 10 years, “an explanation of it and comprehensive research are lacking”.

The article is split up in three parts. The first part explains the shear lag effect and in particular the negative shear lag using some figures that also part of this report. In the second part, an analogy between box-girders
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(which also experience shear lag effects) and framed-tube structures is made to justify using some previous research on box-girders in this context. The last part is a parameter study which studies 4 nondimensional parameters and their effect on the shear lag effect. These parameters are:

- Stiffness factor $$S_f$$ ($$S_f = 12EI_bH_s/EA_cW_b^3$$)
- Stiffness ratio $$S_r$$ ($$S_r = EI_cW_b/EI_bH_s$$)
- Ratio number of stories to the number of bays in a half-flange $$p$$
- Aspect ratio $$A_r$$ (ratio between the flange and web dimensions)

In which $$E$$=Young’s Modulus, $$I_b$$ and $$I_c$$ the moments of inertia of the beam and column respectively, $$H_s$$ the story height, $$A_c$$ the column cross-section and $$W_b$$ the span of the beam.

**Conclusion of the research**

An important part of this research is the explanation of the negative shear lag effect, because a clear explanation was not available in literature at the time. For description of the behavior above a certain floor level, called the $$j^{th}$$ level, the system is split up into two systems. One contributing to the positive shear lag effect and one contributing to the negative shear lag effect. The first subsystem consists of all the floor above the $$j^{th}$$ level, loaded by the loads that are working on this part of the building. At the nodes of the $$j^{th}$$ level, the structure is assumed to be fixed. The second subsystem also consists of the structure above the $$j^{th}$$ level, but no external loading is considered. Only the translations and rotations of the $$j^{th}$$ level are put in the model.

First the contribution of the first subsystem to the positive shear lag is explained. Because the system is now

![Figure 106: A model (consisting of one flange and half a web) to determine the behavior of a framed-tube structure, using two subsystems. (a) depicts the full system, (b) the first sub system and (c) the second subsystem (Courtesy of [26])](image)

a framed-tube structure (of the stories above the $$j^{th}$$ level) under lateral loading the well known explanation can be given here. Because the elastic continuity of the webs is not as good as in a non-perforated shear walls, the expected linear axial stress distribution of the webs is not found. Instead, a non-linear distribution is found with higher stresses near the corners and smaller stresses near the middle of the web. This then leads to a non-uniform axial stress distribution in the flange façades.

The second subsystem, contributing to the negative shear lag, is a new effect that is described by the research. All the nodes a the $$j^{th}$$ level are given the lateral translations and rotations present in the structure at that point. Because the floor slabs can be considered very stiff in their plane, all joints at the same level undergo the same lateral translations. Between the vertical translations and the rotations, the vertical translations can be considered to be predominant. Now the axial forces as a reaction to the vertical translation profile of the $$j^{th}$$ level can be found. Because the vertical translation profile is concave upward because of the positive shear lag effect, the reaction forces are convex upward. This axial force distribution is the contribution of the negative shear lag.

It is important to note that negative shear lag is a reaction to the positive shear lag. If positive shear lag would not be present, a uniform vertical translation distribution would be present in the flange façade. The reaction of the structure to such a distribution would not be a convex upward axial load distribution.
After the description of the workings of positive and negative shear lag, it is important to investigate whether both are in the same order of magnitude. By doing this, it can be prevented that engineers spend time on an effect which should be ignored. Especially for negative shear lag it is interesting whether this is the case. A study has been done to check the magnitude of the shear lag effects over the non-dimensional height $x/H$ of a framed-tube structure. To do this, two parameters were introduced. The first parameter, $(\Delta F)_1$, is the difference in force between the middle column and the corner column in the first subsystem and is an accurate indicator for the magnitude of the positive shear lag. A similar parameter $(\Delta F)_2$ is defined in the second subsystem and can be considered an indicator of the negative shear lag.

In a graph in this section, the value for $(\Delta F)_1$ and $(\Delta F)_2$ is displayed over the non-dimensional height $x/H$. The value of the positive shear lag indicator, $(\Delta F)_1$, decreases linearly with the height of the structure, which can be explained by the height of the loaded structure above a certain level. The value of $(\Delta F)_2$ is a non-linear relationship. It increases for an increasing height to the point where $(\Delta F)_1$ and $(\Delta F)_2$ have the same value, but hereafter it decrease for an increasing height. This height, where the share of the positive and negative shear lag is the same is called the shear lag reversal level and is depicted by $\eta$. Because the negative shear lag is a reaction effect of the positive shear lag, it is comprehensible that it reaches its largest value just before it becomes the predominant shear lag effect itself. As can be seen in the figure, the level of shear lag reversal is

Figure 107: A model of the first subsystem (left) and the second subsystem (right). For both, the deflection is given (a) and the resulting axial force distribution (b) (Courtesy of [26])

Figure 108: The variation of $(\Delta F)_1$ and $(\Delta F)_2$ over the non-dimensional height of a framed-tube structure (Courtesy of [26])
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between one forth and one third of the building height. This means that negative shear lag is dominant in the largest part of the structure and a convex upward axial force distribution is present, because an addition of the subsystems yields the final force distribution.

The third part of the study was a parameter study into the effect of 4 non-dimensional parameters on the shear lag effect. The results were as followed:

- A reduction of the stiffness factor \( S_f \) (\( S_f = 12EI_bH_s/EA_cW_b^3 \)), results in an elevated positive and negative shear lag
- If the Stiffness ratio \( S_r \) (\( S_r = EI_cW_b/EI_bH_s \)) is smaller, the positive and negative shear lag is increased.
- The ratio between the number of stories to the number of bays in a half-flange \( p \), yields a higher shear lag effect (positive and negative) if it is smaller.
- Changing the aspect ratio \( A_r \) (ratio between the flange and web dimensions), does not have an effect on the shear lag which should be considered during calculation.

Additionally, for the first three parameters it was found that decreasing them shifted the level of shear lag reversal (\( \eta \)) up.

Relevance to this thesis

The relevance of this thesis lies in the comprehension of the shear lag effect in general and the comprehension of negative shear lag in particular. Because the shear lag effect will have a large influence on all framed-tube structures which will be used in the research stage as well as on the structure that is to be designed in the design stage. The conclusions found with respect to the force distribution and the shear lag distribution over the height of the structure are important as well, because they can be used to explain results from calculations with for instance Finite Element Method programs. Moreover, no research has been found which describes both negative shear lag and prefabricated concrete structures, so this might yield some interesting results.

A.2.14 Design and Displacements of Precast Concrete Shear Wall Structures, D.C. van Keulen, 2013

This research [36] was conducted by D.C. van Keulen and J.Vamberský, both affiliated with Delft University of Technology.

Description of the research

The goal of this research is to give an indication which parameters for a shear wall in prefabricated concrete reduce the strength and stiffness the most. Reduction of these properties could lead to failure or high deflections which have to be prevented in a stability structure. To gain an answer to this question, a large number of shear walls have been analyzed in a Finite Element Method (FEM) program, each time changing one of the parameters. The results are presented as graphs in which the deformation increase with respect to a cast-in-place structure in percentages is plotted against the slenderness ratio (height/width) of the shear wall. The following aspects have been researched:

- Stiffness of the horizontal joint
- Element lay-out
- Element size and lay-out
- Element openings and lay-out

The results of these analyses will be presented in the next section.

Conclusion of the research

The first aspects van Keulen analyzed was the influence of the stiffness of the horizontal joint in shear walls with a masonry configuration with open vertical joint. All horizontal joints consist of bars in grouted sleeves and are filled with mortar between elements. The slenderness of the shear wall has been varied between 10 and 0.33. The results of the analysis lead to the graph shown in the image. Four different lines can be distinguished for four different horizontal stiffnesses, varying from minimally reinforced with a low mortar quality to heavily reinforced and relatively high stresses. The analysis yields that there is an effect on the behavior of the shear wall when changes to the stiffness of the horizontal joints are made. Unsurprisingly, the stronger connection yields better results than the connection that is categorized as "very weak".

The shape of all lines in the graph is roughly the same and remains the same for all relationships found in this publication. The deformation increase in percentages, compared to a cast-in-place wall, is highest for shear walls with a low slenderness, and decreases to a value below 10% if the slenderness ratio exceeds a value of approximately 5. The high deformation increase for these walls with a low slenderness should not be a problem however, because absolute deformations of structures with a low slenderness are generally low.
A LITERATURE REPORT

Figure 109: The analysis results for horizontal joints with a different stiffness (Courtesy of [36])

The second and third analysis performed by van Keulen are about changing the size and configuration of the prefabricated concrete elements. The configuration variants that were researched include the standard horizontal masonry configuration and the horizontal regularly stacked elements, with continuing vertical joints. Both these configurations have also been analyzed with elements that do not span one floor height but two, changing their main direction into a vertical one. The size changes in the third analysis have variants with elements that are a half floor height or only half the length of the other elements in these analyses. In this analysis horizontal and vertical masonry configurations are researched as well.

The conclusions of these two analysis are the following:

• Lay-outs with larger elements perform better than lay-outs with smaller elements. This can be explained by the lower number of joints in walls with larger elements and the negative influence joints (horizontal and vertical) have on the overall stiffness
• Configurations with horizontal elements perform better than configurations with vertical elements.
• The results between different horizontal lay-outs and the results between different vertical lay-out are all relatively close to each other
• Whether a horizontal or vertical configuration is chosen has more influence on the stiffness of a shear wall than whether larger or smaller elements are used.

The final aspect looked into by van Keulen is the behavior of shear walls with openings in them. These kind of shear wall could be the walls of a framed tube system. Three different configurations and two different opening sizes lead to 6 different variations that are analyzed. As can be seen in the figure, similar curves as for all analyses are found for the relation between the wall slenderness and the deformation increase. The best configuration is lay-out IV, which has the smaller of the two opening sizes and the masonry type configuration. The worst performing lay-out, lay-out III, has the larger one of the two opening sizes and a connection through the lintel of the element. From the performance of these two lay-outs, and the ones that perform on a level in between these extremes the following conclusions were found:

• Larger openings lead to a lower stiffness and consequently to higher values of the deformation increase
• Putting the joint in the lintel of an element has a clear negative influence on the behavior of perforated shear walls
• Especially in the variants with the larger openings, the difference between a masonry configuration with open vertical joints and a regular configuration with a profiled structural joints is negligible.

A last conclusion by van Keulen is about the amount of deflection caused by shear deformation in relation to the total deformation for structures with a different slenderness ratio. He found that the share of shear
deflection on the total deflection is lower for structures with a high slenderness. This means that for high-rise structures, that usually have a high slenderness, the shear deformation is not that important. Because joints in prefabricated walls mostly influence the shear deflection and not the bending deflection it can be concluded that high-rise structures especially are very fit for a solution in prefabricated concrete.

Relevance to this thesis

All conclusions by van Keulen and Vamberský in this publication are relevant to this thesis. The subject of the publications is about prefabricated shear walls and shear walls with high slenderness ratios are a key part of the analyses. Furthermore, shear walls with openings are analyzed. These kind of walls are what a framed tube is made of and the conclusions for these walls are therefore critical for the remainder of this thesis. The last conclusion described in last section, about the ratio of the deformations caused by shear in relation to the total deformation, describes the possibilities of high-rise construction is prefabricated concrete and can be seen as a motivation for this research. This is also underlined by the results of all slender walls having a deformation increase compared to a cast-in-place wall of less than 10%.

A.2.15 EEM en prefab beton, D.C.van Keulen, 2014

This article [35] is published in Cement and written by D.C.van Keulen, affiliated with Delft University of Technology. The Dutch title translates to: FEM in prefabricated concrete.

Description of the research

In this research, van Keulen describes the important aspects of calculating prefabricated concrete shear structures in a Finite Element Method (FEM) program. As he points out, calculating and modeling the joints between the element is the most crucial part of this method. The research was conducted in order to get the correct equations for the normal and shear stiffness between elements for the values used in a particular FEM-program (AxisVM) and the unit used in this program (kN/m/m). The normal stiffness determination is rather straightforward and a equation for this is rather easily found. For the shear stiffness however, multiple different factors contribute to this shear stiffness:

- Adhesion between the materials
- Resistance due to the roughness of the interface
- Friction due to the normal stresses
- Friction due to extra normal stresses because of the extension of the reinforcement
- Dowel action of the reinforcement

An additional problem with these different factors is that they are active at different slip values of the joint, so that they cannot simply be added up to reach the stiffness of the connection. After van Keulen determines the equations for the stiffnesses needed, a case study on the Ziggo Dome in Amsterdam is given where the formulated equations were used in practice. He also warns that using a FEM-program usually costs more time than conventional methods and should only be used in cases where for instance...
complex shapes are present or optimization is important.

**Conclusion of the research**
The most important conclusions of this article are the equations for the normal and shear stiffness of the joints. For the normal stiffness the following results are found:

\[ k_{y,normal} = \frac{A \cdot E_{cd}}{d} \]

In which \( k_{y,normal} \) is the normal stiffness of the joint (\([kN/m/m]\)), \( A \) is the area of the joint per m (\([mm^2/m]\)), \( E_{cd} \) is the Young’s modulus of the concrete surrounding the joint (\([N/mm^2]\]) and \( d \) is the thickness of the joint (\([mm]\]).

For the shear stiffness of a joint first the shear resistance between the two concrete interfaces (\(v_{Rdi}\)) has to be determined. This can be done with an equation from the Eurocode:

\[ v_{Rdi} = c_f c_t d + \mu \sigma_n + \mu \rho f_{yd} \leq 0.5 \nu f_{cd} f_{\alpha} = 90^\circ \]

In which \( v_{Rdi} \) is the design shear resistance at the interface (\([N/mm^2]\)), \( \mu \) is a factor dependent on the smoothness of the interface [-], \( \sigma_n \) is the minimum normal stress at the interface (\([N/mm^2]\)), \( \rho \) is the fraction between the area of the reinforcement and the total area investigated [-], \( f_{yd} \) the yield of the reinforcement (\([N/mm^2]\]), \( \nu \) is the strength reduction factor for concrete in shear (\( \nu = 0.6(1 - \frac{f_{ck}}{250}) \)[-]), \( f_{ck} \) the characteristic compressive strength of the concrete (\([N/mm^2]\]), and \( f_{cd} \) the design compressive strength of the concrete (\([N/mm^2]\]).

Van Keulen describes that a connection has either strong adhesion or weak adhesion. For strong adhesion connections the \( \mu \rho f_{yd} \) part of \( v_{Rdi} \) is left out of the equation, for weak adhesion connections the \( c_f c_t d \) part is left out. The reason behind this exclusion of some parts of the equation is the earlier mentioned different connection slips in which the different factors occur. For the strong adhesion the slip is very low (around 0.05mm) and for the low adhesion slips of 0.5mm – 1.5mm are used.

To get from the shear resistance to the shear stiffness of the connection the following equation is used:

\[ k_{x,shear} = \frac{v_{Rdi}}{s} \cdot 10^3 \]

In which \( k_{x,shear} \) is the shear connection stiffness (\([kN/m/m]\)), \( v_{Rdi} \) is the shear resistance at the interface (\([N/mm^2]\)), \( t \) is the width of the connection (\([mm]\)) and \( s \) is the value of the slip of the connection.

**Relevance to this thesis**
The article by van Keulen is very relevant for this thesis because it gives a clear way of determining the stiffnesses between prefabricated concrete connections in shear structures. Furthermore, the fact that these calculations are already used in practice confirms that these equations are reliable and safe. The distinction between the strong and the weak adhesion connections and the slips belonging to those modes can be of great value in the rest of this thesis.
A.3 Relevant Buildings

In this section of the literature report a review of some existing buildings relevant to the thesis subject will be described. The importance of a review of existing structures lies in two aspects. The first one is putting the theory of prefabricated concrete framed tube structures into practice. This gives some insight into the behavior of real structures and the dimensions that are used in practice to get results that are within the building codes. Secondly it also proves that these kind of structures are financially possible. If in the real world prefabricated concrete structures can be competitive, research into these kind of buildings and therefore is more worthwhile.

A.3.1 Het Strijkijzer, Den Haag

At the time of its construction, the residential tower dubbed the Strijkijzer by the people of The Hague, was the highest prefabricated concrete structure in the Netherlands. Its 42 stories reach a height of around 132m and its construction was completed in 2007. Most of the information comes from [11]

![Het Strijkijzer in The Hague](https://www.haagsetoren-wonen.nl)

Figure 112: Het Strijkijzer in The Hague (Courtesy of www.haagsetoren-wonen.nl)

**Description of the building**

The main floor plan of the tower consist of two wings connected perpendicularly to form an L-shape. At the base of the tower, as well as between levels 26 through 32 the ends of these two wings are connected to form a triangular shape. All the corners of this triangle are rounded off, giving the building its distinct shape. Although the majority of its 30,000m² is used for residential purposes on the lower levels there is some space for offices and retail. A restaurant is featured on the highest level giving its customers a great view of the city.

**Stability system**

When the tower was designed it was designed as a regular cast-in-place concrete tower. This was a logical choice considering that the outside walls as well as the internal walls would feature in the stability system of the structure. The main contractor also wanted to work with a tunnel formwork system using relatively high concrete classes. These classes, for instance B65 for all vertical elements, would be used to accommodate an amount slenderness in the elements. Near the bottom of the structure steel profiles would have to be cast in to withstand the forces that would occur there.

To improve the building speed and to simplify the logistics, a choice was made to use prefabricated concrete
from the fourth floor upwards. A big factor in this choice was the very small building site they had to work on, with roads, railroads and tram lines all around it. To cope with the loss of stiffness from the horizontal and vertical joints in comparison to the cast-in-place designed structure an even higher concrete class was used in the bottom 4 levels of the structure, B85.

The main stability system can be described as a framed tube, in which the façade walls provide lateral stability. In this structure however, there was a focus on making use of every possible element for the overall stability of the building, giving the client a lot of efficiency in that sense. For this reason all the internal walls and other vertical elements in the building are either used to tie the façade elements together or to provide some lateral stability themselves. To prevent this action from creating a fixed floor plan for the entire life cycle of the building, large cutouts were made to facilitate the creation of coupled apartments if the market for the now used smaller units might become less profitable in the future.

Connections
The elements of the building are fairly large, weighing up to around 25 tonnes. Because the prefabricated method was chosen with building speed in mind, the connection between the elements should not take a lot of time to construct. This goal was achieved and two stories every week were constructed. The element layout and some connections are shown in the image below on the next page. As can be seen in the image, most connections between elements are interlocking connections, which transfer the shear force in an effective way without the need of an time consuming vertical joint over the whole height of the element. The cutouts in the internal walls, talked about in the last section are also visible in this picture, as is this the stability system as a whole.

Conclusions relevant for this thesis
A few important conclusions can be found in the investigation of this building. First of all this building is a classical example of the benefits of prefabricated high-rise building. The building was constructed in the middle of the city on a compact building site using traditional tower cranes. The building time was estimated to be 32 months using regular cast-in-place construction, but with the prefabricated building method this was reduced to 20 months. This shortening of the building period is an advantage for all stakeholders of the tower. For the client it is an advantage to be able to rent out the building a year earlier, making the investment in the slightly more expensive concrete elements worth it. The construction work itself is also less complex, creating a cleaner and better working environment for workers. For people living near the building site it an advantage to have less hindrance from the construction site in respect to noise, smell and possible other aspects.

A second conclusion we can find is that it is possible to use a framed tube stability system in an irregular shaped building. Although this building can not be referred to as a pure framed tube building, using its interiors walls
to couple the different façade elements. Then again, pure framed tube buildings are rare, more often the framed tube system is combined with bracing or a core to provide additional stability.

Thirdly it is interesting to see that there has been made a horizontal stiffness calculation for the regular cast-in-place variant and the prefabricated one, so that these values can be compared. For the prefabricated construction only a lower horizontal stiffness of between 5 and 10 percent was found. The small difference between these two values means that the difference in top deflection, often governing for high-rise structures, will also be relatively low. This makes the choice to construct these kind of buildings in prefabricated concrete better justifiable.

The fourth and last thing we can learn from "Het Strijkijzer" is that there are no difficult and expensive vertical connections needed between the elements to come close to the strength of a cast-in-place construction. Interlocking connections can do a good job to transfer the shear loads between the different elements. There is no need for any "wet" connections that require a lot of time, formwork, work and therefore a lot of money. For these kind of connections it is key though to make sure that tolerances are something that is taken care of in the design of the connection.
A.3.2 JuBi Towers, Den Haag

The JuBi towers are two towers in the The Hague city center that are used by the ministry of Security and Justice and the ministry of the Interior and Kingdom Relations. Both towers are 41 stories high and reach a height of 140m. Construction was finished during December of the year 2012. This information is derived mostly from articles in Cement magazine [31], [24] and [21].

Figure 115: JuBi Towers in The Hague (Courtesy of www.recystel.nl)

Description of the building

The two towers of both ministries are connected through a 10 story high office plinth. Combined, these two towers and the office plinth there is an available area of 130,000m². Most of this area is used as office space for both ministries, but at ground level there is also a public atrium and some retail located. The shape of the towers is somewhat irregular with some very sharp corners at certain spaces. It was derived by German architect Hans Kollhoff to ensure the lines of sight from the city center were not blocked by the towers. Within the floor plans this design yields some areas where offices cannot be located, in practice these areas are used as break or meeting areas for the staff. Another signature design characteristic is the setbacks in the façade, which makes the tower more slender towards the top. The structural consequences of these setbacks will be discussed in a later section.

Tube-in-tube structure

The stability system of both towers is identical. They both use their façade and their cores to cope with horizontal wind loads. This system, in which the façade and the cores together provide lateral stability is called a tube-in-tube system. In order for the façade and the cores to work together they have to be connected. In this case the horizontal loads are transferred through a bubbledeck floor, which is a two way spanning, partially precast floor system. In their plane these type of floors are capable of connecting the bending cores and the shearing framed tubes together.

The façade is constructed out of prefabricated concrete panels that are as wide as two window bays (5400mm). They do not cover two complete bays, because the panels are cut through the windows. The panels are stacked in a masonry configuration and are connected by cast in welded plates. The element layout can be seen in the
The thickness of the façade panels is 350mm below the setback of the façade and 250mm above this setback.

The cores of the towers are constructed with a climbing formwork cast in situ type system. Near the foundations the width of the walls is 350mm, but on the 27th floor this thickness is reduced to 250mm.

Setbacks in façade

In certain areas of the tower there is a setback in the façade to make the tower look more slender near the top. These setbacks will always have some impact on the structural system near this level. In this case, because the façade is key in transferring horizontal and vertical forces to the foundations the structural alterations to make these setbacks possible are interesting to look into. This section will do just that.

To explain the consequences of these kind of setbacks one will be looked into and discussed. On the 23rd floor there is a setback of 2.5m. This does not even seem to be a very large setback but the implications for the structure are very visible. To transfer the vertical forces between the two different planes, the one above and the one below the setback, additional tilted columns are introduced that have a height of three floors and are inside the building. These columns take up valuable floor area that cannot be sold or rented out. It could also make fitting your offices into your floor plan, a task already not easy due to the shape of the floor plan, even
harder. The tilted columns transfer normal forces from the façade plane above the setback to the façade plane below it. Because of tilt in the columns, these forces are not only a vertical force anymore. Due to the tilt horizontal forces are introduced into the floors where the columns start and finish, as can be seen in the image on this page. The compression forces at the top of the column can relatively easy be taken up by the floors, in which the bubbledeck balls were left out of the concrete construction. For the tension forces near the bottom of the columns the same massive floors were applied in combination with heavy tension ties to cope with the tension forces introduced by the tilt.

Concluding one can say that setbacks in framed tube structures are possible, but the structural implications can be costly. For projects on a tight budget these kind of changes to the main load bearing structure are not recommended and this should be communicated by structural engineers in an early stage in the project.

**Conclusions relevant for this thesis**

The aspects that make this building relevant to this theses are very clear. The stability system of these towers is a framed tube structure made of prefabricated concrete panels combined with a structural core to form a tube-in-tube structure. Due to the architects wishes the towers do not have a regular shape in floor plan and they both have some setbacks which alter the structural system.

The floor plans that do not have a rectangular shape do not seem to lead to too many problems in constructing a framed tube structure in prefabricated concrete. The combination of a precast concrete façade and the used bubbledeck floor also seems to be a good match. Because this floor plan is two way spanning the vertical forces are transferred to all façade walls, which decreases the possibility of tension in the bottom of the structure due to any wind direction.

The setbacks and the way they are coped with in this structural design might be illustrative for these kind of discontinuity within a framed tube structure. The conclusion might always be that setbacks are structurally feasible, but always at a price. This of course is something that a structural designer is used to in his practice. He can do a lot, but finding that solution that is both cost effective and does not interfere with the architectural concept too much is the one he has to find.

**A.3.3 Erasmus MC, Rotterdam**

The Erasmus MC is a university medical center and hospital in Rotterdam. For an expansion on the site of the medical center itself a few new buildings were built. Because there was not a lot of space to work with, but a lot of new square meters were required one of the buildings is a high-rise of 31 floors reaching a height of 120m.

The information in this section is mostly derived from [14].

**Description of the building**

The tower has a rectangular floor plan with a length of 43m and a width of 20m. The tower has a few cores
which house the elevators as well as a lot of the piping present in a building like this. These cores are not exactly in the center of the floor plan. The floors are prefabricated and they span between the façade and the cores. Where there are not any cores present a system of columns and THQ-beams is used.

**Framed tube**

This tower is a good example of a framed tube structure in prefabricated concrete. All the lateral loading is taken up by the elements in the façade. These elements are constructed out of a load bearing inner façade with
a thickness of 320\text{mm} and a concrete quality of C53/65. The outside of the sandwich panel is the cladding of the building. Using these kind of panels makes mounting a façade at a later stage unnecessary and therefore can save a lot of time. Dimensional tolerances are extremely important in these kind of systems, because the façade should provide a water an wind tight connection directly after placing.

The elements have a length of 7.2\text{m} and each element contain 4 windows. The elements are placed in a masonry configuration, eliminating the need for structural wet vertical joints between the elements. This effect has been described by Falger in [10] and is discussed in another section of this literature report.

![Figure 121: Element layout in one of the walls of the new Erasmus MC tower (Courtesy of [14])](image.png)

**Hoisting shed**

This tower was not constructed using tower cranes as are most buildings, it was created using a hoisting shed. A hoisting shed is a contraption that is permanently on top of the structure during its erection. It is capable of lifting the prefabricated elements from the ground up to the layer that is being constructed at that time. From there it uses gantry cranes to place the elements in place. If a level is finished it can lift itself up on the previously built level and start constructing a new level. The hoisting shed also functions as a safe, dry and windless construction site on top of the building, reducing the amount of days lost to bad weather.

**Conclusions relevant for this theses**

This tower is a good example of the use of the precast possibilities that are available today. It is a fairly straightforward design with decent dimensions. It is a good example of the use of load bearing sandwich panels, which do not require putting on a separate façade at a later stage. The use of the hoisting shed is also an aspect where this project can distinguish itself. Not a lot of hoisting sheds have been used in the Netherlands to this point, but projects like this show that it is a viable option for prefabricated high-rise projects in the future.
A.3.4 Maastoren, Rotterdam

The highest building in the Netherlands at the moment of writing of this report is the Maastoren in Rotterdam. Construction of its 165m was completed in November of 2009. Because the facade is constructed in prefabricated concrete the structure is relevant to this report. Most of the information comes from [34].

Description of the building

The Maastoren is a concrete high-rise tower of 44 floors, mainly for commercial use. With exception of the restaurant situated on the ground floor all of the 69,000 m² is to facilitate the two companies that use this tower as their offices. The first 10 floors house about 637 cars in a split level parking garage. The first office floor is located on the 12th floor. The floor plan is shaped as a capital letter H, two wings are connected to a central core in which the vertical transport takes place. The east wing is kinked in order to let it south facade line up with the embankment of the river Nieuwe Maas. From the 30rd floor upward the west wing is discontinued, leaving the east wing to continue on its own for the last 14 floors. For these final 14 floors the slenderness of the tower is very high. The top floor of east wing is constructed in steel and houses a board room with floor high windows.

Stability system

The stability system of the Maastoren consists of both the load bearing facades in prefabricated concrete and the cast-in-place central core of the building, creating a tube-in-tube structure. The single span hollow core slabs with a reinforced top layer that are present in both wings are not enough to link these two stabilizing elements together. This linkage is provided by two prefabricated walls on each floor which are connected with a wet connection to the core.

The load-bearing elements in the facade are in this building installed separately from the outer facade. The elements have a width of two window bays and have a thickness which is dependent on the vertical position in the tower. In 5 steps the thickness changes from 600 mm on the lowest 3 floors to 250 mm in the top 14 levels in the east wing of the tower. The elements are stacked in a masonry configuration. The advantage of using such a configuration is mainly that the vertical joints between elements do not have to fulfill a structural role and can be constructed as an open (non-structural) connection. These kind of connections are cheaper and less labor intensive [8] while being just slightly less stiff compared to a cast-in-place wall structure [10], performing better than widely used vertical connections that are not stacked in a masonry configuration. The corner connections are so called staggered connections, in which the corner is overlapped by the wall of the next level. The behavior of this connection in prefabricated concrete cores is described in [30].

Conclusions relevant for this theses

The relevance for this theses for this building should be very clear. This is the highest building in the Netherlands.
and it uses a prefabricated concrete façade and a concrete for its structural stability. Furthermore, the floor plan of the tower is irregular and has a very noticeable discontinuation which changes the floor plan over the height of the structure.

All of these aspects seemed to have been coped with well in this structure. The most prominent help that this building gives for this thesis is the demonstration of concepts that were researched by the likes of Falger, de Boer and Tolsma, in another layout than the standard rectangular one. Especially for a tower that almost reaches
the target height of this thesis (200m) is gives confidence that using the available techniques in combination with some clever engineering prefabricated tube structures can reach to higher heights. Even without giving up a lot of the architectural freedom in the process.

A.3.5 First, Rotterdam

First is a building currently construction in the city of Rotterdam, near the Central Station. Prefabricated concrete is the primary building material and its highest point will be around 125m, which makes it relevant within this literature study. This section is based on [6].

Description of the building

The total building has a gross floor area of 54,000m². Part of this floor area is situated below ground level and consists of a two level parking garage. On top of this garage, also covering the entire plot is a plinth building of 7 stories high. On top of these 7 levels, on the east side of the plot a prefabricated tower of 31 floors will be constructed.

The floor plan of the tower will be a capital letter H, in which the wings are slightly shifted from each other. The connection between both wings is made by the cast-in-place concrete core and houses the vertical transportation in the building, such as elevators and emergency stairs. The façade is made out of prefabricated concrete elements. The thickness of these elements depend on the vertical position in the building. Until the eighth floor the thickness is 500mm but near the top it is twice as thin with a thickness of 250mm. To be able to construct such a slender façade high concrete strengths were used in specific areas, up to a concrete class of C90/105. A similar class was used for the cast-in-place concrete near the foundation of the building for the basement walls and the core.

Framed tube and tube-in-tube insufficient

For the stability of the structure different systems were looked into. The first one was a simple framed tube structure, using only the load bearing façades as a lateral stability system. This was not deemed possible because of two elements in the architectural design. The first one was the open character of the plinth building on which the tower was build. To have such an open character the structure cannot be very dense. A high structural density was needed however according to calculations. The second problem was the architectural concept of the façade. The large window sizes and overall look just did not suit the elements needed for this stability system.

The framed tube structure can be upgraded by also using the core to provide some lateral stability. This was the second option which was looked into. If the core could take up enough lateral loads the façade elements could be redesigned to fit in with the architectural boundary conditions described above. In the end though, even using a tube-in-tube structure was not enough to come to a profitable solution. For a tube-in-tube structure the forces on the floor increase for they have to transfer forces between the façade and the core. In this particular case the floors would get too deep and therefore too heavy. This not only has cost implications for the floors.
themselves, but also due the added weight on other parts of the structure, rendering this stability system unfit for this building.

Outrigger system
To provide this building with a fitting stability system, after the framed tube and tube-in-tube options were seen unfit, engineers turned to an outrigger system. In an outrigger system the core of a building is connected at one or more levels to vertical load bearing parts in the façade. If the core bends the outrigger pushes the façade elements on one side of the building down, leading to compression and pulls elements on the other side up, leading to tension forces. For each tower an optimum floor for an outrigger can be calculated. Unfortunately it was not possible to place the outrigger on the optimum floor due to demands of future tenants to the uniformity of their floors. The outrigger will be placed on the 20th floor, a few levels below the optimum floor. The outrigger will be placed perpendicular to the wings of the building, creating only an outrigger on the shortest axis in the building. In the other direction, parallel to the wings, the framed tube action of the prefabricated elements was enough to resist the lateral forces.

Conclusions relevant for this theses
First in Rotterdam is an example of a tower in which a framed tube structure and even a tube-in-tube structure are not enough to provide the lateral stability a tower needs. This is not because of the large height of the tower, 125m framed tube structures are possible, as can be read in earlier sections. It were other aspects that prevented this building from using this particular stability system. One of the reasons was a preferred openness on the lower levels, something that is demanded quite often by architects. The other reason was that the structural system did not fit the façade, something a structural designer cannot influence most of the time. So the most important conclusion for this theses that can be extracted from the research into this building is the following: sometimes it just cannot be done with this stability system for a reasonable price. A high structural density is required for this structural system in the façade near the bottom of the tower and this density remains fairly high throughout the façade. This density can limit the applicability of this structural system.
Figure 127: Floor plan of First in Rotterdam (Courtesy of [6])