CT5060 Master Thesis

Application of Higher Strength Concrete in Tubular Structures

Main Study Report

August 23, 2011

Author:
Heykal Balbaid
1344102

Delft University of Technology (TU Delft)
Civil Engineering and Geosciences

Thesis Committee:
dr.ir.drs. C.R. Braam
prof.ir. R. Nijsse
ir. K.C. Terwel

Abstract

Most buildings constructed in the Netherlands are constructed with steel and/or reinforced concrete as the main load-bearing material. The use of concrete in high-rise buildings has the advantage of being able to build rather stiff and fire proofing structures thanks to the large applied amount of concrete. Structural stiffness is important in high-rise structures since the maximum lateral displacement at the top has to be limited to meet the required comfort level in a building.

An often-used type of structure is the framed tube structure. Several studies have pointed out that the tubular structure is able to achieve great heights. It highly improves the building's stiffness, resulting in less lateral displacements. In the last decades, higher buildings are being built in seeking the ultimate height limit of a structure.

While buildings were being built higher and higher, the quest for higher quality materials continued as well. Until now, many new types of concrete have been developed, which have better properties than its predecessors. This thesis applies two new types of concrete in a framed tube structure, namely High Strength Concrete (HSC) and Ultra High Strength Concrete (UHSC).

It appears that higher strength concretes can be applied in the structure. Its behaviour changes since the higher strength concretes have a different modulus of elasticity. The structures are proven to give better performance: the lateral displacements reduce and higher structures can be built while still fulfilling the requirement to maximum deflection.

Obviously, the higher strength concretes come with a higher price. Consequently, the structure becomes more expensive. However, thanks to the better material properties, the building can be built higher. To acquire knowledge in whether the higher building is feasible, the costs per floor are calculated. A higher building contains more commercially available area and the analysis shows that the costs per unit floor area decrease by 14% if HSC is applied. Despite the fact that the building with an UHSC structure contains more commercially available area, it is not beneficial due to the high price of the UHSC mixture: the costs increase up to 25%. The Very High Strength Concrete (VHSC) mixture is not studied in this thesis, but is expected to provide costs and structural performance that lay in between the HSC and UHSC performance.

Several changes to the structure are recommended to optimise the structure's costs-performance ratio. The most important recommendation is to utilise the material properties of the higher strength concretes as much as possible. This is achieved by applying a hybrid structure: combining higher strength concrete with ordinary concrete in one structure. HSC is applied in only the lower eight floors, while in the remaining 22 floors of the structure OC is applied.

The thesis shows that the application of higher strength concretes in framed-tubular structures is possible. It provides better performing structures and, in some cases, a reduction in costs. While the HSC models proved to provide a good performance to costs ratio, the UHSC models currently do not.

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1. Preface

Until now, most of the buildings constructed in the Netherlands are constructed with steel and/or reinforced concrete as the main load-bearing material. When it comes to high-rise buildings, the amount of steel reinforcement in concrete becomes very significant. Sometimes the required amount of steel reinforcement is that high, that the chance on brittle failure becomes too high and additional measures are required.

The use of concrete in high-rise buildings has the advantage of being able to build rather stiff structures due to the large applied amount of concrete. This is important in high-rise structures since the maximum lateral displacement at the top has to be limited to meet the required comfort level in a building.

Structure
Concrete can be used as a load-bearing material in various types of structures. An often-used type of structure is the core structure. The inner core provides the building's stability, but has limited stiffness and limits the layout possibilities. One of the alternative structures is the tubular structure. Its structure is placed in the façade in order to provide load-bearing capacity and, at the same time, great stiffness due to its great lever arms. A downside of the tubular structure is that it does not suit itself for (very) large façade openings.

Several studies have pointed out that the tubular structure is able to achieve great heights. It greatly improves the building's stiffness, resulting in less lateral displacements. Mostly, the maximum possible height is determined by the maximum allowable lateral displacement. If the building's height increases, this requirement might not be met.

Another factor, which can limit the maximum possible height, is the structure's strength. To be able to increase the building height, the structure can be designed more robust. This is often not favourable from an aesthetics point of view. Consequently, engineers were looking for other solutions to increase the strength; hence, the demand for materials with improved properties grew. Generally, concrete structures in the Netherlands are built with Ordinary Concrete (OC) with strength classes between C28/35 and C45/55. The numbers indicate the characteristic compressive stresses (cylindrical stress/cubicle stress).

In the last decades new types of concrete have been developed, such as High Strength Concrete (HSC). Concrete strength classes between C55/67 and C100/115 refer to HSC. When these strength values go further we arrive in the category of Very and Ultra High Strength Concrete. The compressive stress level of Very High Strength Concrete (VHSC) varies between 100 N/mm² (C100/115) and 150 N/mm². Above these stresses, the concrete is called Ultra High Strength Concrete (UHSC). By applying higher strength concretes, the engineer is able to create a stronger structure and is able to reduce the size of the structural elements.

Complexity
Despite the great advantages of the tubular structure, it also comes with some disadvantages. Large forces can occur in the façade-elements because of the window-openings. In addition, the structure's displacements are not easy to calculate due to its rather complex structural form.

Consequently, large geometries are required and a Finite Element Method (FEM) analysis is preferable.

Goal
Will the use of higher strength concretes in tubular structures provide any advantages when it comes to structural behaviour? The goal of this thesis is to research the possibility of the application of High Strength Concrete (HSC) and Ultra High Strength Concrete (UHSC) in tubular in-situ structures, summing up advantages and disadvantages and making comparisons with similar OC structures, including a limited discussion on the costs of the various models. The model's geometry is kept the same throughout the research to ensure proper comparison.

2. Objectives

The objectives of this thesis can be described as follows:

- Researching the possibility of the application of HSC and UHSC in tubular structures;
- Summing up advantages and disadvantages;
- Performing a comparison with similar OC structures, including a limited discussion on the costs of the various structures;
- Researching the possibility of achieving a greater building height while maintaining the same geometry and element sizes and still meet the requirement of maximum lateral deflection.

2.1 Researching the possibility of the application of HSC and UHSC in tubular structures

As one of the limits of a tubular structure is the material's strength limit, it is interesting to research the possibilities of a tubular structure constructed with higher strength materials. A tubular structure generally suffers from certain weak points (large structural elements and peak forces). The question is whether there will be any improvements when using higher strength materials. To answer this question, the structure needs to be researched and compared with the structure that uses OC. Very High Strength Concrete, which lies in between HSC and UHSC, is not studied in this thesis, as no additional problems or large differences are expected in the results.

2.2 Advantages and disadvantages

Obviously, higher strength concrete comes with the advantage that the material can resist higher compression forces. At the same time, the material is stiffer, which can be useful when it comes to overall deflections and building stiffness. Because of the high strength, more slender structures can be built while still meeting the requirement on lateral displacement.

These advantages also have a downside. The higher strength materials require additional care when applied in a structure. The behaviour is different and, therefore, the approach on structural design methods is different.

The thesis presents the advantages and disadvantages and discusses how to deal with the problems that might be present when using higher strength concrete.

2.3 Comparison with similar OC structures

To determine whether the structure that uses High Strength Concrete (HSC) or Ultra High Strength Concrete (UHSC) performs better, a comparison with a similar structure designed with Ordinary Concrete (OC) is required. This way the differences become very clear between the different structures.

The costs of the different structures are discussed to show if it is a feasible option to use a higher strength concrete. One has to bear in mind that the costs aspect is not limited to only the costs of the material. The actions required on site differ in each structure and, therefore, the costs can be different each time.

2.4 Optimisation of the Structure and Alternatives

One of the results of the research could be that a HSC or UHSC structure does not fully utilise its high potentials. The use of this type of concrete is maybe not necessary as a lower strength concrete could fulfil the task. However, one can imagine that, when using higher strength concrete, more slender structures can be built. This is one of the possible optimisations, which could result in a more feasible structure.

Above optimisations can make the use of HSC and UHSC in tubular structures more interesting and are therefore taken into account in this thesis.

3. Literature Study

Before the start of the thesis's analysis, a literature study (1) is performed. This chapter is a summary of this literature study report and briefly discusses the major points found in the study.

3.1 The Building's Structure

When designing and constructing a high-rise structure several challenges are being encountered. These challenges deal with issues in the foundation, the load bearing structure, and the structural shape.

Foundation
The foundation of the building is required to carry the loads from the building into the soil. The local ground conditions and the behaviour of the subsoil are major risk factors in the building industry, particularly in high-rise construction. Insufficient load-bearing capacity of the subsoil can result in serious settlements of the structure. Several types of foundations are developed to cope with the various soil types. A proper foundation prevents large settlements or settlement differences. These differences can cause damage to the building, varying from little cracks to severe failure of the structure. Furthermore, a rotation of the foundation results in an additional horizontal deflection at the top of the building.

Load bearing structure
In a high-rise building, the gross floor area is much larger than in a low-rise building. As a result, the magnitude of the vertical forces is far greater as the forces concentrate on a relatively small area. Due to the increasing height of the building, the horizontal forces acting on the building will increase as well. The load bearing structure is designed to carry both of the loads properly.

To absorb the lateral loading caused by wind, a building must contain a structure that provides resistance against this lateral loading. For tall or high-rise buildings, it is necessary to apply special stabilising structures such as moment resisting frames, shear walls or cores. Several types of stabilising structures are developed. One of these structures it the Framed Tube Structure.

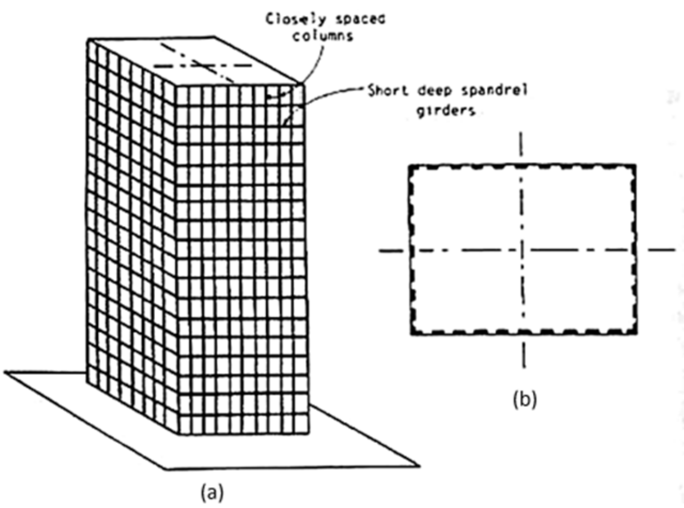


Figure 3.1 Framed tube structure (a) elevation; (b) structural plan. (1)

Framed Tube Structure

In a tube structure, the structure is placed in the façade of the building. The most basic framed-tube structure consists of four orthogonal rigidly jointed frame panels forming a tube in plan. Closely spaced perimeter columns that are connected by deep beams at each floor level form the frame panels. The most important requirement of a framed-tube structure is to place as much of the load-carrying material at the extreme edges of the building to maximize the moment of inertia of the building's cross section.

When the structure is subjected to bending under the action of lateral forces, the primary mode of action is that of a conventional vertical cantilevered tube. The columns on opposite sides of the neutral axis are subjected to tensile and compressive forces, as indicated by the dashed lines in figure 3.2. In addition, the frames parallel to the direction of the lateral load are subjected to the usual in plane bending, and the shearing action associated with an independent rigid frame.

Because of the flexibility of the deep beams, the structure acts slightly different. This flexibility causes the structure to increase the stresses in the corner columns and reduces those in the inner columns of both flange and web panels. This behaviour is illustrated in figure 3.2 as the solid lines and is known as the "Shear-lag Effect".

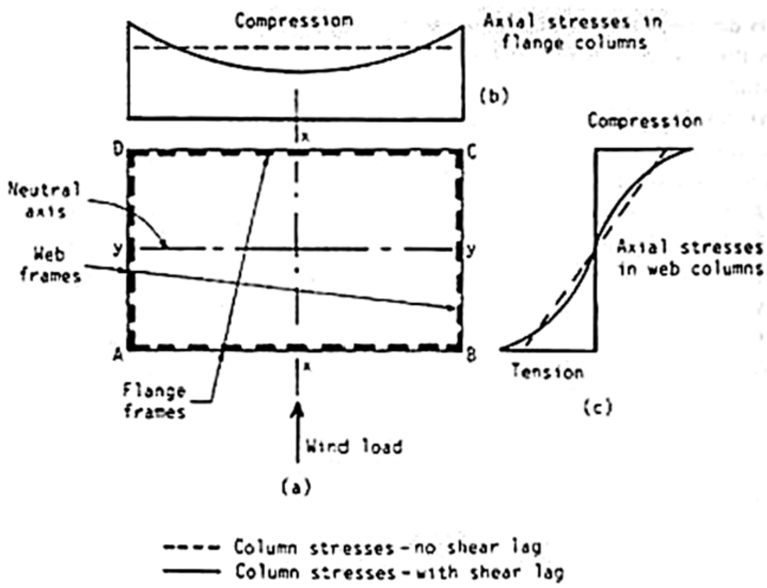


Figure 3.2 Shear-lag Effect. (1)

The framed-tube structure is a non-parallel combined system. Therefore, the total deformation of the structure consists of the deformation of the "Overturning moment" component plus the deformation of the "Shear racking" component. Figure 3.3 illustrates the two components.

In a study written by Faessen (2), several geometries of tubular structures were discussed where insight is given on the maximum height of a concrete tubular structure. Variation is made in the sizes of the structural elements and the plan geometries. The building´s height is limited by the maximum sway of the building as described in the code, which is 1/500th of the total building height.

To make it possible to acquire an estimation of the most imported forces and stresses in a framed-tube structure, Faessen (2) made several rules of thumb. The rules of thumb provide an accurate

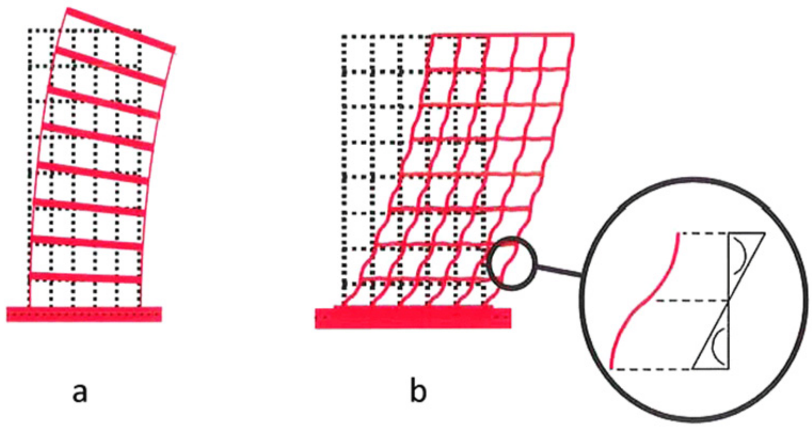


Figure 3.3 Deformation caused by the "Overturning moment" component (left) and "Shear racking" component (right). (1)

estimation of the lateral displacement at the top of the building, maximum normal force in the outer columns, maximum shear force and bending moment in the columns and deep beams and the maximum vertical reaction force at the corner columns of the structure.

3.2 From Ordinary Concrete to Ultra High Strength Concrete

At the moment, many researches are being performed to acquire more knowledge about High Strength Concrete (HSC). Improving the performance of Ordinary Concrete (OC) can be achieved by many diverse optimisations, which have a lot in common. Concrete can be classified as a three-component-system consisting of:

- the aggregates,
- the matrix,
- and the contact surface between the aggregates and the matrix.

The aggregates are schematised as round balls with equal diameters. The acting force is being absorbed via the contact areas between the aggregates. As they are part of staggered grid not only normal forces occur, lateral forces are also introduced to create the force equilibrium. These lateral forces are being absorbed by the cement, which is acting as a sort of glue. Compared to the high normal forces the lateral forces occurring in the matrix are relatively low, making it easy for the cement to transfer the forces.

Usually the strongest component in the concrete's composition is the aggregates. Generally, failure in the concrete occurs when the tensile force in the concrete cannot be absorbed, see figure 3.4. By increasing the packing density and the cement strength, the total strength of the mixture is increased. However, due to this increase, a different failure mechanism will occur, see figure 3.5. Therefore, the aggregates are required that reduce the chance of this failure mechanism. As the result of these optimisations, High Strength Concrete (HSC) is developed.

To improve the strength of the concrete further, more optimisations are required. These optimisations are to improve the homogeneity, increase the packing density, improve the microstructure, and to increase the ductility. To achieve these optimisations smaller aggregates,

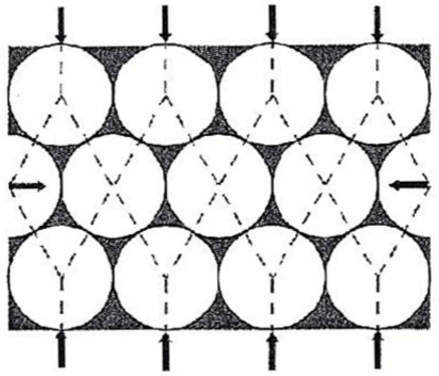


Figure 3.4 Simplified illustration of the failure mechanism of ordinary concrete. (1)

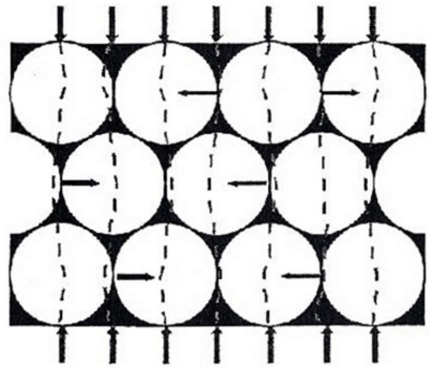


Figure 3.5 Failure mechanism in HSC: cracks through the aggregates. (1)

puzzolanes, silica fume, and steel fibers are added to the concrete mixture. Eventually, higher strength concretes are acquired from these optimisations. The maximum stress level of the concrete mixtures can go up to 200 N/mm² (and higher).

Summary of Concrete Mixtures

Table 3.1 summarises a few of the various concrete mixtures available on the market. The noted properties in the table are the most important properties of the mixtures. Next to the listing of the HSC and UHSC material properties, the values of OC are also listed to make comparison easy.

Mixture	C35/45	C70/85	C100/115	C200	BSI	Secutec S9	Ductal
Cement (kg/m³)	360	475		1075	1100	-	710
Binder (kg/m³)					-	1100	-
Silica fume (kg/m³)	-	25		165	165	-	230
Quartz powder (kg/m³)					-	-	210
Sand (kg/m³)	790	785		1030	1050	-	1020
Bauxite 0-1mm (kg/m³)					-	685	-
Bauxite 5-8mm (kg/m³)					-	625	-
Gravel (kg/m³)	1110	960		-	-	-	-
Steel fibers (kg/m³)				235	235	200	40-160
Plasticizer (kg/m³)	0.5	4.6		40	40	-	13
Water (kg/m³)	145	150		200	200	200	140
Mass density (kg/m³)	2405	2400		2810	2800	2850	2500
Water-cement factor (-)	0.4	0.3		0.16	0.15	0.18	0.15
Compression strength f_{ck} (N/mm²)	35	70	100	200	180	183	200
Mean tensile strength f_{ctm} (N/mm²)	3.23	4.6	5.2	7.8	16.8	9.5	21.7
Young's Modulus E_c (N/mm²)	33500	39300	48600	55000	65000	64000	50000

Table 3.1 Concrete mixtures with their properties

Higher Strength Concrete as the Main Building Material

Creating a higher strength mixture requires more care and attention. The most important factors that need more care in comparison with OC are the mixing procedure, production capacity and the pouring ability. Therefore, a detailed plan of mixing is required to assure that the mixing will take place in a proper way, resulting in the required product. Because of the different method of mixing, the capacity of the factory will be lower. Therefore, the planning of delivery to the building site is crucial.

3.3 Conclusion Literature Study

Possible application of HSC and UHSC in tubular structures

Higher strength concretes can be used without any major issues or changes to the design. The main principle and design of the tubular structure can be maintained with the application of higher strength concrete mixtures.

The higher strength concretes come with both advantages as well as disadvantages. However, the disadvantages can be acceptable by applying good care during design and construction stage.

Advantages

- Improved material strength and properties.
- When applying UHSC with fibers no additional steel reinforcement is required (in certain situations). When needed, pre stressing still needs to be applied. The lack of additional reinforcement reduces production costs.
- A high density comes with a high durability. Consequently, concrete covering can be reduced or even neglected if no reinforcement bars are applied, reducing overall thickness.
- A high strength is achieved very fast after pouring, creating a higher build speed.
- Higher pre stressing can be applied.

Disadvantages

- Higher strength concrete has a larger magnitude of autogenous shrinkage compared to OC. Most of the shrinkage occurs in the first few days after pouring.
- The hydration process in higher strength concrete is very fast, resulting in a higher temperature production. This can result in cracks in the concrete, in most cases this happens when the concrete is used in thick elements.
- UHSC without fibers acts very brittle. Adding the fibers solves this problem.
- Production capacity at the concrete factory is reduced.
- Higher strength concrete is more expensive than OC.
- Until now there is no standardisation when it comes to strength classes of UHSC.

3. Literature Study

4. The Building's Design

The main idea of the tubular building's design is to make as much as possible use of the load-bearing elements in the façade of the building. There is no need of using a load-bearing inner core as the horizontal wind loads are also absorbed by the façade, resulting in a more flexible surface area in the building.

4.1 Geometry

The geometry of office buildings is mostly bounded by Dutch regulations concerning the health and safety level of people in the building. The Health and Safety Act (Arbo-wet) requires a workstation to be located near a window within a specified maximum range. This directly limits the maximum distance between the facades as otherwise the centre area cannot be used as the requirement to daylight entry is not met.

In residential buildings, similar regulations have to be met. This time, most rooms in an apartment are required to have sufficient daylight entry. Consequently, a maximum width is preferable to create an efficient apartment layout that meets the requirement of daylight entry.

The building as analysed in this thesis, can function as a residential and/or office building. The main structure and geometry will remain the same in both situations. The main difference between the two is found in the dead and live loads. However, the difference is small and therefore, for the calculations of the model, the governing loads (in the case of usage as an office building) are applied.

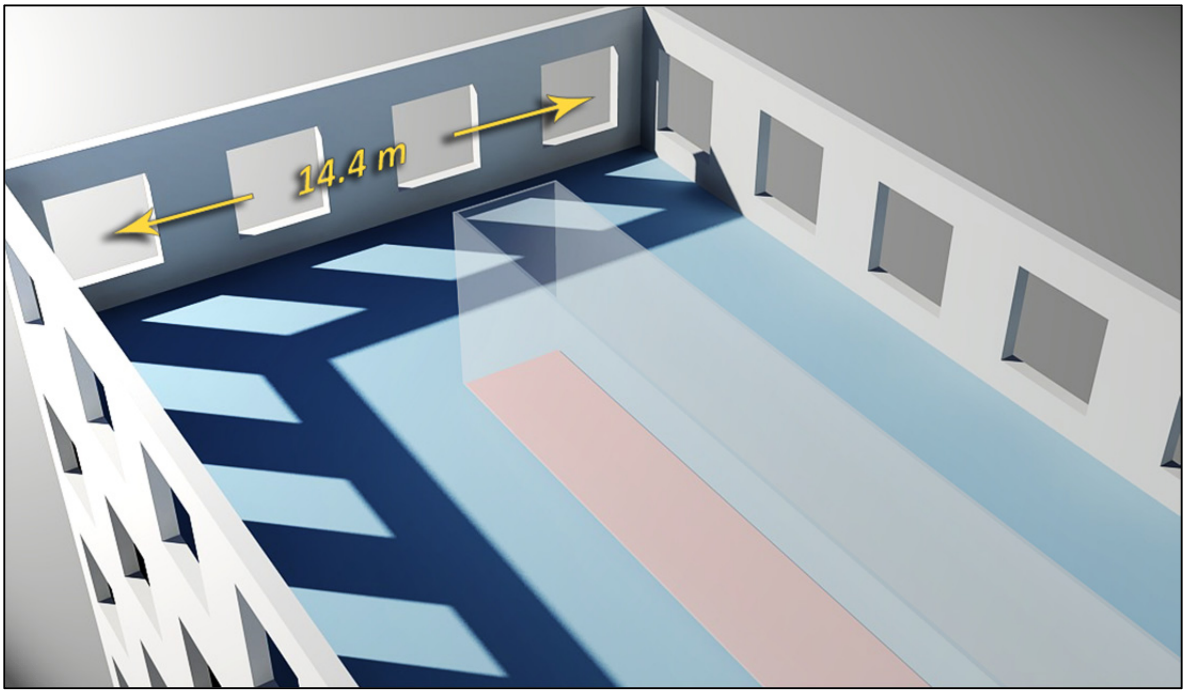


Figure 4.1 Typical floor layout.

The geometry principle analysed in this thesis originates from the thesis by Faessen (2). It has a typical floor layout with a length of 36 metres and a width of 14.4 metres. The rectangular shape can be recognised and is a direct result of the required daylight entry. Figure 3.1 illustrates the typical floor layout where the blue area represents the "office area".

The floor layout represents a real-life situation in an office building. At the same time, the floor plan can easily fit in the requirements of a residential building with several apartments at each level. Apart from being a very realistic floor plan, the floor plan is identical to the floor plans used in Faessen's thesis (2). As a result, an easy comparison is possible when it comes to the results of the rules of thumb as stated in Faessen's thesis (2) and the actual behaviour of the building.

4.2 Height

While the building's length and width are set as a fixed numbers, the height of the building is a variable. The reason for this is that one of the goals of this thesis is to analyse what the maximum height can be. Several factors come into play when determining this limit like element geometry and material stiffness.

Figure 4.2 Rendered model of the building's structure.

The building height starts at 20 floor levels. With a typical 3.6 metres floor level height the total building height starts at 72 metres. From this point, the building height is increased until the limit is reached. The building's height is limited by the maximum sway of the building as described in the code (3), which is 1/500th of the total building height.

4.3 Structure

The building's structure is a framed-tube structure without any load-bearing inner core. The tube structure itself will carry the vertical loads and will provide the building's stability. The tube structure transmits the loads into the foundation, via a full level height wall. This wall is located on the basement floor and only a limited number of openings are required. In most buildings, these walls are present in the lower basement floors and provide a solution to distribute the loads more evenly to the foundation. Consequently, less peak stresses are found in the structure.

The structure rests on a foundation slab that is slightly larger than the typical floor plan. This is done to provide sufficient rotational stiffness, which is required to minimise the rotation of the building when loaded in horizontal direction. The foundation distributes the loads over a grid of foundation piles, which are installed at a certain depth to ensure sufficient load bearing capacity.

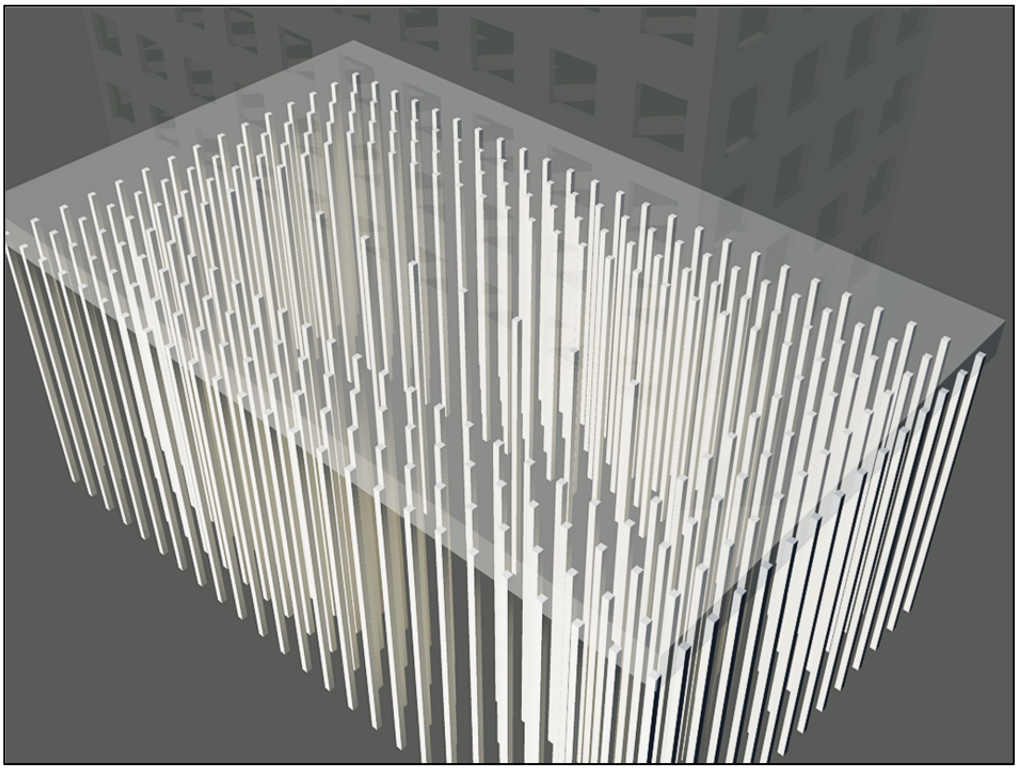


Figure 4.3 Foundation slab (transparent) and piles

To analyse the influence of the application of different concrete types, several models have been developed to acquire insight in the behaviour of each model. Apart from the column size difference, which will be discussed in chapter 7, the geometry of the structure itself is not any different. The types of concrete that are taken into account in the analysis are stated in table 4.1.

Concrete Type	Strength class as used in this thesis
Ordinary Concrete (OC)	C35/45
High Strength Concrete (HSC)	C90/105
Ultra High Strength Concrete (UHSC)	C180/200

Table 4.1 Concrete Types

4.4 Floor System

The floor system used in the structure is the often-used hollow-core slab. This system enables to achieve great span lengths without a high self-weight. The slabs rest on console-like elements connected to the load bearing façade. The slabs span in the “short” direction from façade-to-façade, which results in a span of 14.4 metres. This span is only achieved with hollow-core slabs with a height of 320 or 400 mm (4).

Figure 4.4 Hollow core slabs

The required fire resistance for the main supporting structure of a residential or office building with a height greater than 13 metres is 120 minutes. This implies that the floor system used in this building has to meet this requirement. The hollow core slab with a height of 320 mm does not fulfil this requirement. Therefore, only the hollow core slab with a height of 400 mm can be applied.

Due to the wind loading, the floor slabs need to have sufficient stiffness to distribute the loads to the other parts of the structure. The concrete filling between the floor elements is able to transfer the loads to the adjacent element but it does not provide sufficient slab stiffness to ensure proper load distribution throughout the whole slab. Therefore, a 70 mm thick layer of reinforced concrete is applied on top of the slab to create the required stiffness.

5. Loads

5.1 Vertical Loads

The vertical loads acting on the building are based on common used assumptions for office-loadings. The reason for using typical office loads instead of residential loads is that the office loads are governing.

5.1.1 Self-weight & Dead Loads

Structure

Due to the use of several different concrete types, the self-weight of the structure is not always the same in every model. The mass density of each concrete type is listed in table 5.1.

Concrete Type	Strength class	Mass density [kg/m³]
Ordinary Concrete (OC)	C35/45	2405
High Strength Concrete (HSC)	C90/105	2410
Ultra High Strength Concrete (UHSC)	C180/200	2810

Table 5.1 Concrete mass densities

The mass densities are based on both the code (3) and previous research (5). The high mass density of UHSC is a result of the high number of steel fibers in the concrete mixture.

The concrete types, as listed above, will be applied to the whole structure except the floor slabs. The self-weight is calculated and taken into account directly by the finite element analysis software in a separate load case.

Floor loads

The floor construction of a typical office area is determined and is listed in table 5.2.

Type	Composition	kN/m²
floor level	hollow-core slab (4)	5.00
	concrete topping d=0.07*24=	1.68
	ceiling	0.50
	finishing	1.00
	$q_{g;k}$ =	8.18

Table 5.2 Floor loads

All floors of the structure, except the roof, are loaded with the loads as mentioned above. As the floors span from façade to façade in short direction, the long façade is loaded with these loads. The short façade is only loaded with the weight of the cladding (the self-weight is already taken into account).

Façade loads

The cladding present on the façade is connected directly onto the structure. The composition of the façade is set as a medium-heavy type façade. This implies that cladding like brickwork or natural stone elements can be used.

The weight of the openings is taken into account as window frames with double glazed fillings or its equivalent. Table 5.3 lists the façade loads.

Type		kN/m²
Façade (without inner concrete structure)	$q_{g;k}$ =	2.00
Façade openings	$q_{g;k}$ =	1.00

Table 5.3 Façade Loads

5.1.2 Live Loads

The live load acting on the floors is based on the recommendation as stated in the code (6) & (7) supported with the National Annex for The Netherlands. As the building functions as an office, it is categorised in category B. This implies the following:

	$q_{q;k}$ [kN/m²]	$Q_{q;k}$ [kN]	ψ_0 [-]	ψ_1 [-]	ψ_2 [-]
Category B: Office area	2.50	3.00	0.5	0.5	0.3

Table 5.4 Live Loads & ψ-factors

The above values have to be taken as a minimum. To take into account potential changes in loading during the lifetime of the building the surface load is chosen to be 3.5 kN/m² instead of 2.5 kN/m².

The code (7) requires that the weight of potential separation walls is classified as a live load. Therefore, the total live load acting on the floors is the sum of the office load and the weight of the separation walls, see table 5.5.

Total Live Load	$q_{q;k}$ [kN/m²]
Category B: Office area	3.50
Separation walls	0.80
Total	4.30

Table 5.5 Total Live Load

5.1.3 Line Loads

As the floor spans in the short direction, the line loads on the short and long side of the façade differ from each other. Figure 5.1 shows the distribution of the line loads.

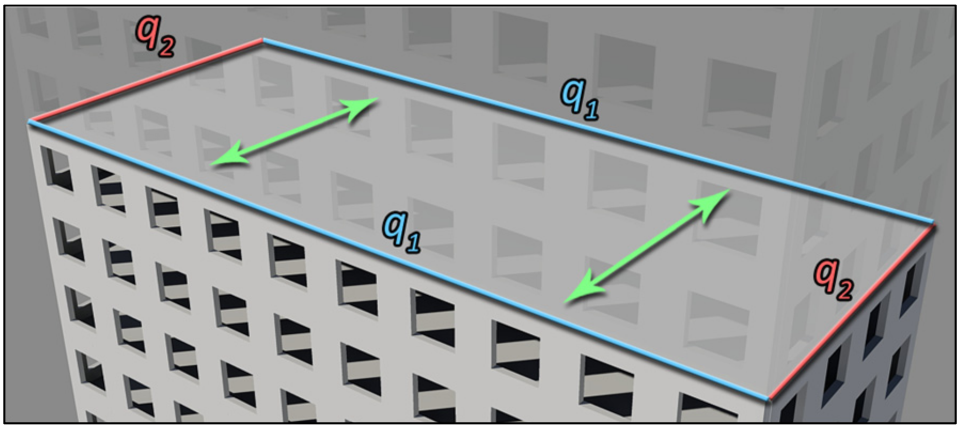


Figure 5.1 Line Loads

By multiplying the loads to the correct span length or height, the following line loads are acquired:

q_1

	ψ	#	length	kN/m^2	kN/m^1		kN/m^2	kN/m^1
floor level	1	1.00	7.20	8.18	58.90	extr.	4.30	30.96
façade (without inner structure)		0.69	3.60	2.00	4.97			
façade openings		0.31	3.60	1.00	1.12			
				$q_{g;k}$ =	64.98		$q_{q;k}$ =	30.96

q_2

	ψ	#	length	kN/m^2	kN/m^1		kN/m^2	kN/m^1
façade (without inner structure)		0.69	3.60	2.00	4.97			
façade openings		0.31	3.60	1.00	1.12			
				$q_{g;k}$ =	6.08		$q_{q;k}$ =	0.00

5.2 Horizontal Loads

5.2.1 Wind Loads

The wind loads acting on the structure is determined by using the recommendation as stated in the code (7) supported by the National Annex for The Netherlands. The wind load depends on several factors, which influence the maximum wind pressure:

- Building location: Area II; built-up
- Width & Depth: 14.4 x 36.0 m^2
- Height: 72-180 m

The wind pressure acting on the outside of the building can be determined by:

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

Where:

$q_p(z_e)$ is the extreme wind pressure (table NB.4 in the national annex of Eurocode 1 (7))
z_e is the reference height
c_{pe} is the pressure-coefficient

Reference height

The reference height z_e depends on the total height h and width b of the building. This reference height is acquired using the recommendations in the code as seen in figure 5.2.

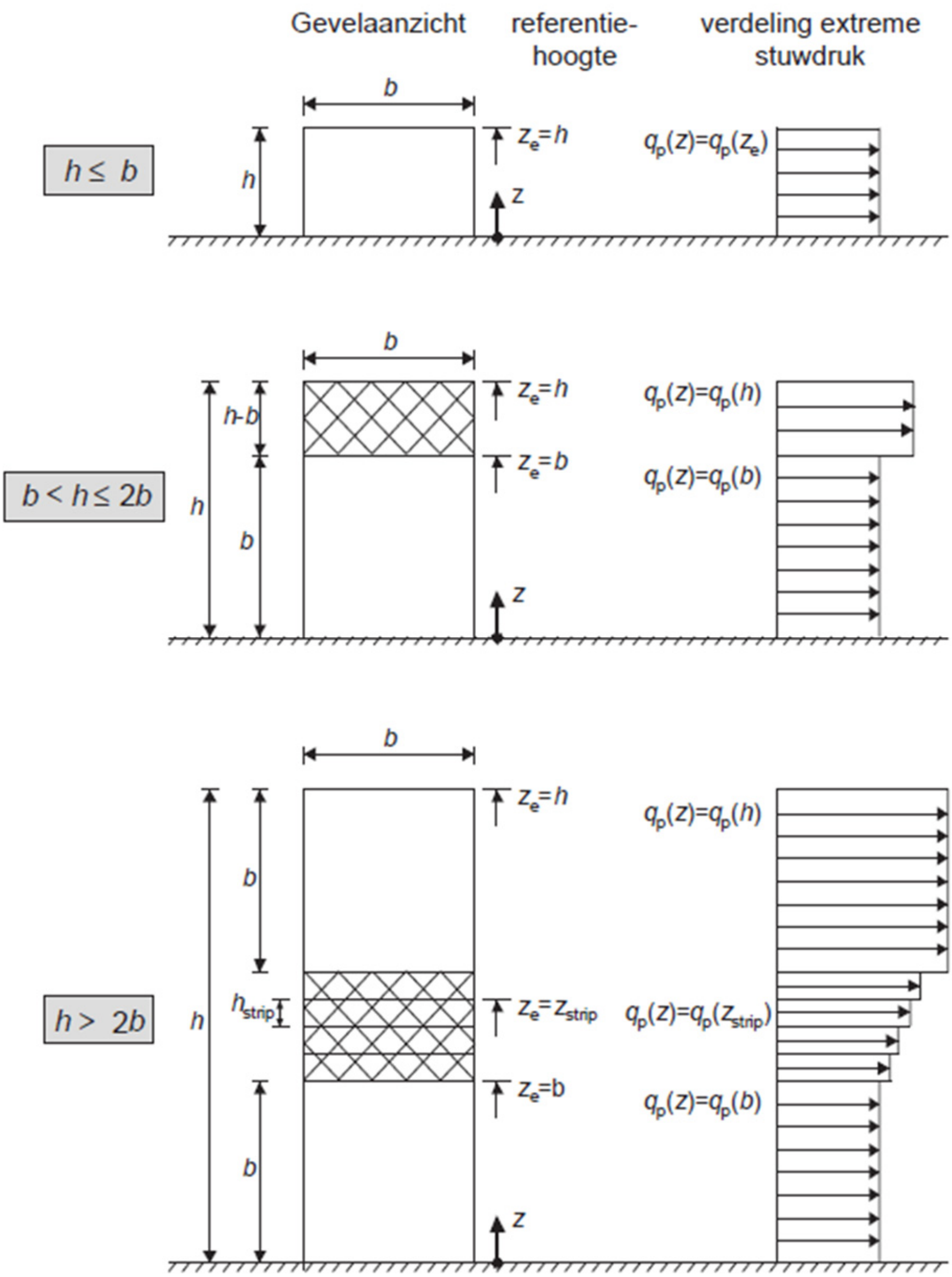


Figure 5.2 Reference heights (7)

In the situation where "h > 2b" a height z_{strip} is introduced. This height is chosen to be five levels high, making it 18 metres. Making the height z_{strip} smaller will result in smaller steps of increasing the wind pressure. However, the smaller steps make creating the model more time-consuming. The chosen height of 18 metres results in acceptable increments while it is not very time-consuming.

Pressure coefficient

The factor c_{pe} is a factor that depends on the building's geometry and consists, in this specific case, of the factors $c_{pe,D}$ and $c_{pe,E}$. Figure 5.3 (7) illustrates side D and E.

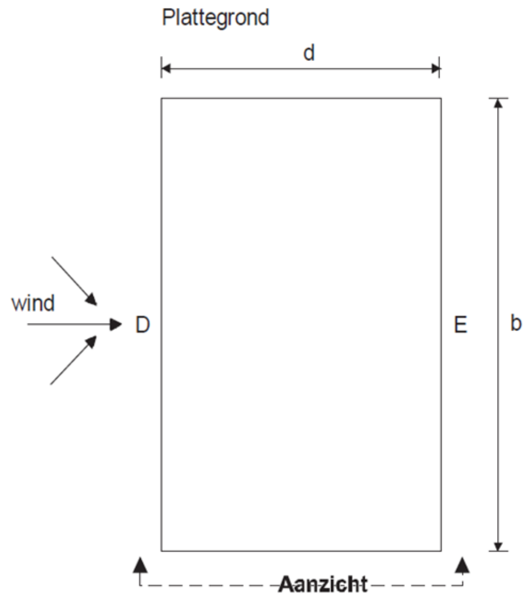


Figure 5.3 Pressure-coefficient determinations as in the code (7).

It is clear that side D is the windward side and side E is the lee side of the building. To determine each factor the code recommends the following:

Zone	D	E
h/d [-]	$c_{pe,10}$ [-]	$c_{pe,10}$ [-]
5	0,8	–0,7
≤ 1	0,8	–0,5

Table5.6 Pressure Coefficient

For the h/d values between the numbers stated in the table, linear interpolation must be applied (7). Additionally, due to the lack of correlation of wind pressures between the windward and the leeward side a factor of 0.85 must be applied on the resulting wind pressure w_e (7) (National Annex).

Calculated wind pressure

The wind pressure can now be calculated with the previous stated equations and factors. Because several factors are involved in determining this end value, a worksheet is used to calculate all of the relevant values. This worksheet is presented in appendix B and is based on all of the above equations and factors. The end result is the line load that is working on the façade in x- or y-direction, depending on which side is loaded with the wind pressure: the short side (b=14.4) or the long side (b=36.0).

Table NB.4, from the national annex of the code, is used to determine the extreme wind pressure. However, to make it more usable in the worksheet, the increments have been made smaller by linear interpolation.

The final value q_{wind} is the line load acting on the structure caused by the wind pressure. The wind pressure is multiplied by half of the width of the building and is applied on both sides of the structure's façade.

5.2.2 Initial Skew Loads

The additional loading by imperfections of the building can be taken into account by the initial skew loads. Eurocode 2 (3) recommends taking this load into account by the following formula:

$$\theta_i = \theta_0 \cdot \alpha_h \cdot \alpha_m$$

Where:

θ_0	is the base value	
α_h	is the reduction factor for the length or height:	$\alpha_h = {}^{2}/_{\sqrt{l}}\ ;\ 2/3 \leq \alpha_h \leq 1$
α_m	is the reduction factor for the number of elements:	$\alpha_m = \sqrt{0.5(1+1/m)}$
l	is the length or the height	
m	is the number of vertical elements	

The national annex of Eurocode 2 (3) states that a base value of 1/300th is to be used. Due to the large building height and the large number of elements (levels) in the building, the two reduction factors stay the same for every model.

The factors become:

$$\alpha_h = \frac{2}{3}\ ;$$
$$\alpha_m = \sqrt{0.5}\ ;$$

As a result, the initial skew is:

$$\theta_i = \theta_0 \cdot \alpha_h \cdot \alpha_m = \frac{1}{300} \cdot \frac{2}{3} \cdot \sqrt{0.5} = 0.00157\ [rad].$$

The consequence of the skew is the addition of a horizontal load. To determine this horizontal load the weight of the building needs to be multiplied by the skew. This will result in the total horizontal load caused by the initial skew. To apply this load on the structure it is divided by the total height of the building and then applied as a horizontal line load onto the structure.

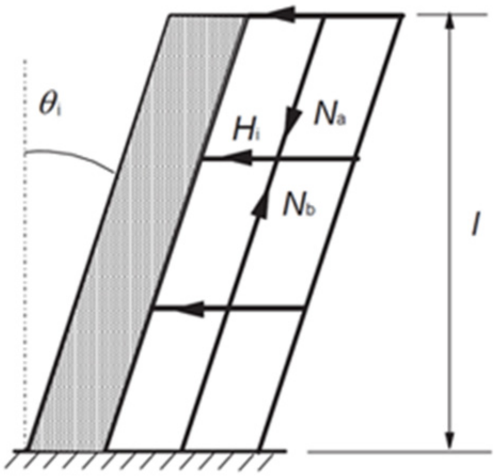


Figure 5.4 Initial skew (3)

Total horizontal load: $H_i = \theta_i \cdot N$

The parameter N represents the total weight of the building. The horizontal load is then applied to the structure as a horizontal line load: $q_i = H_i/l$.

The total weight of the building models varies as the concrete types differ with each having a different mass density. Obviously, the total weight of the building is also influenced by the total height. Therefore, these two variables are taken into account when determining the loads by caused by the initial skew. The results can be found in the tables below.

Model variant 1 (beams 1.6m & columns 1.6m)							
OC & HSC	levels	N [kN]	L [m]	H_i [kN]	q_i [kN/m]	**UHSC** [-]	q_i [kN/m]
	20	174944	72	274.9	3.818	x 1,048*	4.001
	30	271113	108	426.0	3.945	x 1,048*	4.134
	40	371009	144	583.0	4.049	x 1,048*	4.243
	50	462998	180	727.5	4.042	x 1,048*	4.236

Table 5.7 Loads by Initial Skew; *average increase in weight

Model variant 2 (beams 1.6m & columns 0.8m)							
OC & HSC	levels	N [kN]	L [m]	H_i [kN]	q_i [kN/m]	**UHSC** [-]	q_i [kN/m]
	20	172402	72	270.9	3.763	x 1,046*	3.934
	25	216348	90	340.0	3.777	x 1,046*	3.950
	30	260456	108	409.3	3.790	x 1,046*	3.963
	35	308196	126	484.3	3.844	x 1,046*	4.019

Table 5.8 Loads by Initial Skew; *average increase in weight

The difference in the building's total weight between OC and HSC is negligibly small. However, the increase in weight in the case of UHSC cannot be neglected. Therefore, the load by initial skew is multiplied by a factor that represents the average increase in weight of the UHSC models.

The initial skew load is relatively small compared to the wind load. On average, the wind load is a factor eight to nine higher than the initial skew load.

5.3 Loading combinations

The loading combinations in Ultimate Limit State (ULS) and Serviceability Limit State (SLS) are as follows:

Ultimate Limit State	ULS1	ULS2	ULS3	ULS4	ULS5
Self-weight & Dead Load	1.2	1.2	1.2	1.35	0.9
Initial Skew Load	1.2	1.2	1.2	1.35	0.9
Live Load 2 floors	1.5	0.75*	0.75*	0.75*	0.75*
Live Load other floors	0.75*	0.75*	0.75*	0.75*	0.75*
Wind Load x-direction	0	1.5	0	0	0
Wind Load y-direction	0	0	1.5	0	1.5

Table 5.9 Loading Combinations in Ultimate Limit State (ULS) (*combination value Ψ_0 included)

Serviceability Limit State	SLS1	SLS2	SLS3
Self-weight & Dead Load	1.0	1.0	1.0
Live Load 2 floors	1.0	0.5	0.5
Live Load other floors	0.5	0.5	0.5
Wind Load x-direction	0	1.0	0
Wind Load y-direction	0	0	1.0

Table 5.10 Loading Combinations in Serviceability Limit State (SLS)

The loading combinations are determined by applying the recommendations in Eurocode 0 (6) chapter 6. The initial skew load is only taken into account when making a calculation in Ultimate Limit State, according to Eurocode 2 (3) art 5.2 rule 3. A variant of loading combination ULS5 where the wind load is applied in x-direction instead of the y-direction is not expected to give important or governing results and is therefore not taken into account.

6. Foundation

As discussed in chapter 4, all structural models rest on a foundation slab supported by piles. The foundation slab itself does not differ in any of the models. However, it is realistic to assume that a higher or heavier building requires a stronger or stiffer foundation. To make realistic calculations this has to be taken into account, otherwise calculation results of the forces or displacements involved or caused by the foundation are not realistic.

6.1 Geometry

Foundation slab

In the initial design stage, a foundation slab was used which had the same length and width as the typical floor plan. This slab did not provide sufficient rotational stiffness and therefore enlarged. The enlarged foundation slab is eventually used in all models.

The geometry of the foundation slab is fixed at a length of 43.2 metres and a width of 25.2 metres. This is slightly larger than the typical floor plan of 36.0 by 14.4 metres. By creating a larger foundation slab, more piles can be installed on the edges of the foundation slab. This way the rotational stiffness of the foundation is increased.

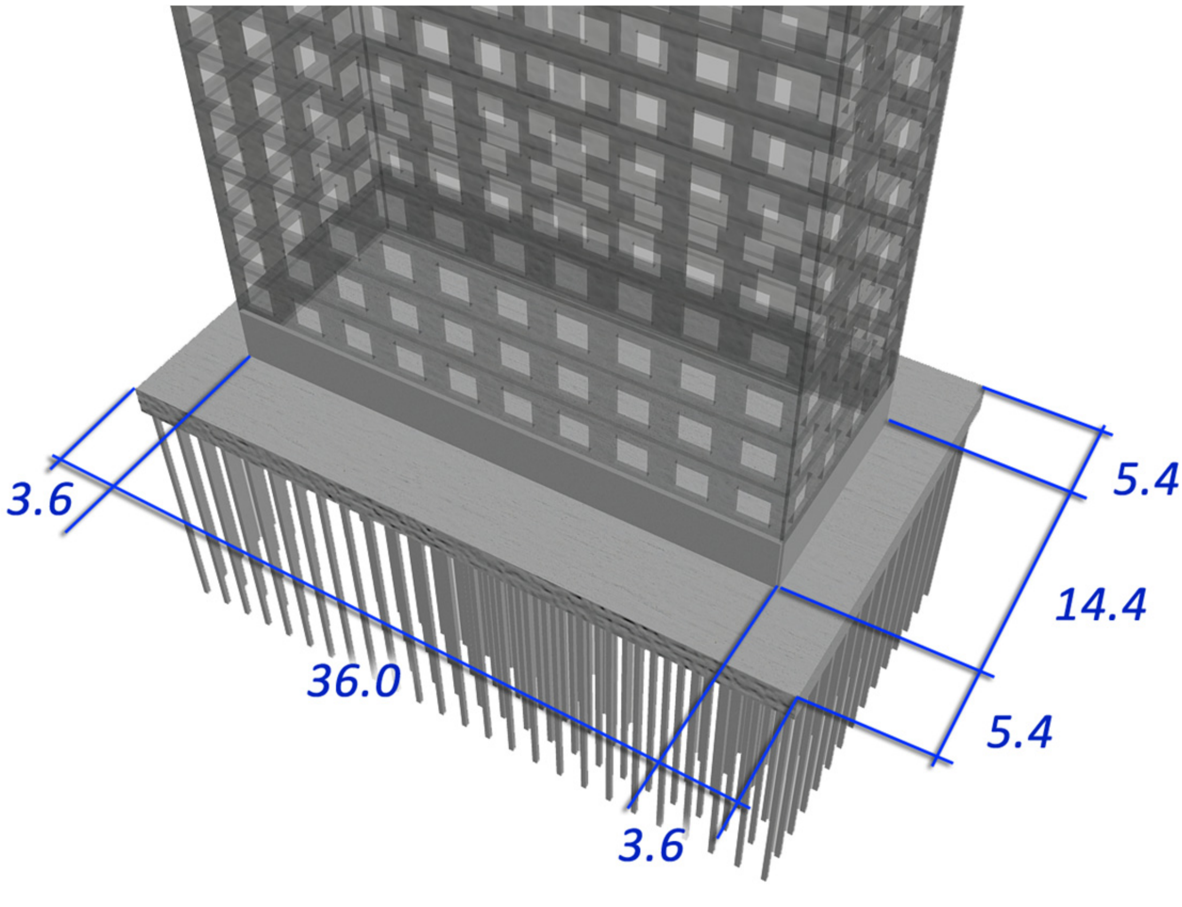


Figure 5.1 Foundation slab geometry

The thickness of the slab is chosen to measure 1.5 metres. Due to the placing of piles at the edges of the slab, larger bending moments and shear forces can occur in the foundation slab. The slab must be able to offer resistance to these moments and shear forces.

Concrete strength class C35/45 is used for the foundation slab. With the application of proper steel reinforcements, the moment resistance is found to be at around 15 MNm (8). This is sufficient to resist the moments found in the analysis of all structural models (see chapter 7) where the maximum moment found in the slab is at around 10 MNm. The shear force is absorbed without extra shear reinforcements due to the great thickness of the slab.

Piles

The used foundation piles are prefabricated piles with an area starting at 450x450 mm^2. Concrete with a strength class of C55/67 is applied and the maximum total length of the piles is set to 18 metres.

The piles are installed at a grid of 1.8x1.8 metres. In the centre of the foundation slab fewer piles are installed, otherwise it would result in too many piles. The design's goal was to create a foundation, which performs, as efficiently as possible. This means that most piles are loaded almost to their maximum capacity.

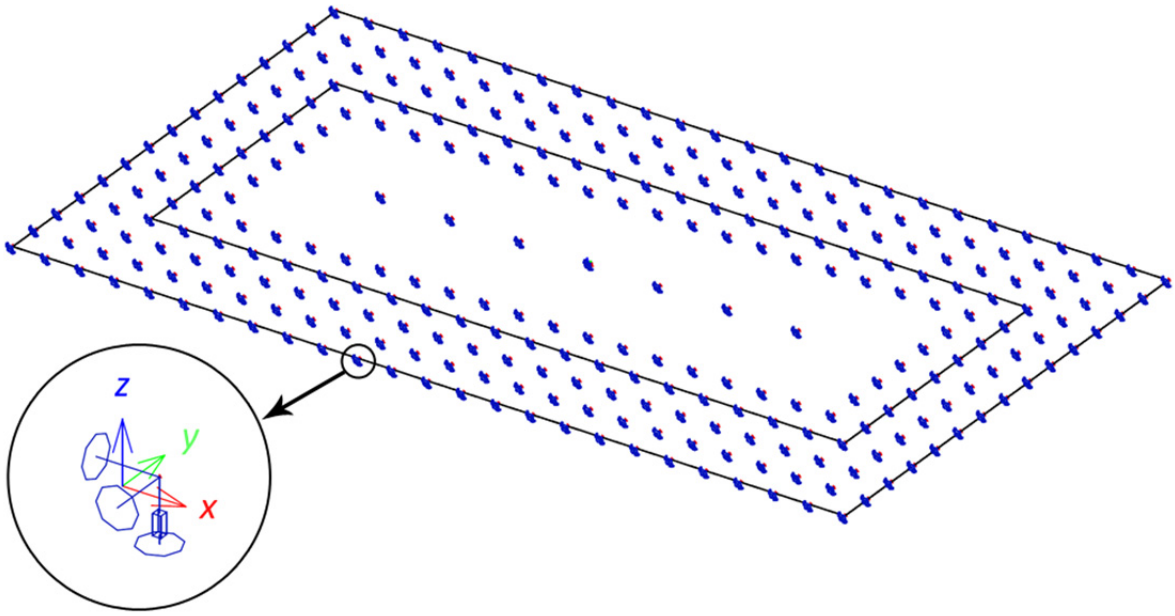


Figure 6.2 SCIA-Engineer Foundation Model

All supports are schematised as non-rotational stiff in all directions, rigid in the (horizontal) X- and Y-direction and flexible in the Z-direction. This flexibility is determined in the next section.

6.2 Stiffness

The rotational stiffness of the foundation depends on the stiffness of the slab and of the piles. The slab does not change in any model resulting in a fixed stiffness of the slab. To ensure that the model of a higher or heavier building has proper foundation stiffness, the pile stiffness is changed in each different model corresponding to the weight and height of the building.

The change in pile stiffness is mostly due to the fact of applying piles with a larger cross section. However, there is a limit to the pile's cross section. To ensure enough load-bearing capacity in the foundation more piles can be installed, which results in a smaller grid.

To change the grid of each model is very time-consuming. Therefore, only the stiffness of each support (pile) is changed in the various models. A result of this method is that there can be a model with very large piles in a grid of 1.8x1.8 metres. This will actually represents a foundation with smaller piles in a smaller grid, having the same total stiffness. This way the stiffness difference can be taken into account whilst it is not time-consuming. (In the situation with 40 levels, a pile-grid of piles 550x550 mm² with a centre-to-centre distance of 1.65 m is found. This distance is exactly the minimal recommended distance of three times the pile-diameter. In the situation with 50 levels, this centre-to-centre distance becomes 2.6 times the pile-diameter. This 50 level high model is however not thoroughly analysed as it exceeds the maximum lateral displacement at the top. The correction of the foundation slab to meet the minimal recommended centre-to-centre distance is therefore not executed in this thesis.)

To make the variation in support stiffness, the maximum stress of a pile is chosen to be at around 8 N/mm². With this fixed variable and the maximum reaction force, all of the other stiffness's can be determined.

The stiffness of a pile can be split into two parts: the pile stiffness and the soil stiffness. Due to the unknown soil stiffness, an approximation is used to determine the stiffness.

Pile stiffness

Hooke's Law gives:

$$\Delta l = \frac{F \cdot L}{EA}$$

The stiffness of a pile can be found by:

$$k_p = \frac{F}{\Delta l} = \frac{EA}{l}$$

While the area A differs in most models, the following variables are determined to be:

$$E_{c,d} = \frac{f_{c,d}}{\varepsilon_{c3}} = 22 \cdot 10^3 N/mm^2$$

$$l = 18000\ mm$$

Soil stiffness

A method to approximate the soil stiffness at the pile's end is described as follows (9):

$$k_g = 90 \cdot b \cdot \sigma_d$$

Where:

b is the width of the rectangular pile;
σ_d is the normal stress, which is set at 8 N/mm².

Total stiffness
The total stiffness becomes:

$$\frac{1}{k} = \frac{1}{k_g} + \frac{1}{k_p} \xrightarrow{yields} k = \left(\frac{1}{k_g} + \frac{1}{k_p}\right)^{-1}$$

The stiffness' of the support used in all the models is calculated and listed in table 6.1.

levels	Rz,max	b_{pile}	A_{pile}	σ_d	k_p	k_g	k
	[kN]	[mm]	[mm²]	[N/mm²]	[kN/mm]	[kN/mm]	[kN/mm]
20	1600	450	202500	7,90	250,0	324,0	141,1
25	1800	475	225625	7,98	278,5	342,0	153,5
30	2000	500	250000	8,00	308,6	360,0	166,2
35	2425	550	302500	8,02	373,5	396,0	192,2
40	2850	600	360000	7,92	444,4	432,0	219,1
50	3800	690*	476100*	7,98	587,8	496,8	269,2

Table 6.1 Support Stiffness (*change in foundation geometry or number of piles is recommended)

The maximum reaction force F_z is nearly the same in the models with large or small column sizes. Therefore, the support stiffness only changes with the height of the building.

Sample results

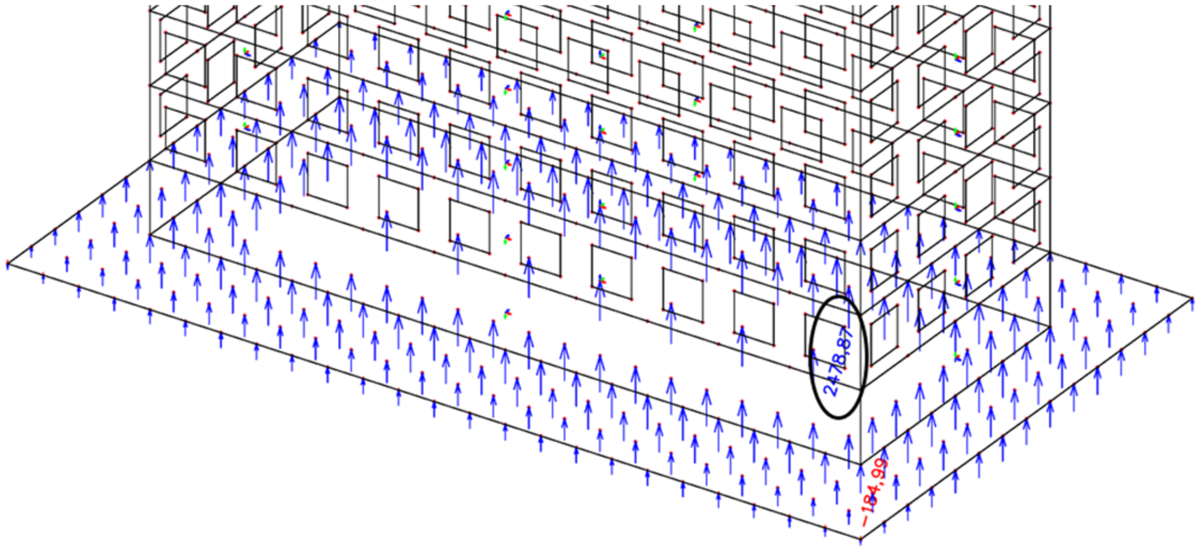


Figure 6.3 Maximum reaction forces in model with 35 levels loaded by the wind load.

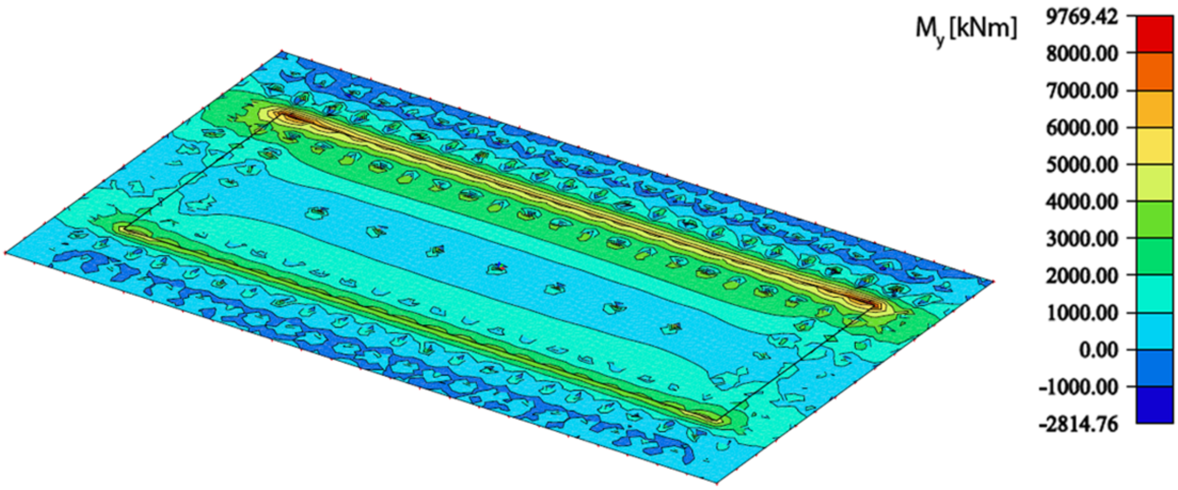


Figure 6.4 Maximum bending moments in foundation slab.

6.3 Second order factor

When the structure is loaded with a horizontal load, in this case the wind load, the foundation will undergo a slight rotation. Due to this rotation the structure will move, which will introduce second order loading. This second order factor has to be taken into account in SLS as well as ULS and is determined as follows.

$$\tan \varphi = \frac{u_{top}}{l}$$

$$\varphi = \frac{M_{rep}}{C} \Longrightarrow C = \frac{M_{rep}}{\varphi} = \frac{M_{rep}}{\tan^{-1}\left(\frac{u_{top}}{l}\right)}$$

The foundation's second order factor is now:

$$\frac{1}{n_f} \quad where\ n_f = \frac{2C}{Nl}$$

Where:

C is the rotational stiffness of the foundation;
N is the weight of the building;
l is the height of the building.

The second order factor of all models is calculated on a worksheet, together with the results of each model. See appendix C for this worksheet.

7. Load Bearing Structure

7.1 Geometry

The framed-tube structure is a system of columns and (spandrel) beams, which are repeated every 3.6 metres. The size of the spandrel beams remain the same in every model. While the beams do not differ in size, the columns do. In total, there will be two variants of columns, one with a column width of 1.6 metres and one with a column width of 0.8 metres.

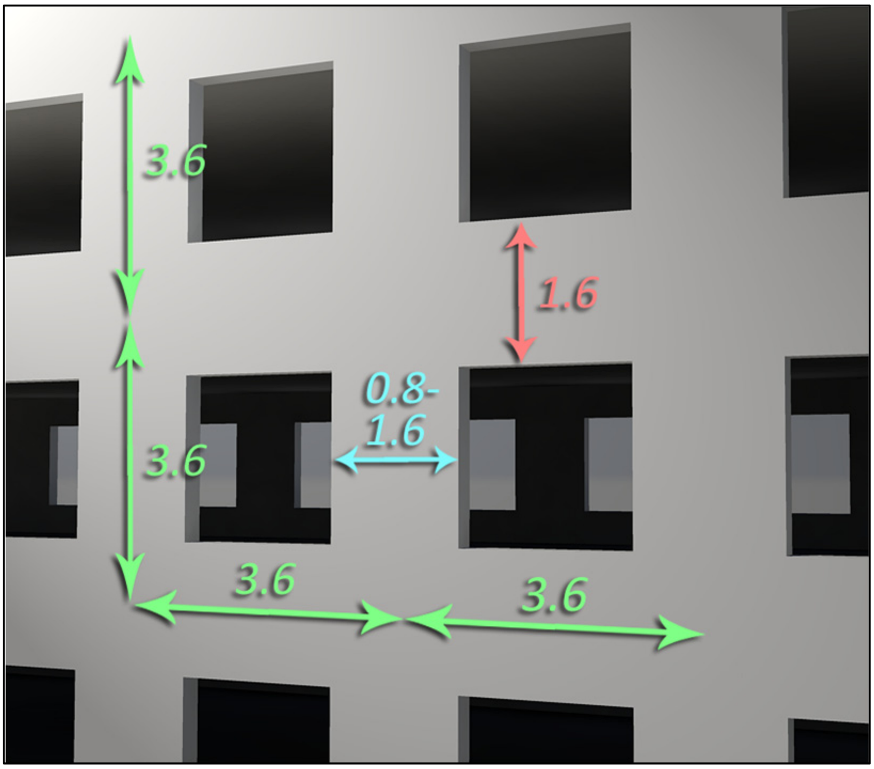


Figure 7.1 Geometry of the framed-tube structure

Due to the column width difference, the span of the spandrel beams increases. Overall, the variant with the smaller columns is more flexible. The variant with a column width of 1.6 metres is mentioned as “Variant one” and the variant with a column width of 0.8 metres is mentioned as “Variant two”.

7.2 Materials

Each model is analysed by using the three different concrete types. This way the differences between the models can be found and compared. As discussed in chapter 3, three concrete types are used and are listed in table 7.1.

Concrete Type	Strength class as used in this thesis
Ordinary Concrete (OC)	C35/45
High Strength Concrete (HSC)	C90/105
Ultra High Strength Concrete (UHSC)	C180/200

Table 7.1 Concrete Types

The Young's Modulus

The model's stiffness is a major aspect when it comes to the behaviour of the building. While the second moment of area (moment of inertia) depends on the geometry of the structure, the modulus of elasticity depends on the applied material. The total stiffness depends solely on these two variables. To determine the modulus of elasticity of each concrete type the following approach is used.

$$E_c = \frac{f_{cd}}{\varepsilon_{c3}}$$

It is noted that the concrete stress-strain relationship from this equation refers to the ULS design. It is strictly speaking not applicable in a SLS check. Since concrete is usually in the linear elastic stage in SLS, the only relevant concrete property in SLS design is the Young's modulus. This modulus can be read from EN 1992-1-1 table 3.1 (3). However, this table presents a modulus related to short term loading only. Long term loading can be incorporated by taking into account the creep coefficient. However, creep is often difficult to quantify precisely. This is partly caused by the difficulties encountered when predicting the loading history (short- and long-term load components and their duration). Therefore, in practice often estimated values are used (10). This is also done in this analysis, where the Young's modulus of concrete is derived from the ULS stress-strain diagram.

This approach results in the following Young's moduli:

Concrete type	f_{cd}	ε_{c3}	E_c
	[N/mm²]	[‰]	[N/mm²]
C35/45 (OC)	23.3	1.75	13×10^3
C90/105 (HSC)	60.0	2.30	26×10^3
C180/200 (UHSC)	120.0	3.00	40×10^3

Table 7.2 Modulus of Elasticity

In example, EN 1992-1-1 table 3.1 (3) shows $E_c = 34 \cdot 10^3$ N/mm^2 for C35/45. This implies that implicitly a creep coefficient of 34 / 13.3 - 1 = 1.6 is included in the analysis. This approach gives a decent estimated values, a precise calculation should reveal the exact magnitude of the creep coefficient.

The Effective Young's Modulus

Not all elements are subjected to a long-term load. The spandrel beams in the structure are expected to have a smaller creep effect. However, these elements, in contrast to the columns, will have cracks, caused by the large bending moments. Consequently, the effective Young's Modulus is reduced by the cracks and the creep-effect, and is expected to be near the same value as calculated above.

Eurocode 2 (3) does not provide a method to determine the effective Young's Modulus. To be able to determine the effective Young's Modulus a method is used as is found in the previous Dutch code "VBC 1995" (11), table 15. As an example, the spandrel beam in the OC model with 30 levels is used (see chapter 7).

$$\alpha_n = \frac{N'_d}{A_b f'_b + (A_s + A'_s) f_s} = 0.102;$$

$$\bar{\omega}_{0t} = \frac{A_s + A'_s}{A_b} \cdot 100 = 1.056\%;$$
$$E_f = 2200 + 4400\bar{\omega}_{0t} + (24000 - 2200\bar{\omega}_{0t})\alpha_n = 13791\ N/mm^2 = 14 \cdot 10^3 N/mm^2$$

As expected, the result of the Effective Young's Modulus is nearly the same as was calculated in table 6.2. Therefore, the same Young's Modulus is used in the entire model.

Determination method

The method used to determine the Young's Modulus could have major influence on the total behaviour of the structure. A good representation of the actual structure requires a good substantiated method in determining the Young's Modulus. Eventually, the acquired Young's Modulus is used to analyse the structure in the design phase.

To determine the exact Young's Modulus of the structure, an in-depth analysis is required. A method to determine the exact Young's Modulus of the structure is to calculate several M-N-Kappa diagrams. The diagrams are calculated at certain heights of the structure, resulting in different stiffness's at each different section. Eventually, these stiffness's are taken into account in the analysis. A number of iterations may be required before the best model of the structure is acquired. Obviously, this method is not suitable for use in an analysis like performed in this thesis due to the large number of models.

The Young's Modulus as used in this thesis is an accurate estimation of the exact value. While it is not exact, the determination method does not have major influences on the thesis's results. The reason for this is that the used method is consistent throughout the analysis.

7.3 FEM Model

The software used to analyse the structure is Nemetschek SCIA Engineer, previously known as SCIA ESA PT. The model is built-up from several 2D-elements with a certain thickness placed in a XYZ-plane, resulting in a 3D-model.

The façade elements are 2D-elements with a thickness of 350 mm. The window elements in the façade are inserted as 2D-openings in the façade elements. The mechanical properties of the elements are specified and the self-weight of the façade elements is automatically calculated and taken into account in a separate load case.

The floor elements in the structure ensure proper horizontal load distribution in the façade and enhance the structural integrity. In the FEM model, these floor elements are schematised as plates, supported onto the façade by nodes that allow rotations. The weight of the floor elements is set to zero as the weight and the loading of the floor levels is directly applied on the façade elements, see chapter 5.1.

For the FEM analysis, the elements need to be meshed. The maximum mesh-size is set to 0.5x0.5 m^2, where needed this size is decreased (e.g. at the corners near an opening). The model has been analysed with smaller mesh-sizes, but the chosen mesh-size shows to be accurate enough whilst maintaining limited calculation/processing time.

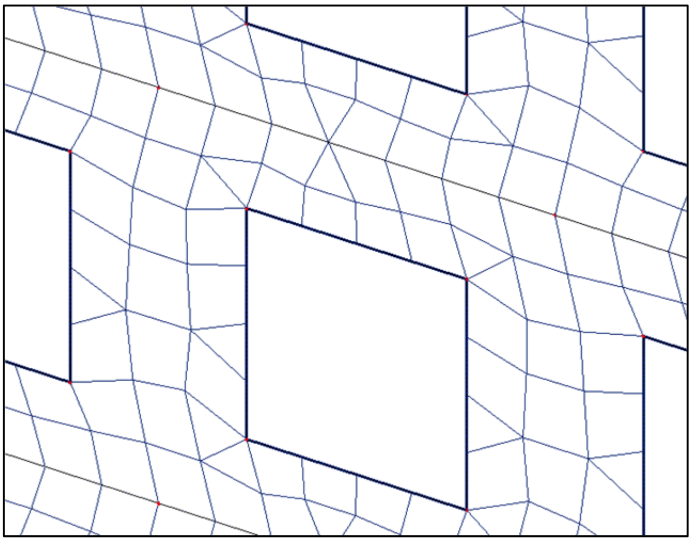


Figure 7.2 Mesh Generated in FEM Model

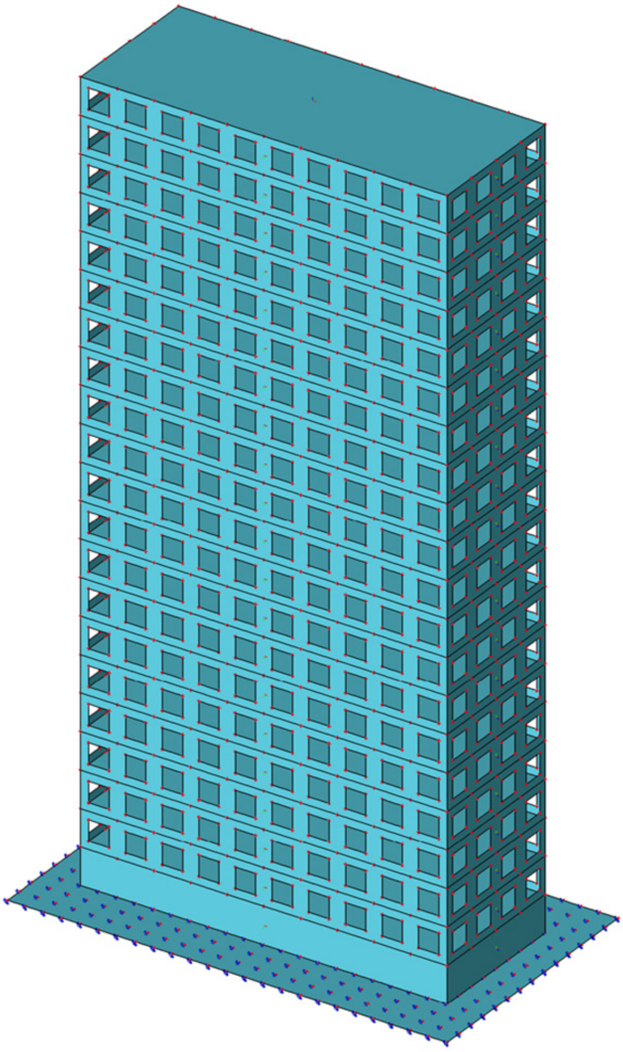


Figure 7.3 Full 20 level FEM Model

The displacements of the model can directly be obtained from the model, which shows a clear picture of the obtained displacements. However, not only the displacements of the building are relevant to analyse, the forces in the elements also play an important role in the analysis.

1D-elements
In the Faessen's thesis (2) it is made clear that the greatest forces in a framed-tube structure are found in the corner columns and in the lowest spandrel beams of the short-side façade. To acquire these forces the FEM model needs to be changed at the elements of interest. Instead of leaving it as a 2D-element, it is converted to a 1D-element.

The 1D-bar has different connecting points than the real 3D-element would have. This would result in inaccuracy and high peak forces, which should not be there. Therefore, the 1D-bar is connected to the rest of the elements by unlimitedly stiff 1D-elements (∞EI), which span exactly the length where the element of interest would have its connection. Figures 6.4, 6.5 & 6.6 illustrate this. The loads from the adjacent 2D-element are absorbed by the infinitely stiff 1D-element and transferred to the beam or column of interest (also modelled as a 1D-element). In this way the flow of forces stay the same and unrealistic peak forces are not present.

The stiffness of the 1D-elements, which represent the column or beam, have exactly the same stiffness as the element should have. Therefore, no significant changes in the behaviour of the model are expected.

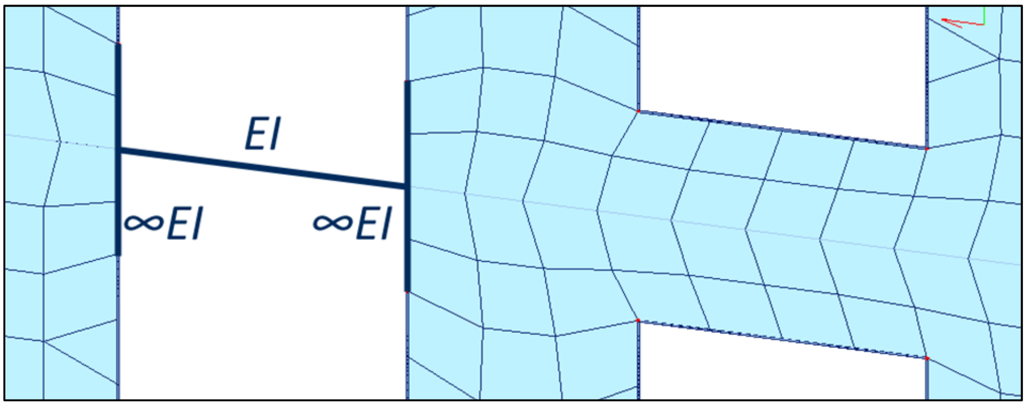


Figure 7.4 Spandrel Beam as 2D-element (right) and as 1D-bars (left)

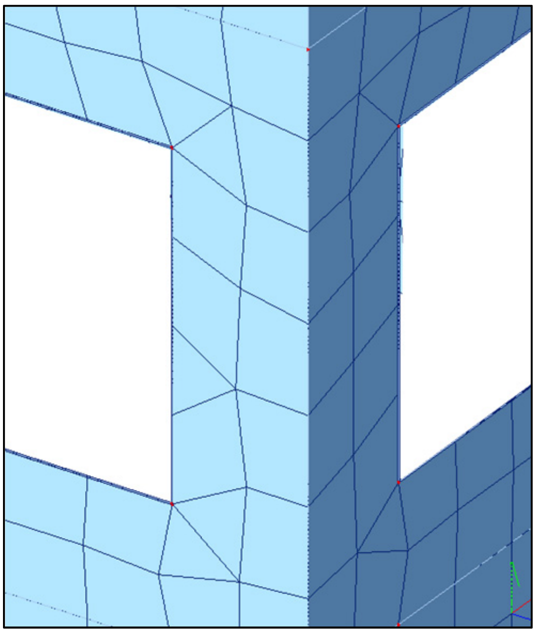


Figure 7.5 Corner Column: 2D-element

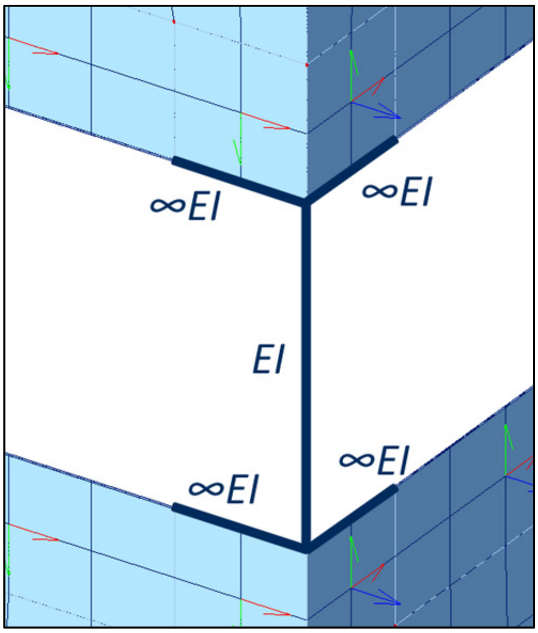


Figure 7.6 Corner Column: 2D-element transformed to 1D-bars

During the analysis, the model with the 1D-bars instead of full 2D-elements shows no difference and behaves the same. Therefore, accurate results can be acquired from the model.

7.4 Behaviour

One of the goals of this thesis is to research the possibility of achieving a greater building height while maintaining the same geometry and element sizes and still meet the requirement of maximum lateral deflection. Another goal was to compare the UHSC model with similar OC models.

To achieve these goals several models are analysed. The differences between the models can be found in element sizes, building height, and applied concrete type. In total, 19 models have been analysed and the results are listed in a worksheet, see appendix C.

The height of an OC model is limited at a certain height (due to the lateral deflection). To make comparison possible, this model with its maximum height is also analysed in HSC and UHSC. An overview of the analysed models is listed below.

Number of levels	OC	HSC	UHSC	**Number of levels**	OC	HSC	UHSC
Geometry variant 1	20	20	20	Geometry variant 2	20	20	20
(columns 1.6m)	30	30	30	(columns 0.8m)	25	25	
		35*				30	30
	40	40	40				35
			50				

Table 7.3 Analysed Models (* analysed in order to find the optimum height of the HSC model)

7.4.1 Second Order Effect

The results given by the FEM analysis are the first order results. To consider the second order effect, these results need to be multiplied by the second order factor. The second order factor of the foundation is already calculated in section 6.2. The second order factor of the structure itself is calculated below. In the end, these factors are combined to create the total second order factor.

The analytical approach to determine the maximum displacement at the top is as follows:

$$u_{top} = \frac{q \cdot l^4}{8EI_{str}} \Longrightarrow EI_{str} = \frac{q \cdot l^4}{8 \cdot u_{top}}$$

Where:

EI_{str} is the stiffness of the structure;
l is the height of the structure.

The second order factor of the structure is then (12) (derived from Euler's critical buckling force):

$$\frac{1}{n_s} \; where \; n_s = 7.8 \cdot \frac{EI_{str}}{N \cdot l^2}$$

The total second order factor is now:

$$\frac{1}{n} = \frac{1}{n_f} + \frac{1}{n_s} \xrightarrow{yields} n = \left(\frac{1}{n_f} + \frac{1}{n_s}\right)^{-1}$$

The second order factors are all directly calculated in the worksheet with the corresponding structural model and limit state (ULS or SLS), see appendix C.

7.4.2 Displacements

The lateral displacements of the structure are both in x-direction as well as in y-direction. However, the loading in x-direction is not significant due to the smaller loading area (less wide façade). The lateral displacements in x-direction are noticeably smaller than in y-direction, which consequently is the governing direction.

A summary of the maximum lateral displacements of each analysed model with its unity checks is listed in table 7.4. The displacements as listed in the table are the end result, including the second order factor for Serviceability Limit State (SLS).

Note: The maximum allowable displacement u_{max} is determined using the structural height, thus including the basement level.

Geometry variant one (Column width: 1600 mm)

OC levels	h [m]	u_{max} [mm]	u_y [mm] (End result)	u.c.
20	72	151.2	52.1	0.34
30	108	223.2	206.2	0.92
40	144	295.2	641.1	2.17

HSC levels	h [m]	u_{max} [mm]	u_y [mm] (End result)	u.c.
20	72	151.2	30.2	0.20
30	108	223.2	111.3	0.50
35	126	259.2	198.8	0.77
40	144	295.2	323.9	1.10

UHSC levels	h [m]	u_{max} [mm]	u_y [mm] (End result)	u.c.
20	72	151.2	22.4	0.15
30	108	223.2	78.4	0.35
40	144	295.2	220.1	0.75
50	180	367.2	556.2	1.51

Geometry variant two (Column width: 800 mm)

OC levels	h [m]	u_{max} [mm]	u_y [mm] (End result)	u.c.
20	72	151.2	111.4	0.74
25	90	187.2	235.4	1.26

HSC levels	h [m]	u_{max} [mm]	u_y [mm] (End result)	u.c.
20	72	151.2	60.3	0.40
25	90	187.2	124.2	0.66
30	108	223.2	235.4	1.05

UHSC levels	h [m]	u_{max} [mm]	u_y [mm] (End result)	u.c.
20	72	151.2	42.0	0.28
30	108	223.2	158.3	0.71
35	126	259.2	271.1	1.05

Table 7.4 Lateral displacements of all models

One can see that variant two performs significantly poorer than variant one. This is due to the decrease in stiffness of the total structure by applying smaller column widths. Where in variant one of the OC model a height of 30 levels is achieved, this is only limited at 20 levels in variant two. This difference of 10 levels can also be recognised in the HSC and UHSC models.

Variant one

If only one variant is analysed one can see improved performance when applying higher strength concrete. For example, in variant one at 30 levels, the OC model shows a displacement of 206.2 mm. The displacement of the HSC model is reduced by 43.4% to 116.7 mm. The UHSC model performs even better with a displacement of 78.4 mm, a reduction of 62.0% compared to the OC model and 32.8% compared to the HSC model.

The maximum height improves as well in the case of applying a higher strength concrete: 5 levels extra for the HSC model and 10 levels extra for the UHSC model. This is an increase of respectively 17% and 33%.

Variant two

The same improvements are recognised in variant two. At a height of 20 levels, the maximum height of the OC model, the HSC model shows a displacement of 62.6 mm where the OC has a displacement of 111.4 mm, a reduction of 43.8%. The UHSC model shows a displacement of 42.0 mm, which is a reduction of 62.2% (OC) and 32.9% (HSC).

The main reason for having fewer displacements is the difference in the modulus of elasticity of each concrete type. The modulus of elasticity of OC is 48.9% lower than that of HSC, which is comparable to the reduction of 43.4% and 43.8% in displacements. The same is true for UHSC: the modulus of elasticity of OC is 66.7% lower, which is, again, comparable with the reductions of 62.0% and 62.2%.

There is still a small difference in the reductions (4.7%-5.5%). This is explained by the fact that a part of the displacement is due to the foundation's stiffness.

Rules of Thumb Comparison

In an earlier thesis by Faessen (2) several rules of thumbs were derived, see appendix H. The thesis focuses on framed tubular structures with various dimensions. The rules of thumb give estimations on displacements and forces in the critical elements of the structure. The rules of thumb are applied to each model. Eventually, comparison was made between the results of the rules of thumb and the FEM analysis.

Comparison FEM analysis and Rules of Thumb				
		Lateral Displace-ment at the Top	Shear Force in Spandrel Beam*	Normal Force in Corner Column**
	levels	Δ	Δ	Δ
OC	20	-13.1%	-20.3%	-14.4%
OC	30	18.9%	-18.7%	0.8%
HSC	30	6.2%	-15.0%	-0.1%
HSC	35	22.5%	-15.1%	-12.0%
UHSC	40	68.7%	-11.3%	-4.9%

Table 7.5 Variant 1 results of the comparison FEM analysis and Rules of Thumb. (Rules of Thumb as base values)

* Spandrel beam is located at the first floor in the 14.4 metres wide façade (short side).

** Corner Column is located on the compression side of the structure between ground level and first floor.

One can see that the results vary from the FEM analysis. There are several reasons for this. The first one is that the rules of thumb never were designed to be used with higher strength concretes. The HSC and UHSC results show significant difference compared to the FEM analysis results. Another reason for the difference is that the loading in the rules of thumb is applied as a linear line load, while in the FEM analysis a trapezium-like line loading is applied. Yet another factor is that the Young's Modulus is not taken into account in the determination of the shear and normal forces. The different Young's Modulus does have an influence on the forces in the structure (see section 6.4.4).

To make sure that the rules of thumb are applicable when applying higher strength concretes, the following steps are recommended:

1. Introduce a factor in the displacement calculation to take into account the different behaviour of concrete mixtures with high Young's moduli in tubular structures.
2. Take into account the concrete's stiffness (Young's Modulus) in determining the forces in the structure.

Overall, except for three situations, the difference seems to be no greater than 20%. This difference is acceptable when designing a framed-tubular structure with Ordinary Concretes, as the rules of thumb were designed to provide an approximation in the design stage. However, the rules of thumb are not recommended for application with Higher Strength Concretes as the results become less accurate.

7.4.3 Internal Forces

The internal forces in the critical elements are obtained by the output of the FEM analysis. For each model the normal forces, shear forces and the bending moments are noted and listed in a worksheet, see Appendix D and E. The highest forces or combination of forces in each element is listed. In most cases, this was found in the loading combinations ULS3 and ULS4.

Spandrel Beams

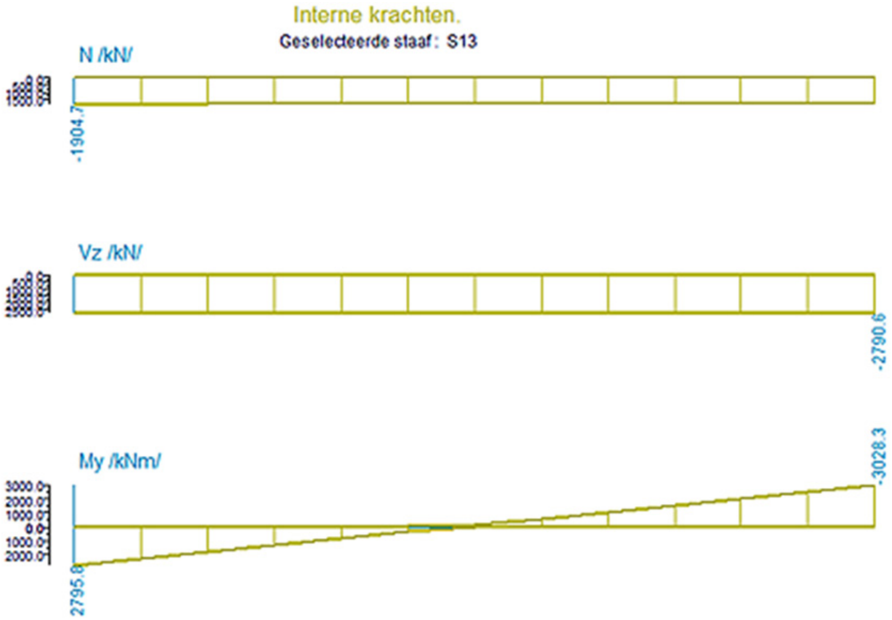


Figure 7.7 Output internal forces 1D-element (Spandrel beam)

As can be seen in figure 7.7, very high shear forces go through the spandrel beams. This is exactly what was expected and is a typical behaviour of the framed-tube structure.

In a comparison of the forces in the spandrel beams between variant one and two, one can see that the shear force is reduced, while the maximum moment increases. The reason for this is found in the longer element length due to the smaller column sizes.

Corner Columns

The corner columns of the structure are loaded with a very high normal force. This force is significantly larger than the other columns due to the shear-lag effect. This subject is discussed in section 7.4.4.

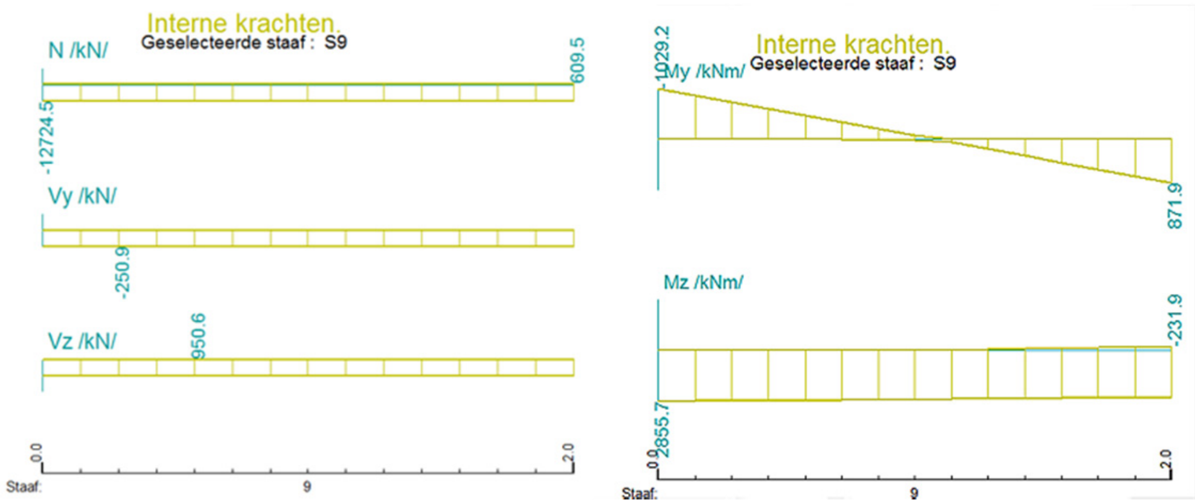


Figure 7.8 Output internal forces 1D-element (Corner column)

There is some difference in forces and moments when comparing the models with the same height. The higher mass density of the higher strength concrete, which makes the building heavier, mainly causes this difference.

These columns are located at every corner of the structure. Because of the horizontal loading, a bending moment will be present at the bottom of the structure. Consequently, the columns are located at either a compressive side or a tension side.

In most loading combinations, no tensile forces are found in the columns. The high self-weight of the building is the main reason for that. However, in loading combination ULS5, tensile forces occur in the corner columns. The tensile forces have a negative impact on the moment resistance of the columns, but the magnitude of the tensile force is not that high that it will cause heavy reinforcements, see chapter 8.

7.4.4 Shear-Lag

The shear-lag effect is still present in all of the analysed models. However, the applied concrete type shows a difference in the magnitude of the shear-lag effect. The reason for this is the stiffness of the material. Higher strength concretes deliver stiffer spandrel beams, therefore the columns in the façade are more evenly loaded, meaning less shear-lag.

Figure 7.9 shows three models with a height of 30 levels. In each of the three, a different concrete type is used.

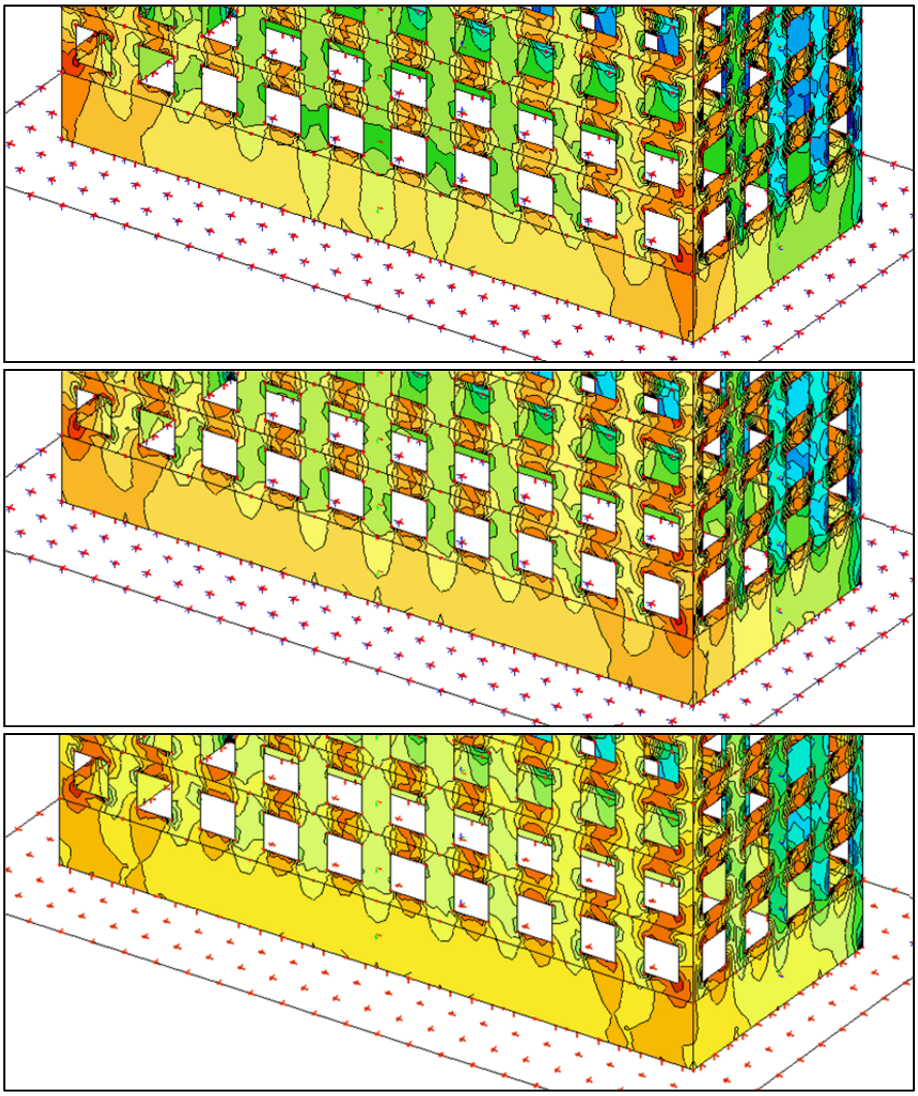
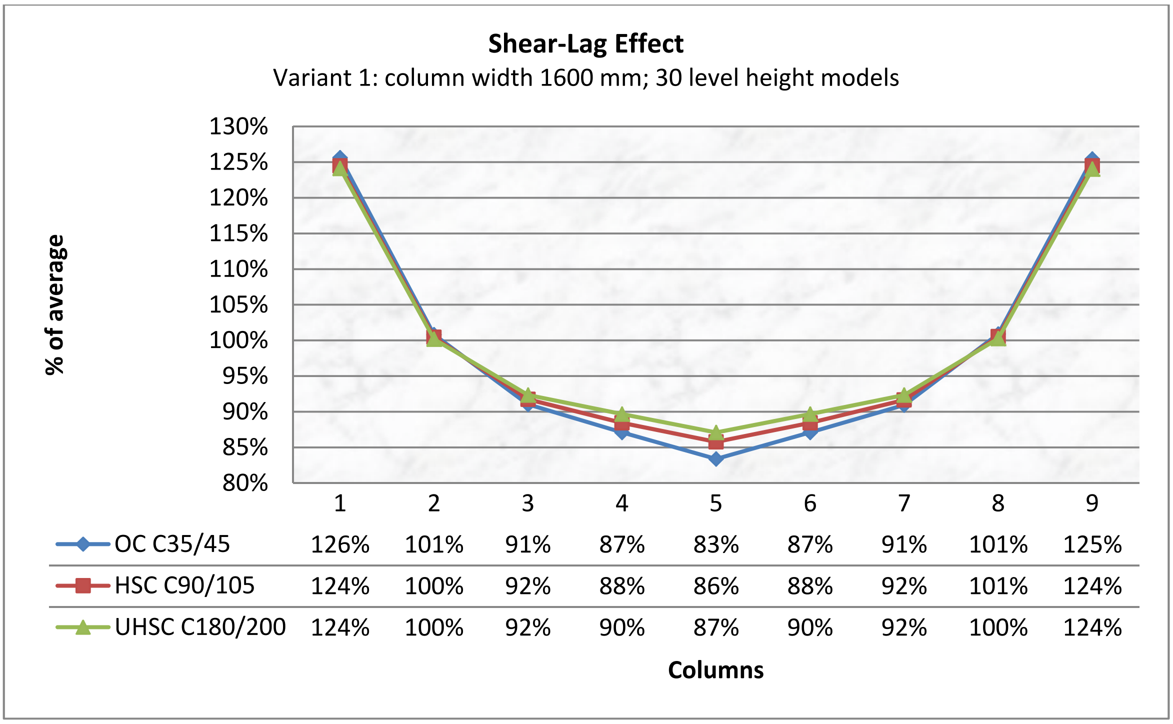


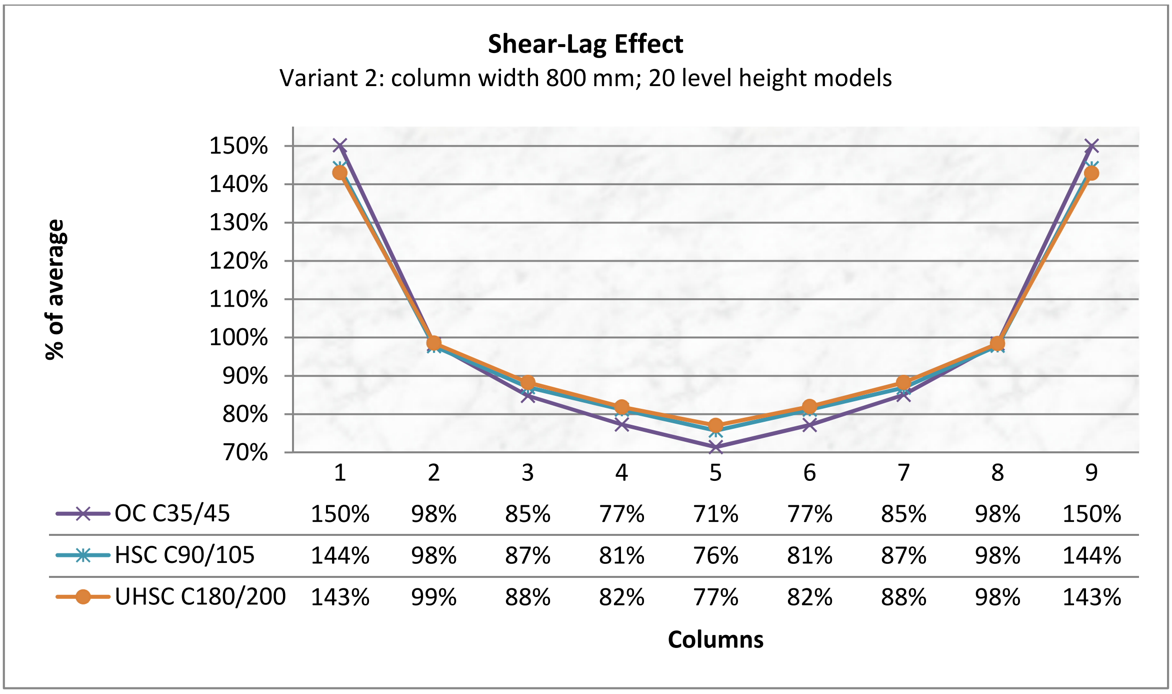
Figure 7.9 Shear-lag Effect: Normal force in the OC model (top), HSC model (middle) and UHSC model (bottom)

In figure 7.9, the colour represents the magnitude of the normal force. In the figure, red represents a high normal force and green represents a low normal force. The focus goes to the lower part of the structure. One can see that in the OC model less normal force is present in the centre of the structure than is the case at the corners. A reduction of this effect is found in the UHSC model.

To know the exact reduction of the shear-lag effect, the difference in normal force of each column compared to the average is calculated. Graph 7.1 and 7.2 show the results.

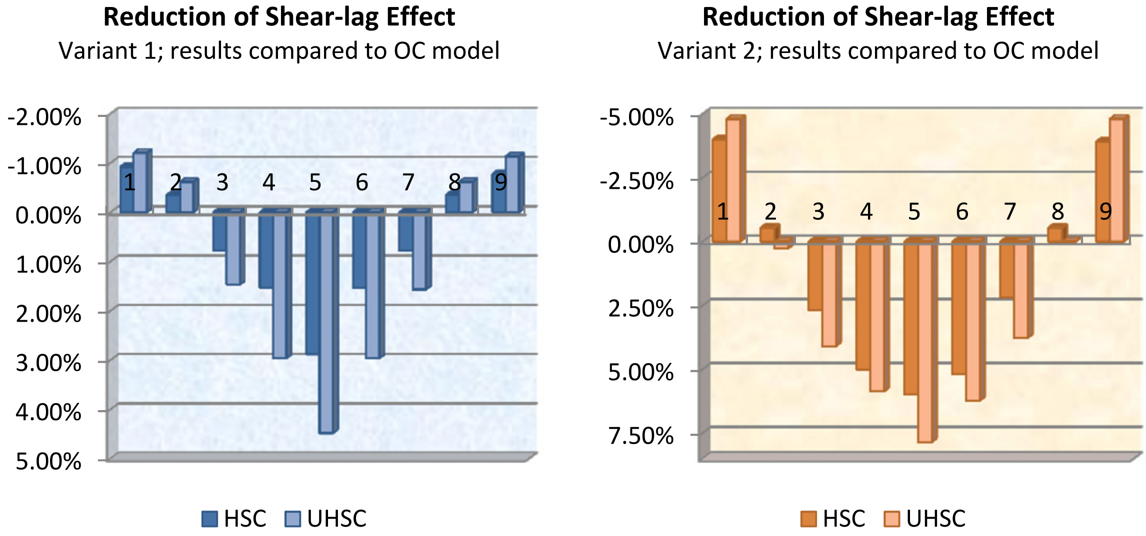


Graph 7.1 Shear-lag Effect in variant 1



Graph 7.2 Shear-lag Effect in variant 2

The results show a reduction in the shear-lag effect when applying higher strength concretes. The magnitude of the shear-lag effect is higher in variant two due to the less stiff structure. The reduction of the shear-lag effect is an advantage of the use of higher strength concretes, but the magnitude of this reduction remains very little in both variants. It is questionable whether this small advantage could be a decision-making argument in applying a higher strength concrete.



Graph 7.3 & 7.4 Reduction of Shear-lag Effect both variants

The reduction of the shear-lag effect is not linear to the increase of the Young's Modulus. The UHSC model shows little difference when compared to the HSC model. The reason for this is that the analysed models have the same height, but are not at its ultimate (displacements) limits. Therefore, the differences between the two models stay little.

7.4.5 Building's Natural Frequency

The building's natural frequency is the important measurement to ensure whether the building's comfort is still within an acceptable range. The used software for the FEM analysis is able to execute a dynamic calculation. The result is that the building's natural frequency [f_e] and natural period [T] are acquired.

The results of the model considering the natural frequency and period are also listed in appendix C. A summary is given in table 7.5.

In general, the more stiff the structure gets, the higher the frequency. This is confirmed in the results. A model with the same geometry has a higher natural frequency when a higher strength concrete is applied. The frequencies of variant two are lower due to the less stiff structure.

The role of comfort becomes more important as buildings have greater heights. The dynamic behaviour of a building could become the governing factor instead of the strength and lateral displacement. Recommendations found in the code (7) ensure a building has the required comfort level. Two variables are linked with this comfort level: the building's natural frequency and its acceleration. The limit of the combination of the two variables is set in the codes and is based on several tests.

Each model in this thesis is checked whether it fulfils the comfort requirements. The checks are done by following the "NTA Hoogbouw (03-A)"-report (13). This report is based on Eurocode 1 (7) & "CUR Aanbeveling 103" (14).

Geometry variant 1				Geometry variant 2			
OC				**OC**			
levels	h [m]	f_e [Hz]	T [sec]	levels	h [m]	f_e [Hz]	T [sec]
20	72	0,85	1,18	20	72	0,64	1,56
30	108	0,46	2,16	25	90	0,47	2,14
40	144	0,29	3,47	30	108	0,35	2,82
50	180	0,19	5,14				
HSC				**HSC**			
levels	h [m]	f_e [Hz]	T [sec]	levels	h [m]	f_e [Hz]	T [sec]
20	72	1,10	0,91	20	72	0,86	1,16
30	108	0,62	1,62	25	90	0,63	1,60
35	126	0,48	2,06	30	108	0,48	2,09
40	144	0,39	2,57				
50	180	0,26	3,78				
UHSC				**UHSC**			
levels	h [m]	f_e [Hz]	T [sec]	levels	h [m]	f_e [Hz]	T [sec]
20	72	1,29	0,77	20	72	0,93	1,08
30	108	0,66	1,51	30	108	0,52	1,93
40	144	0,42	2,37	35	126	0,41	2,44
50	180	0,29	3,46				

Table 7.5 Natural Frequencies and Periods

The comfort criteria are expressed in terms of peak accelerations. This peak acceleration is found in the NEN-EN 1991-1-4 (7) as formula B.4:

$$\hat{a} = k_p \times \sigma_a$$

A conservative estimation of the peak acceleration is found by applying $k_p = 4$. The other factor σ_a is found by (13):

$$\sigma_a = c_f \times \rho \times l_v(z_s) \times v_m^2(z_s) \times R \times \frac{K_y \times K_z}{\mu_{ref}} \times \frac{\Phi(y,z)}{\Phi_{\max}}$$

where:

$$\frac{\Phi(y,z)}{\Phi_{\max}} = 1$$

μ_{ref} is the mass of the building per unit area

The other variables have been determined one by one:

Variable c_f (NTA Hoogbouw 03-A.6.1 (13))

$$d/b = 36/14.4 = 2.5 \xrightarrow{yields} c_f = 1.075$$

Variable ρ (NEN-EN 1991-1-4 (7))

$$\rho = 1.25\ kg/m^3$$

Variable $l_v(z_s)$ (NEN-EN 1991-1-4 (7))

$$l_v(z_s) = \frac{k_l}{c_0(z) \cdot \ln\left(\frac{z}{z_0}\right)} \xrightarrow{in\ this\ case\ becomes} \frac{1.0}{1.0 \cdot \ln\left(\frac{z}{z_0}\right)}$$

Variable $v_m^2(z_s)$ (NEN-EN 1991-1-4 (7) & NTA Hoogbouw 03-A.8.1 (13))

$$v_m(z_s) = c_r \times c_0 \times v_b = 1.0 \times 1.0 \times 19.4 = 19.4\ m/s$$

Variable R (NEN-EN 1991-1-4 (7) & NTA Hoogbouw 03-A.7.2 (13))

$$R^2 = \frac{\pi^2}{2 \cdot \delta_s} \cdot S_L(z_s, n_{1,x}) \cdot K_s(n_{1,x})$$

where:

$$\delta_s = 0.10$$

$$S_L(z_s, n_{1,x}) = \frac{6.8 \cdot f_L(z_s, n_{1,x})}{\left(1 + 10.2 \cdot f_L(z_s, n_{1,x})\right)^{5/3}}$$

$$f_L(z_s, n_{1,x}) = \frac{n \cdot L(z_s)}{v_m(z_s)}$$

$L(z_s) = L_t \cdot \left(\frac{z_s}{z_t}\right)^{\alpha}$ with $\alpha = 0.67 + 0.05 \ln z_0$; $L_t = 300;\ z_t = 200;\ z_0 = 0.5$

n is the natural frequency

$$K_s(n_{1,x}) = \frac{1}{1 + \sqrt{(G_y \cdot \phi_y)^2 + (G_z \cdot \phi_z)^2 + \left(\frac{2}{\pi} \cdot G_y \cdot \phi_y \cdot G_z \cdot \phi_z\right)^2}}$$

$$\phi_y = \frac{c_y \cdot b \cdot n}{v_m(z_s)} = \frac{11.5 \cdot b \cdot n}{v_m(z_s)}$$

$$\phi_z = \frac{c_z \cdot b \cdot n}{v_m(z_s)} = \frac{11.5 \cdot b \cdot n}{v_m(z_s)}$$

$$G_y = 1/2$$

$$G_z = 5/18$$

Variable $K_y \times K_z$ (NTA Hoogbouw 03-A.7.2.1 (13))

$$K_y = 1.0;\; K_z = \frac{5}{3}$$

The founded peak acceleration of the building needs to be lower than the limit provided in the code (13). This limit is linked to the building's natural frequency, see figure 6.10.

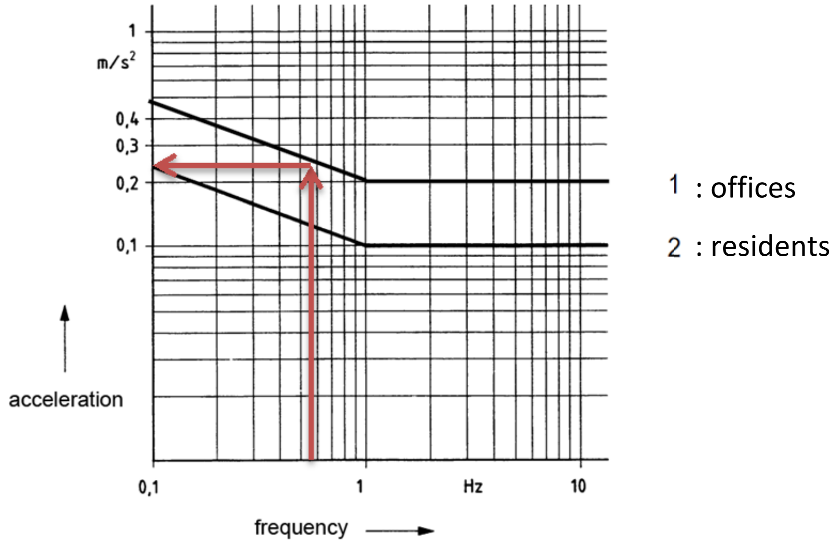


Figure 7.10 Comfort: Maximum Peak Acceleration (Line 1: offices, Line2: residents) (13)

The calculation of the peak acceleration and the corresponding limit is processed in a worksheet, see appendix G. Some results of the calculation are displayed below in table 7.6.

	c_f	ρ	$I_v(z_s)$	$v^2_m(z_s)$	f_e	R	K_y	K_z	μ_{ref}	σ_a	k_p	$\hat{a}$	$\hat{a}_{max}$	u.c.
	[-]	[kg/m³]	[-]	[-]	[Hz]	[-]	[-]	[-]	[kN/m²]	[-]	[-]	[m/s²]	[m/s²]	[-]
30 levels														
OC	1.075	1.25	0.0930	376.4	0.46	0.265	1.00	1.67	549.3	0.038	4.0	**0.151**	0.27	0.56
HSC	1.075	1.25	0.0930	376.4	0.60	0.196	1.00	1.67	549.7	0.028	4.0	**0.112**	0.25	0.45
UHSC	1.075	1.25	0.0930	376.4	0.68	0.169	1.00	1.67	568.8	0.023	4.0	**0.093**	0.24	0.39
35 levels														
HSC	1.075	1.25	0.0904	376.4	0.48	0.245	1.00	1.67	627.7	0.030	4.0	**0.119**	0.27	0.44
40 levels														
UHSC	1.075	1.25	0.0883	376.4	0.43	0.269	1.00	1.67	733.8	0.027	4.0	**0.109**	0.28	0.39

Table 7.6 Peak Acceleration Check

The comfort level check requires many variables and functions. In general, structural engineers are not completely familiar with all the variables and functions. However, the results acquired from the check correspond to the expected value for a typical concrete structure. The high self-weight of the concrete structure reduces the maximum acceleration. Consequently, the building provides a high comfort level. As expected, all analysed models meet the comfort requirement.

8. Material Verification

While the requirement to maximum displacement is already discussed in chapter 6, the requirement to strength still needs to be verified.

8.1 Design Verification Methods

8.1.1 Ultimate Limit State

In the ultimate limit state, safety factors have to be taken into account. The design values for each material are listed in table 8.1 and 8.2. All of the design verification methods used in this thesis are based on the Eurocode 2 (3) or the AFGC, SETRA (15).

Strength Class	f_{ck}	f_{ctm}	$f_{ctk,0.05}$	f_{cd}	ε_{c3}	ε_{cu3}	$v_{Rd,c}$ (beam)	$v_{Rd,c}$ (column)	$v_{Rd,max}$
	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[‰]	[‰]	[N/mm²]	[N/mm²]	[N/mm²] ([θ])
C35/45	35	3.2	2.2	23.3	1.75	3.5	0.33	0.37	3.74 (21.8°)
C90/105	90	5.0	3.5	60	2.3	2.6	0.53	0.59	7.15 (21.8°)
C180/200	180	11	8.0	120	3.0	3.5	0.74	0.74	9.07 (45.0°)

Table 8.1 Material properties: Concretes

Steel type	f_{yk}	f_{yd}	ε_{uk}
	[N/mm²]	[N/mm²]	[‰]
B500B	500	435	5.0

Table 8.2 Material properties: Reinforcement steel

Several elements are checked using M-N-Kappa diagrams. The calculation of the M-N-Kappa diagram is based on the stress-strain relationships of the materials. The stress-strain relationships of OC and HSC are based on the values as stated in the code (3). The stress-strain relationship of UHSC is determined using the recommendations in AFGC, SETRA (15), see table 8.3. Figure 8.3 illustrates the stress-strain relationships of all three materials.

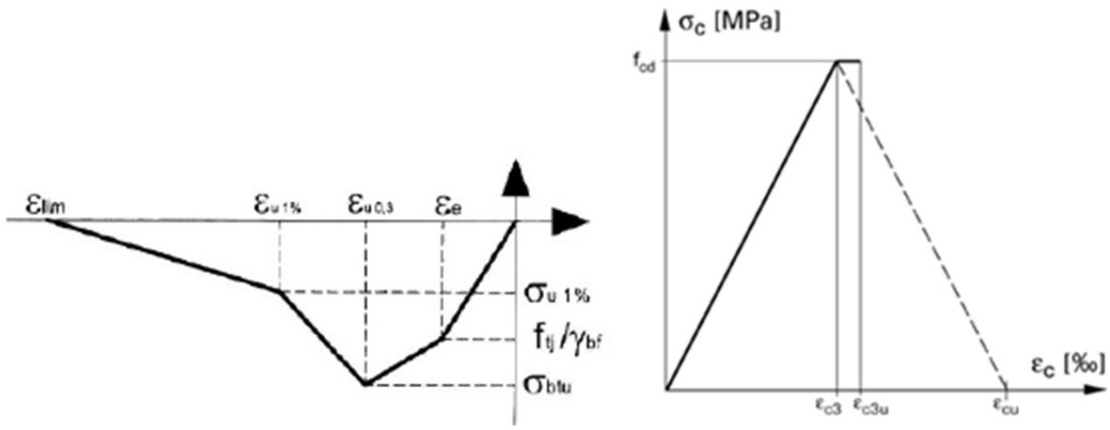


Figure 8.1 and 8.2 Stress-strain relationship tensile part (left) and compression part (right) of UHSC.

C180/200	‰	σ (N/mm²)
ε_{lim}	7.8-2.8*	0
$\varepsilon_{u1\%}$	4.0-1.5*	-2
$\varepsilon_{u,0.3}$	0.88-0.38*	-5.87
ε_{e}	0.097	-3.56
ε_{c3}	-3.0	102
ε_{cu3}	-3.5	102

Table 8.3 Stress-strain relationship of UHSC (*depends on geometry)

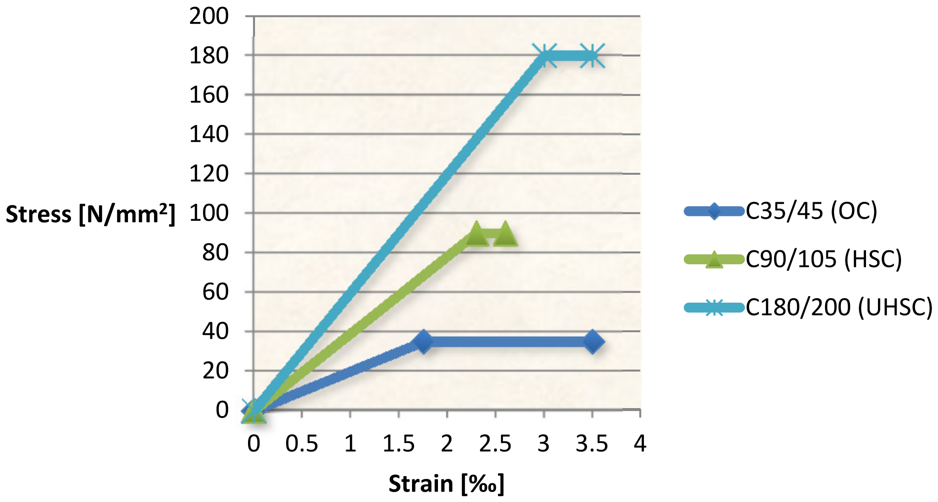


Figure 8.3 Stress-strain relationship of OC, HSC & UHSC (compression part).

8.1.2 Bending Moment Reinforcements

To ensure sufficient capacity of the elements against the normal force and bending moments, sufficient moment reinforcements are applied. The spandrel beams and the corner columns are both significantly loaded by a normal force and bending moments. Therefore, for each element an M-N-Kappa diagram needs to be calculated. In this diagram, the maximum bending moment capacity is determined with the corresponding loads.

To calculate the M-N-Kappa diagram different software is used: Technosoft M-N-Kappa. The software calculates the diagram using geometry, material specifications and the applied forces. The software checks whether the applied axial force and bending moment can be resisted by the element and tests whether the requirement for maximum crack width is met.

The diagrams of each element has been calculated and checked whether sufficient reinforcement is applied. If this was not the case the amount of reinforcement was adjusted until the bending moment capacity of the element complies.

The applied reinforcement has been manually checked whether the requirements on the minimal rebar distance etc. are met.

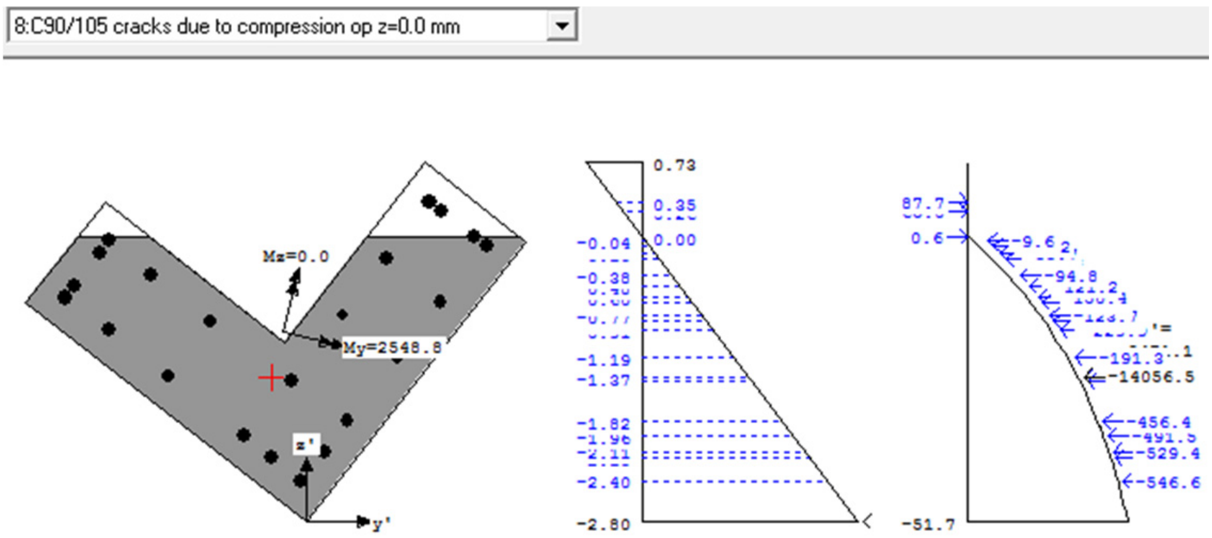
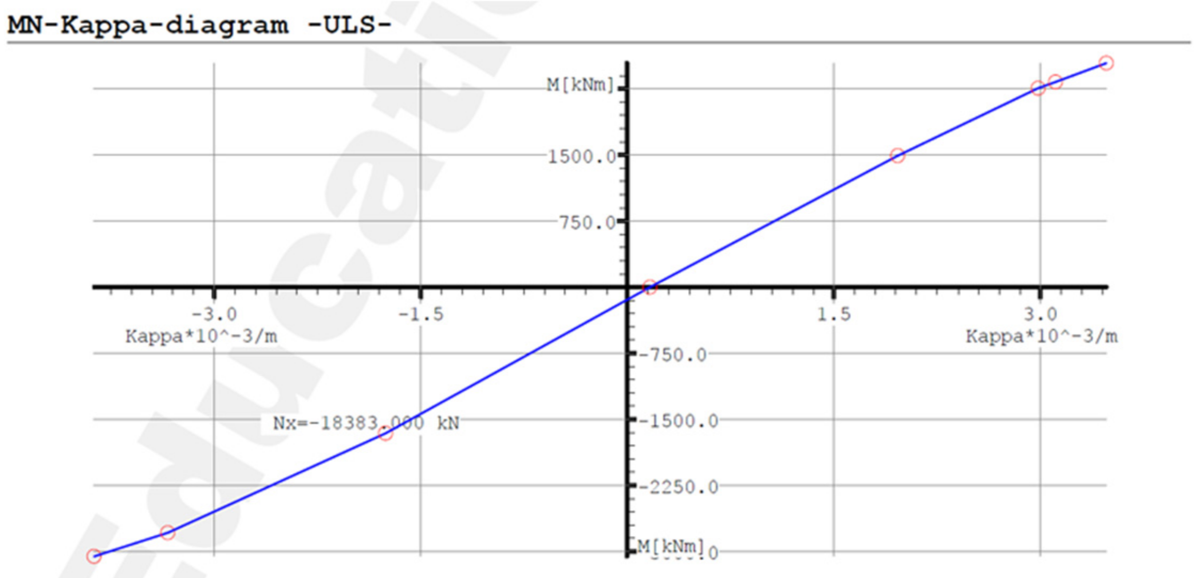


Figure 8.3 Graphical window M-N-Kappa (example is the corner column of the HSC model, variant 1, 35 levels.)



Nx = -18383.000 kN angle=-293.8 graden

Point	z' [mm]	Δε [o/oo]	σ [N/mm²]	x [mm]	ε at top [o/oo]	ε at bottom [o/oo]
1: C90/105 breaks	997.6	-2.800	-51.7	708.9	-2.80000	1.14028
2: B500B yields	891.6	-2.174	-434.8	744.0	-2.53498	0.86423
3: C90/105 cracks	0.0	0.051	2.4	971.8	-1.73592	0.05139
4: M=0.000 kNm	0.0	-0.937	0.0	956.3	-0.76636	-0.93734
5: C90/105 cracks	980.7	0.051	2.4	955.4	0.05139	-1.93866
6: B500B yields	112.3	-2.174	-434.8	817.8	0.50821	-2.52006
7: M=2333.579 kNm	0.0	-2.590	0.0	807.4	0.56349	-2.59010
8: C90/105 breaks	0.0	-2.800	-51.7	779.9	0.73212	-2.80000

Point	My [kNm]	Kappa [10-3/m]	EI [kNm²]	d [mm]	z [mm]	Condition
1: C90/105 breaks	-3052.2	-3.869	757096	835.2	497.4	Eps;c=Eps;cul
2: B500B yields	-2786.8	-3.338	801263	844.9	498.1	Eps's=Eps;spl
3: C90/105 cracks	-1651.0	-1.754	907394	986.2	582.3	Eps;c=fctd
4: M=0.000 kNm	0.0	0.164	0	0.0	0.0	M=0.000 kNm
5: C90/105 cracks	1495.6	1.963	712809	968.0	533.2	Eps;c=fctd
6: B500B yields	2258.3	2.985	709946	866.9	471.0	Eps's=Eps;spl
7: M=2333.579 kNm	2333.6	3.108	704790	866.5	474.1	M=2333.579 kNm
8: C90/105 breaks	2548.8	3.481	688302	866.1	483.6	Eps;c=Eps;cul

Figure 8.4 Output M-N-Kappa Software (example is the corner column of the HSC model, variant 1, 35 levels.)

8.1.3 Shear Force Reinforcements

The shear force verification is performed manually and is based on the code (3) and AFGC, SETRA (15). The shear force capacity of an element is determined by three parts: shear capacity concrete, shear reinforcement and shear capacity fibers.

The shear capacity of the OC and HSC elements are determined using the recommendations in the code (3). When it comes to UHSC, the determination of the shear capacity becomes a bit different due to the different recommendations. The determination of UHSC is as follows:

Shear Capacity Concrete (NEN-EN 1992-1-1)

$$V_{Rd,c} = 0.035 \cdot k^{3/2} \cdot \sqrt{f_{ck}} \cdot b \cdot d \xrightarrow{shear\ stress} v_{Rd,c} = 0.035 \cdot k^{3/2} \cdot \sqrt{f_{ck}}$$

Where:

$$k = 1 + \sqrt{\frac{200}{d}}$$

Shear Capacity Concrete (AFGC, SETRA)

$$V_{Rb} = \frac{1}{\gamma_E} \cdot \frac{0.24}{\gamma_b} \cdot k \cdot \sqrt{f_{cj}} \cdot b \cdot d$$

Where:

γ_E is 1.0
γ_b is 1.5
k is 1.0 (different than in NEN-EN 1992-1-1)

This method to determine the shear capacity of the concrete gives very high values. Therefore, only the first method is used.

Shear Capacity Shear Reinforcement (NEN-EN 1992-1-1)

$$v_{Rd,s} = \frac{A_{sw} \cdot 0.9 \cdot f_{yd} \cdot \cot\theta}{b \cdot s}$$

Where:

A_{sw} is the area of the applied reinforcement bars (2 per loop);
f_{yd} is the yield stress of the reinforcement steel;
θ is the angle of the shear diagonal;
b is the width of the element;
s is the spacing between the reinforcement loops.

Shear Capacity Fibers (AFGC, SETRA)

$$V_f = \frac{S_{eff} \cdot \sigma_p}{\gamma_{bf} \cdot \tan\beta_u}$$

Where:

$$S_{eff} = 0.9 \cdot b \cdot d$$

$\sigma_p = \sigma_{(w,0.3)k} = 8\ N/mm^2$

γ_{bf} is 1.5
β_u is the angle of the shear diagonal

The equation becomes:

$$V_f = \frac{0.9 \cdot b \cdot d \cdot \sigma_{(w,0.3)k}}{\gamma_{bf} \cdot \tan \beta_u} \xrightarrow{shear\ stress} v_f = \frac{0.9 \cdot \sigma_{(w,0.3)k}}{\gamma_{bf} \cdot \tan \beta_u}$$

Total Shear Capacity
The total shear capacity becomes (3):

$v_{Rd} = v_{Rd,c}$ if $v_{Ed} \leq v_{Rd,c}$
$v_{Rd} = v_{Rd,s} + v_f$ if $v_{Ed} > v_{Rd,c}$

The applied reinforcements are listed in a worksheet, see appendix D and E.

8.2 Lower Spandrel Beams

The spandrel beams are 350 mm wide and 1600 mm high. Therefore, the beams are rather high beams, which in this case is very useful. The beams are loaded with a high shear force. The great height of the beam is able to provide much resistance to this shear force.

The beams are also loaded to bending moments. However, the difference in bending moments between the models with the same height is very little. Applying a higher strength concrete does not change much in the maximum bending moment resistance in the beam. Therefore, the needed reinforcement is only reduced by a maximum of 16%.

When it comes to the shear force in the beam it becomes a slightly different story. In all cases, the shear capacity of the beam is smaller than the loaded shear force. Therefore, the shear capacity of the beam may not be added to the total shear force resistance and has to be adopted by either steel reinforcement or steel fibers. Due to the same magnitude of shear force, the shear reinforcement (in the OC model and the HSC model) stays the same in the models with the same height.

In contrast with OC and HSC, the application of UHSC benefits from the addition of fibers. These fibers participate in the shear capacity of the beam. This shear capacity is that high, that there is no extra shear reinforcement required. However, due to practical reasons shear reinforcement has to be applied. This is reduced to a practical minimum.

As a result, there is no significant benefit found in amount of needed reinforcement when higher strength concretes are used. Only the application of fibers does give a high reduction in needed reinforcement.

The summary of forces, bending moments and needed reinforcements is listed in a worksheet (see appendix D).

	N	V_z	M_y	Shear v_{Ed}	θ	$v_{Rd,c}$	$v_{Rd,max}$	v_f	A_{sw}		$v_{Rd,s}$	v_{Rd}	u.c.
	[kN]	[kN]	[kNm]	[N/mm²]	[°]	[N/mm²]	[N/mm²]	[N/mm²]	[mm²/m]		[N/mm²]	[N/mm²]	
30 levels													
OC	-1590	-2017	-2309	3.73	21.8	0.33	3.74		1508	R12-150	4.22	4.22	0.88
HSC	-1560	-2052	-2243	3.79	21.8	0.53	7.15		1508	R12-150	4.22	4.22	0.90
UHSC	-1599	-2153	-2307	3.98	45.0	0.74	9.07	4.80	524	R10-300*	0.59	5.39	0.74

Table 8.4 Results of the shear reinforcement calculation, a part from appendix D

Software Check

The required bending moment reinforcement is calculated by using the M-N-Kappa software. A calculation by hand checks whether the acquired values are trustworthy. The spandrel beam in the OC model of 20 levels is the basis of the following example. The applied method for determining the amount of reinforcement is based on a calculation as is described in table 10.2 of GTB (16) and is based on Eurocode 2 (3).

$$Beam:\ \ b \times h = 350 \times 1600;\ \ C35/45;\ \ XC1$$

$$N_{Ed} = 757\ kN$$

$$M_{Ed} = 1981\ kNm \rightarrow e = 2.617\ m$$

$$\frac{N_{Ed}}{f_{cd} \cdot A_c} = \frac{757 \cdot 10^3}{23.3 \cdot 350 \cdot 1600} = 0.058$$

$$\frac{N_{Ed}}{f_{cd} \cdot A_c} \cdot \frac{e}{h} = \frac{757 \cdot 10^3}{23.3 \cdot 350 \cdot 1600} \cdot \frac{2617}{1600} = 0.095$$

$$0.175 = \rho \cdot \frac{f_{yd}}{f_{cd}} \Rightarrow \rho = 0.94\%$$

$$A_s = 0.94 \cdot \frac{350 \cdot 1600}{100} = 5240\ mm^2 \Rightarrow A_{st} = 2620\ mm^2$$

In the calculation with the M-N-Kappa software, a reinforcement amount of 2590 mm² is applied in the beam. Both results are nearly identical, which confirms the software calculation is correct and trustworthy.

8.3 Corner Columns

In some cases, the corner columns are loaded with a very high axial load. At the same time, the columns are loaded with bending moments and shear forces in Y-direction as well as in Z-direction (local axes).

The dimensions of the columns differ in the two variants. The large column has a width and height of 975 mm (variant one) and the smaller column has a width of 575 mm (variant two).

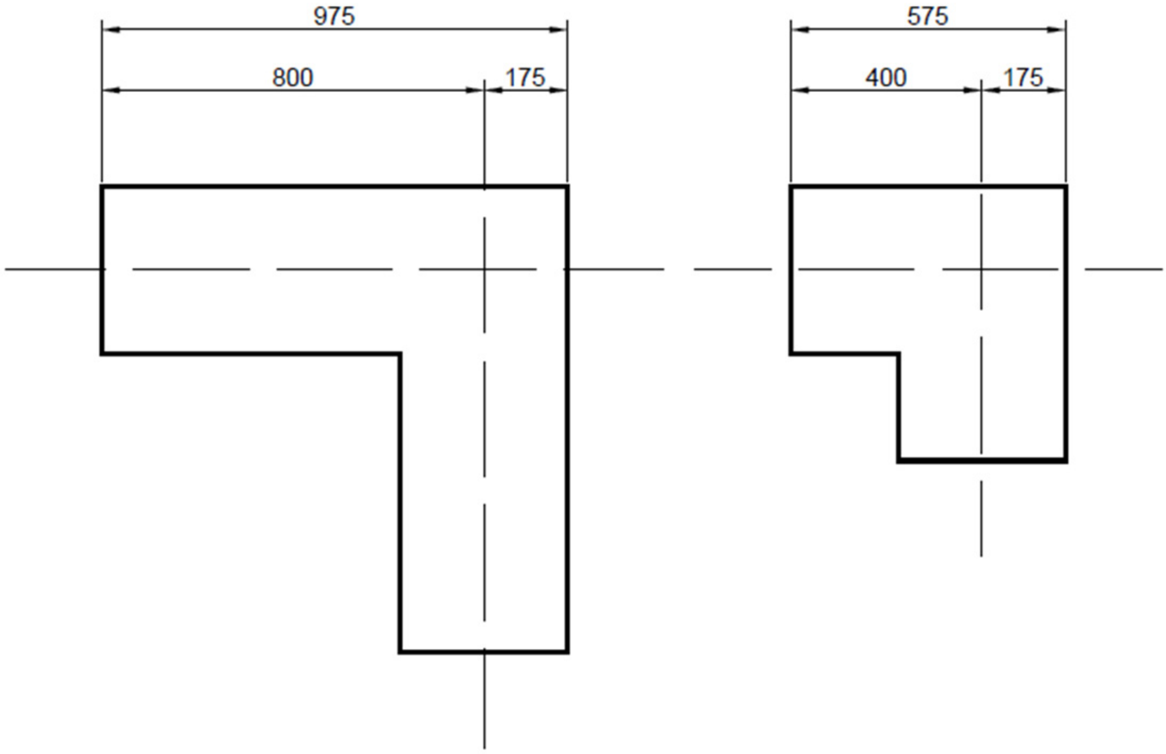


Figure 8.5 Column geometries, variant 1 (left) and variant 2 (right)

Bending Moments Reinforcements

When analysing only a specific model with a certain height, one can see significant difference in the needed reinforcement between the models with different concrete types. For example, the variant one OC model with 30 levels: the column in this model requires a total of 37699 mm² of steel reinforcement, where the HSC model only needs 8875 mm². This is a reduction of 76.5%. The reason for the large amount of reinforcement in the OC model is that the steel reinforcement has to take part in providing enough compression stress. The concrete itself is not able to provide sufficient resistance to the normal force.

However, when the HSC model is pushed to its (displacements) limits, the same column will also need a large amount of reinforcement. By analysing the UHSC model with the maximum height of 40 levels, the concrete provides sufficient normal stress required to absorb the normal force and less reinforcement is required. However, the combination of a bending moment with a tensile force in the column increases the required amount of reinforcement.

As stated before, the amount of reinforcement in the corner columns is significant, especially in the columns of variant two models. In some cases, this amount of reinforcement steel is more than 4%. The code (3) requires that elements loaded by bending moments possess sufficient ductility to warn the users of the structure that the element has reached it maximum. The steel reinforcement in the elements must yield before brittle failure occurs in the concrete's compression zone.

To ensure that yielding occurs before brittle failure in the concrete, steel reinforcement is applied in the compression zone. This makes it possible to strengthen the compression zone and increase its capacity. Therefore, the large amount of reinforcement can be applied in some situations. All of the elements are designed this way.

Shear Reinforcements
In most cases, the shear capacity of the columns varies between 30% and 60% of the required capacity. The required reinforcement to ensure sufficient shear capacity is therefore not very high. The maximum needed shear reinforcement is Ø10-200 (required in OC model).

The shear reinforcement is placed in the two directions, as the shear force is present in Y- and Z-direction (figure 8.6). The reinforcement-loops are placed in each direction, providing its shear capacity for only one direction each. Because the concrete's shear capacity may not be used, the reinforcements must provide the full shear capacity. There will be no overlap in the shear stresses in the cross section. This is not the case in the models with UHSC. The shear resistance of the fibers is very high compared to the applied shear force. It is therefore assumable that there will be no shear failure due to the overlap of shear stresses.

In the UHSC variants, no shear reinforcement is required. However due to practical reasons, the same as in the spandrel beams, practical reinforcement is applied.

The summary of forces, bending moments and needed reinforcements is listed in a worksheet, see appendix E.1.

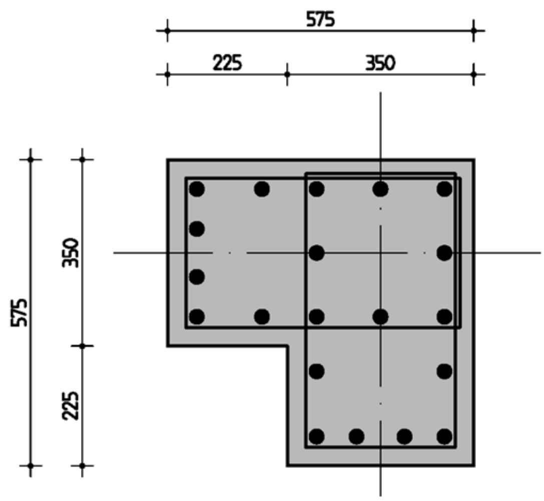


Figure 8.6 Corner column reinforcement.

Column Instability (partial instability)
The large geometry of the columns makes it not very vulnerable for instability. The addition of reinforcement makes the column even less vulnerable. The column slenderness needs to be determined to check whether the columns are vulnerable for instability. Due to the high amount of reinforcements, it is not likely that the columns will fail on instability.

The check is based on Eurocode 2 (3) article 5.8.3 and is described below. In all cases, the element's slenderness λ is required to be smaller than the maximum slenderness λ_{lim}.

The maximum slenderness is calculated by

$\lambda_{lim} = 20 \cdot A \cdot B \cdot C/\sqrt{n}$

where:

$$A = 0.7$$

$$B = \sqrt{1 + 2\omega}$$

$$\omega = \frac{A_s f_{yd}}{A_c f_{cd}}$$

$$C = 0.7$$

$$n = \frac{N_{Ed}}{A_c f_{cd}}$$

The column slenderness is calculated by

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

$$i = \sqrt{\frac{I}{A}}$$

As the columns are stability-providing elements ($l_0 \geq l$), l_0 becomes:

$$l_0 = l \cdot \max\left\{\sqrt{1 + 10 \cdot \frac{k_1 \cdot k_2}{k_1 + k_2}}; \left(1 + \frac{k_1}{1 + k_1}\right) \cdot \left(1 + \frac{k_2}{1 + k_2}\right)\right\}$$

$$k = \frac{\theta}{M} \cdot \frac{EI}{l}; flexibility\ factor, influenced\ by\ adjacent\ beams\ and\ columns$$

The results are calculated and presented on a worksheet, finalized by the check. Table 7.5 shows a summary of the calculation. For the full worksheet, see appendix E.2. As was expected, the columns do not fail on instability. No additional steel reinforcement is required.

	Compr./ Tension	N [kN]	M [kNm]	A_s [mm²]	A [-]	ω [-]	B [-]	C [-]	n [-]	λ_{lim} [-]	$k_1 = k_2$ [-]	l_0 [mm]	i [-]	λ [-]	u.c.
30 levels															
OC	**C**	-20404	1271	37699	0.7	1.26	1.87	0.7	1.56	**14.69**	0.261	3038	274.8	**11.06**	0.75
HSC	**C**	-19561	1206	8875	0.7	0.11	1.11	0.7	0.58	**14.24**	0.261	3038	274.8	**11.06**	0.78
UHSC	**C**	-20042	1218	8168	0.7	0.05	1.05	0.7	0.30	**18.87**	0.261	3038	274.8	**11.06**	0.59
35 levels															
HSC	**C**	-20312	2628	37699	0.7	0.49	1.41	0.7	0.60	**17.72**	0.261	3038	274.8	**11.06**	0.62
40 levels															
UHSC	**C**	-24538	3142	26389	0.7	0.17	1.16	0.7	0.37	**18.78**	0.261	3038	274.8	**11.06**	0.59

Table 8.5 Results of the slenderness calculation, a summary of variant one from appendix E.2

9. Structure Optimisations & Alternatives

Up to this part of the thesis, the structure did not have any changes in the element's geometries. Neither did it consist of a number of steel elements. It remained an entire concrete structure. At certain points in the structure, larger elements, or even, smaller elements could be favourable. High forces demand larger or stronger elements, while elements loaded by relatively small forces can still fulfil their task with a smaller geometry.

This chapter discusses the following optimisations and are calculated briefly where needed:

- Beam height reduction at the upper part of the structure;
- Replacement of the corner columns by steel columns;
- Application of hybrid structures.

Another possible optimisation is to apply UHSC only in the corner columns, while the applied concrete in the beams remains OC or HSC. The high strength of UHSC provides high load bearing capacity without requiring large geometries or large amount of reinforcement steel. However, this optimisation requires a redesigned and time-consuming FEM-model and, therefore, is not analysed.

9.1 Reducing Beam Height

Every variant has a fixed beam height of 1.6 metres. While the lower part of the structure greatly benefits from the great beam height, the structure's upper part can be provided with smaller beams. There are two main concerns when reducing the beam height:

1. Decrease in beam-stiffness.
2. Decrease in shear capacity.

The expectation is that the bending moment resistance would not be an issue as the bending moments are not high in these elements. The decrease in beam-stiffness will result in a less stiff upper part of the structure. Consequently, the total displacement at the top will increase and the structure's natural frequency is lower. This is not a problem, as long as the displacement and natural frequency do not exceed their limit.

As stated in the previous chapters, the spandrel beams are loaded by high shear forces. Therefore, the reduction in beam height is only possible in the upper part of the structure. A reduction of the beam height in the lower part of the structure could result in a large amount of required shear reinforcements. The displacement of the top also increases, as the shear forces are higher in the lower part. Consequently, the shear-racking component of the total displacement increases.

The reduction of the beam height is performed with the above points as the most important issues. Tables 9.1 & 9.2 display the result of the reductions. The height of the beams is reduced to 1.0 metre.

Height Reduction of Spandrel Beams: Variant 1								
	levels	levels with normal beams	levels with low beams	$u_{y;old}$ [mm]	$u_{y;new}$ [mm]	Δ	$u.c._{old}$ [-]	$u.c._{new}$ [-]
OC	30	20	10	189.4	195.3	3.1%	0.92	0.96
HSC	35	25	10	188.8	195.1	3.3%	0.78	0.81
UHSC	40	25	15	206.4	211.7	2.6%	0.75	0.77

Table 9.1 Height reduction of the spandrel beams in variant 1

Height Reduction of Spandrel Beams: Variant 2								
	levels	levels with normal beams	levels with low beams	$u_{y;old}$ [mm]	$u_{y;new}$ [mm]	Δ	$u.c._{old}$ [-]	$u.c._{new}$ [-]
OC	20	13	7	103.4	110.4	6.8%	0.74	0.79
HSC	25	17	8	120.2	128.3	6.7%	0.68	0.73
UHSC	30	19	11	148.5	156.7	5.5%	0.71	0.75

Table 9.2 Height reduction of the spandrel beams in variant 2

The results show little difference in the total displacements. This confirms that the large beam height was not necessary and the structure can still perform well with lower beams at the upper part. As stated before, the reduction in beam height was not applied on all beams. Therefore, the highest shear stress in the beams is still present in the beams at the bottom part of the structure.

Figure 9.1 & 9.2 Height reduction of the spandrel beams. Original beams (left) and Lower beams (right).

9.2 Steel Corner Columns

Variant two´s column geometries are smaller compared to variant one. The standard column width is 1.6 metres where in variant two this is reduced to 0.8 metre. A high normal force loads the corner columns due to the shear lag effect. This is also the case in variant two. Consequently, the corner columns of variant two require large amounts of reinforcement steel. The OC model needs the largest amount of reinforcement steel in the corner columns due to the low concrete strength. Figure 9.3 illustrates the cross-section with the required amount of steel reinforcement.

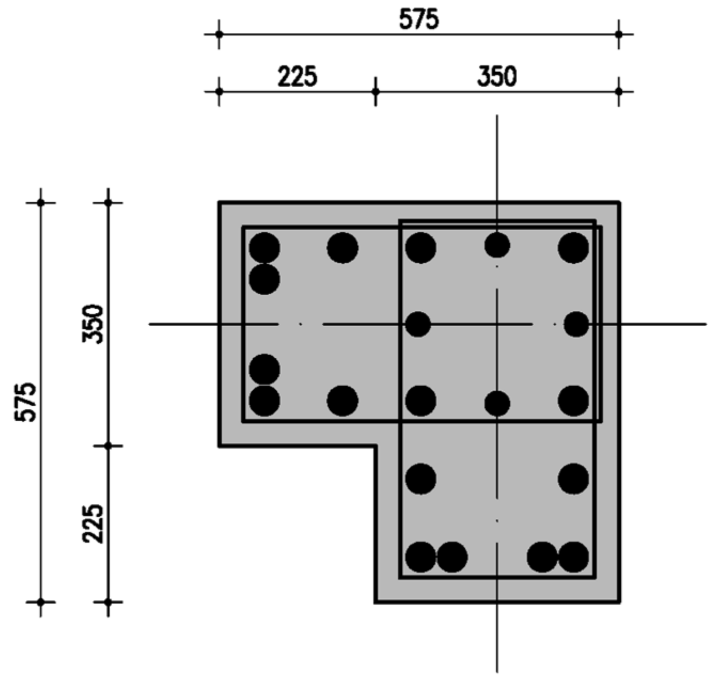


Figure 9.3 Column´s cross section, variant 2, OC model, 20 levels.

The applied reinforcement steel has a total surface of 23323 mm². The shear reinforcements are loops $\varnothing 8 - 250$ (minimum). The results from the M-N-Kappa calculation, as noted in appendix E.1, are summarised in table 9.3.

Column Reinforcements

Variant 2:

	Compr./ Tension	N [kN]	M_y [kNm]	M_z [kNm]	M [kNm]		Required reinforcements: Bending moments A_s [mm²]	A_s [diameters]	M_u [kNm]	u.c.
20 levels										
OC	**C**	-10748	-476	-257	540		23323	16R40+4R32	573	0.94
	T	-63	-693	-242	734				1406	0.52

Table 9.3 M-N-Kappa results for the corner column in variant 2, OC model, 20 levels. (summary of appendix E.1)

The applied amount of reinforcement is very high. Therefore, an optimisation or alternative could improve the column’s design. One of the optimisations is to apply a higher strength concrete, as discussed in the previous section. Another optimisation is to reinforce the concrete column with a steel member. However, the high loads on the column require very large steel members that barely fit into the concrete’s cross-section. Therefore, this type of optimisation is not an option.

9.2.1 Replacement by steel [1]

An alternative is to replace the concrete corner column by a steel only version (only steel; no composite column). The steel columns carry the loads from the structure like the concrete versions do. Replacement by steel columns only takes place at the required levels. If applicable, the standard concrete column is used. This solely depends on the column's loading.

Replacing the corner column by a single steel member soon proved to be impossible. The loading is that high that it would require a steel member that would not fit into the cross-sectional area. The reason for the large steel members is that the column will be loaded with a normal force as well as two bending moments. A steel HEM-type steel member would be a wise solution, as the large normal force requires a large area to provide resistance.

The HEM-type steel members have another thing in common: they do not become wider than 310 millimetres. The members larger than HEM300 actually become less wide. Consequently, the moment resistance in the weak Z-direction is the same for HEM300 members and larger. In this particular case, all HEM-type columns mainly fail on instability in the weak z-direction. This local z-direction corresponds to the main model's x-direction. As stated before, the wind load in x-direction never gave governing results, as it is the structure's strongest axis. If the corner columns would be less stiff in z-direction, the other columns could easily carry the extra load. Consequently, an advantageous reduction of bending moment in z-direction is found in the corner column. To reduce the bending moment in z-direction, the connection is made as flexible as possible in this direction.

This solution would result in a HEM550 steel member. This element would be too large as there would be no space left for (fire) covering of the steel member without staying in the contours of the original concrete column. At the same time, a part of where the original cross section would be is unused. A smaller member (smaller than an HEM550) would fail due to the large normal force in combination with the bending moments. To solve the problem, a second column is applied, which task is only to carry the normal force. Figure 9.4 illustrates this solution.

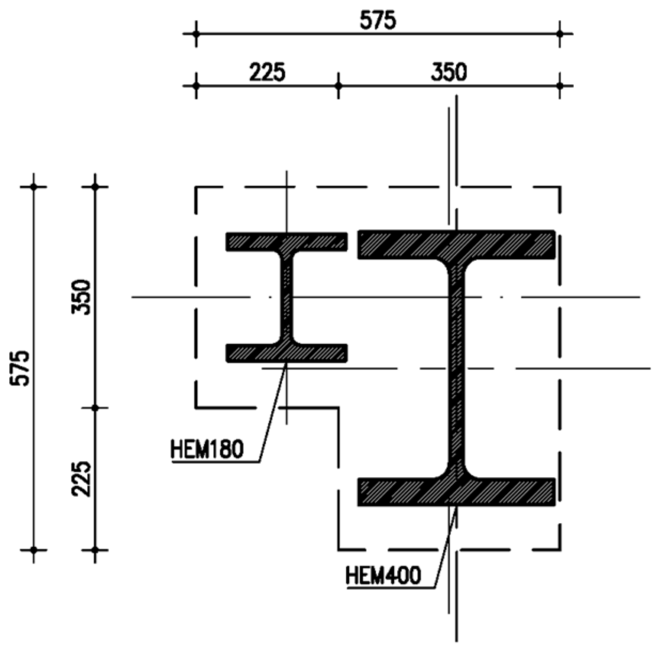


Figure 9.4 Column´s cross section, steel version, standard members. (No concrete is applied.)

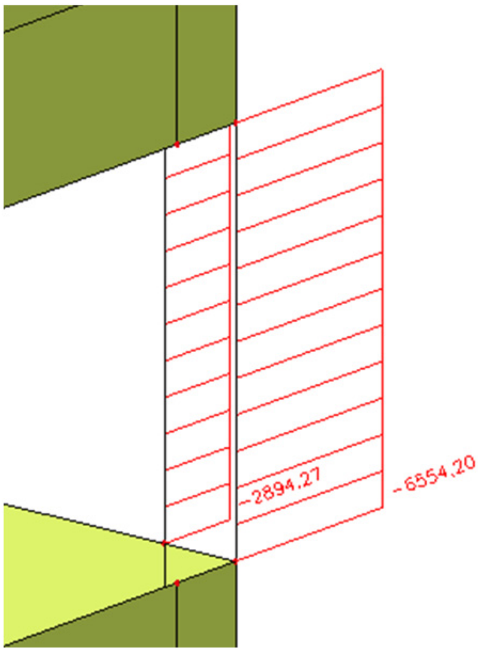


Figure 9.5 Normal force in steel columns in FEM model. HEM180 (left) & HEM400 (right).

Note: The FEM model contains the correct steel members to ensure proper load distribution between the columns.

The second column absorbs a part of the total normal force. As the normal force in the HEM400 element reduces, the element will not fail and is able to provide its resistance to both normal force and bending moment. The unity checks of the members are: HEM400 0.98 and 0.72, HEM180 0.80 and 0.65, where the first unity check belongs to the member on the compression side, and the second unity check to the member on the tension side.

Both columns have flexible connections to the concrete, except for the strong direction of the HEM400 member. The influence on the structure's behaviour is disadvantageous but relatively small and can therefore be accepted to be a suitable solution.

While this is a possible alternative for the concrete corner column, it is expected to be more expensive than only changing the geometry of the concrete column. Discussion on the costs of this solution is discussed in chapter 10.

9.2.2 Replacement by steel [2]

The application of two standard steel members instead of one and a complex connection to the concrete, has a negative but small impact on the structure's behaviour. To ensure similar behaviour as the original structure without any steel members, another solution developed with a custom designed cross-section.

The cross section of this second alternative is designed in a way to ensure proper and almost similar load distribution like the original concrete column. Similar load distribution means that the column is required to carry the normal force and two bending moments. This way, the behaviour of the structure remains the same.

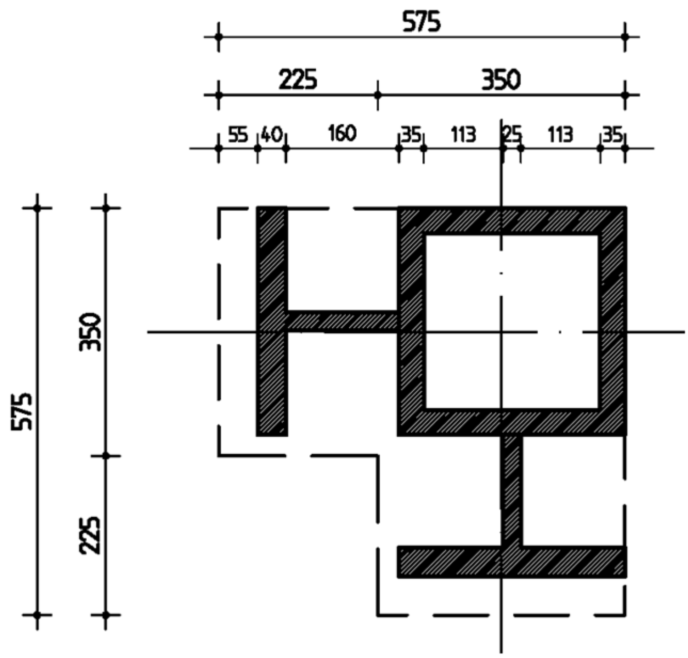


Figure 9.5 Column´s cross section, steel version, custom member. (No concrete is applied.)

The geometry of the steel member's cross-section (figure 9.5) instantly makes clear that high normal forces are loaded onto the member. The thick flanges contain sufficient area to provide high normal force resistance, while the position of the flanges provides great stiffness in both directions. The positioning of the flanges follows the contours of the original concrete column. Therefore, the behaviour of the column remains the same and no complex connection is required. The connection to the concrete mainly consists of an end plate and studs that are bolted or welded to the end plate. The studs ensure the rigid connection to the concrete.

Performance

Due to its large geometry, the column is not vulnerable for column instability/buckling. The governing check for this column is where the combination of normal force and two bending moments is taken into account. This unity check is found to be 0.99.

The building's performance is nearly identical to the original OC model. The steel column with its custom members provides a solution that does not change the behaviour of the structure. The displacements at the top of the building and the loads on the structural members remain nearly identical as in the original OC model.

Compared to the previous solution with steel members, this solution requires more steel. The main reason for this is that no plastic calculation is possible as the cross section is classified as a class 3 cross section. However, if a steel solution is required for replacement of the concrete column, this solution suits best (from a structural point of view).

9.2.3 Replacement by steel: Summary

Both replacements by standard steel members as well as replacement by a custom steel member are possible. Each alternative, which completely replaces the concrete column, comes with its advantages and its disadvantages and are described below.

Replacement by steel [1] *Standard steel members*	**Replacement by steel [2]** *Custom steel member*
Advantages • Relatively cheap solution **Disadvantages** • Negative impact on structure's performance (<10%) • Complex connection to concrete • Increases loadings on other members in the structure	**Advantages** • (Nearly) no impact on structure's performance • Simpler connection to concrete • Absorbs the loads as the original concrete version **Disadvantages** • Expensive solution

While both alternatives have different advantages and disadvantages, they share the disadvantage of requiring steel elements in the structure. The contractor is likely to prefer a concrete only structure as the steel elements interfere with the building flow.

9.3 Hybrid Structure One

One of the results in the previous chapters was that a very high normal force loaded the corner columns. In addition, the columns were loaded with high bending moments. Consequently, the columns are required to have many reinforcements. This is especially the case in the Ordinary Concrete models.

Another result was that the addition of fibers greatly improves the shear capacity of the concrete. With the fiber-reinforced UHSC, as analysed in this thesis, no additional shear reinforcement is required.

These results trigger the idea to optimise the structure with a concrete type that fits exactly into the requirements of the structure. To solve the problem of having a large amount of steel reinforcements in the corner columns, a higher strength concrete can be used in the lower part of the structure. The high strength of the concrete, which provides extra resistance to the normal force, creates the ability to reduce the amount of reinforcements.

The higher strength concrete could also be fiber reinforced. This way no extra shear reinforcement is required; the steel fibers add sufficient shear resistance. The amount of fibers in the Fiber-Reinforced Higher Strength Concrete (FRHSC) is expected to be half of the amount as is applied in UHSC. The tensile strength is estimated to be at around 4 N/mm².

The following optimisation is analysed: Variant one model, the first 13 levels in FRHSC C90/105 and the other 17 floors in OC C35/45. Table 9.4 displays the results acquired in this model. The complete calculation is found in Appendix C.3.

Optimisation using Fiber-Reinforced Higher Strength Concrete

FRHSC & OC

levels	h	u_{max}	u_y	u.c.
	[m]	[mm]	[mm]	
30	108	223.2	127.4	0.57

Table 9.4 Results from optimised model

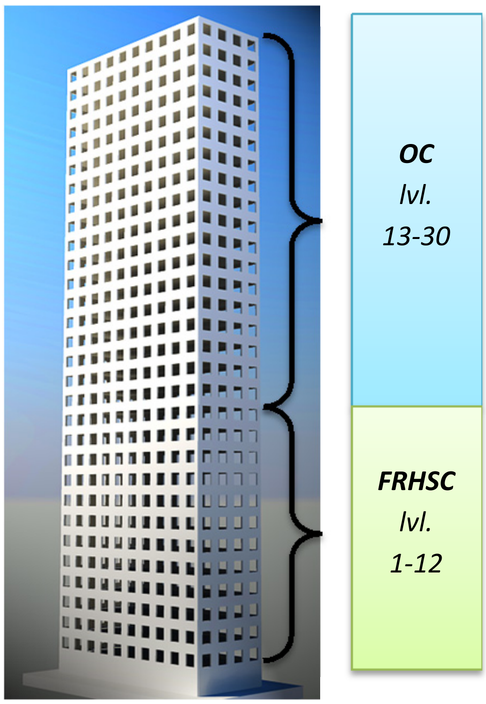


Figure 9.6 Hybrid Structure One

Compared to the HSC model with a height of 30 levels, this is a very good result. The unity check for the displacement only increases from 0.52 to 0.57. When it comes to the shear capacity of the beams, the following results are found. A practical amount of shear reinforcement of $\varnothing 12 - 300$ is applied (shear capacity 2.46 N/mm² in an angle of 21.8°). The maximum shear capacity is exceeded at exactly the 13th floor, as calculated. At this point, FRHSC is applied and the fibers provide the additional required shear capacity. Therefore, in all the beams, a maximum of $\varnothing 12 - 300$ can be applied.

The forces in the structure are almost identical to the "standard" models. Therefore, the same amount of bending moment reinforcements is applied in the columns and the beams.

The use of FRHSC requires little extra attention in the building phase and can be applied without large disadvantages (17). Extra care is required in the joints, resulting from a non-continuous pouring process. The strength of the concrete is reduced at the joints, as no fibers are present in the joints. Chapter 10 continues to discuss this hybrid structure from a costs perspective.

9.4 Hybrid Structure Two

In hybrid structure one, it was chosen to add fibers to the concrete mixture, resulting in FRHSC. However, instead of FRHSC, it is possible to apply HSC in the lower part of the structure. The advantage in shear capacity is obviously not present, but the overall costs could reduce. As the application of HSC is intended to reduce the large amount of reinforcement in the lower columns, the number of HSC levels could be reduced to eight levels of the total 30 levels. This way the structure is optimised in strength and costs. Chapter 10 discusses the costs aspect more in depth.

This type of hybrid structure is a variant one optimisation of the 30 level high OC model with the application of steel corner columns at the lower levels (see section 9.2). The steel corner columns are highly recommended as the amount of steel reinforcement needed in the lower corners columns exceeds the accepted maximum of 4%. The application of the steel columns provides a better quality structure as the chance on brittle failures reduces. However, a full concrete structure without applying too much reinforcement steel would be a better solution.

Hybrid structure two is one of these solutions. It allows the structure to be a full concrete structure without the requirement of steel corner columns. The high strength of the concrete provides sufficient resistance to the normal force. The steel reinforcement is only required to provide bending moment capacity and instability resistance.

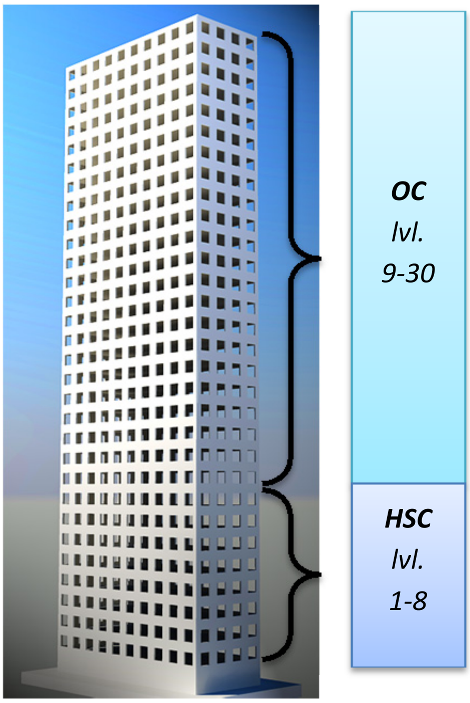


Figure 9.7 Hybrid Structure Two

Performance

The hybrid structure performs better on maximum displacements and required reinforcement steel compared to the OC model. The maximum displacement at the top of the structure is slightly reduced due to the higher Young's Modulus at the lower eight levels. Compared to the full HSC model with the same height, little difference is found in the maximum displacement at the top. It confirms that the top displacement mainly depends on the stiffness at the lower part of the structure, where the bending moment due to wind loading is the highest. Consequently, there is no need in applying HSC in the top levels unless more slender elements are applied at the top levels.

The required reinforcement at the lower columns is the same as in the HSC model with 30 levels (loads are identical). Next to the required amount on the lowest level, the required amount of reinforcement steel on the ninth level is displayed in table 9.5. On and beyond this level, OC is used.

Column Reinforcements

Hybrid Structure Two:

	Compr./ Tension	N [kN]	M_y [kNm]	M_z [kNm]	M [kNm]	Required reinforcements: A_s [mm²]	A_s [diameters]	M_u [kNm]	u.c.
Level one									
HSC	**C**	-19493	-1150	-350	1202	8875 (1.58%)	4R25+22R20	1732	0.69
	T	529	695	534	876			953	0.92
Level nine									
OC	**C**	-11856	-681	-130	693	6032 (1.08%)	30R16	735	0.94
	T	-765	423	326	534			918	0.58

Table 9.5 M-N-Kappa results for the corner column in hybrid structure two.

The required amount of reinforcement steel is significantly reduced compared to the OC model. There is less chance of brittle failure and the lower amount of reinforcement steel saves costs. Section 10.3 discusses how this optimisation could reduce total costs.

10. Costs

This chapter discusses the several analysed models in this thesis from a costs perspective. The costs of optimisations and alternatives are discussed either. The costs are based on price assumptions in consultation with Grünewald (18) and van der Horst (17). This chapter is included in this thesis to discuss whether the applied materials are beneficial in its use. To create a clear view of the costs, only the costs that vary per variant or model are taken into account.

10.1 Construction Costs

The costs of a building mainly consist of the following parts: structural costs, façade costs, finishing costs and operational costs. All four parts are important and necessary for the construction work. What is more interesting is the influence it has on the total building costs. A good estimation is that the first two parts will each take up to 35% of the total building costs. The finishing costs would be at around 15% and the operational costs will add up the last 15% of the total costs (17).

In this thesis, only the structure is analysed. Therefore, the first part, the structural costs, is the most relevant part. In this particular case, this structural part is divided into the foundation, the prefabricated floors and the façade. The foundation and the prefabricated floors in total would take up 65% of the structural costs; the structural façade itself would be responsible for 35% of the structural costs (17). This concludes that the cost of the structural façade is 12% of the total building costs.

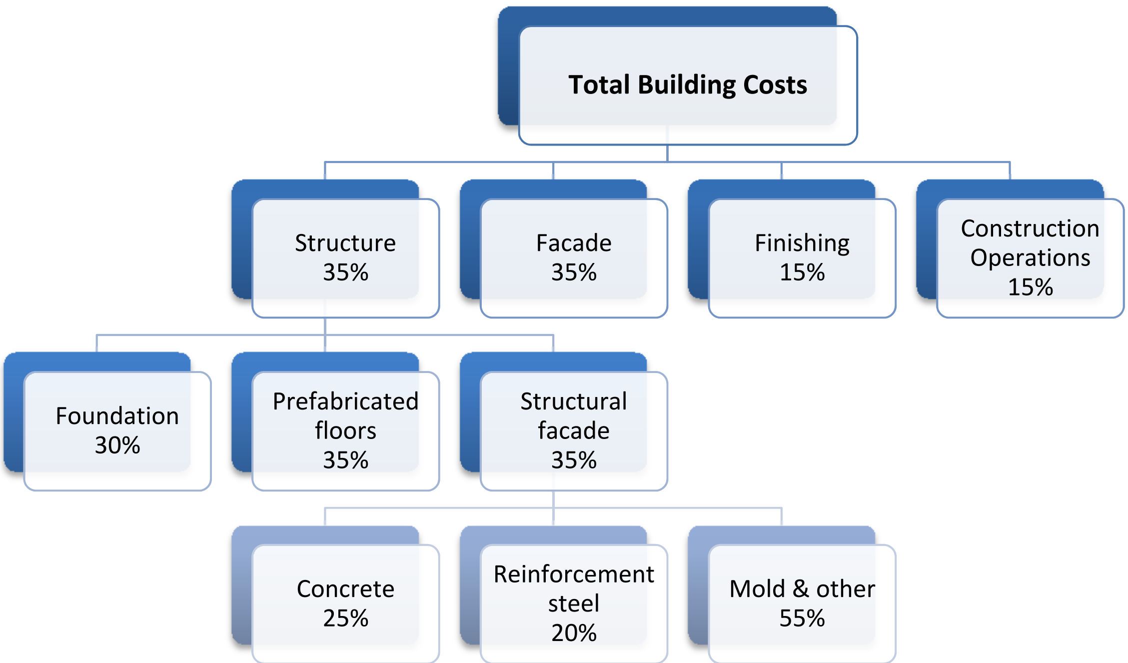


Figure 10.1 Total Building Costs.

Numbers in this figure are determined in consultation with engineers at Koninklijke BAM Groep nv (17).

The structural façade, which is the framed-tubular structure, consists of concrete and reinforcement steel. The concretes used in this thesis have price tags that differ a lot from each other. What makes this difference and how much is this difference? The next section answers these questions.

10.1.1 Concrete Mixtures Prices

The three mixtures used in this thesis differ a lot from each other. To achieve the various strengths, several changes are required in the concrete's mixture, which makes the mixture more expensive. The mixture prices mentioned below are based on an interview with Grünewald (18).

Ordinary Concrete C35/45

The Ordinary Concrete mixture is a mixture that is widely used in many construction works. It contains relatively large aggregates and the amount of cement is relatively low. In most situations, the strength of this concrete is sufficient and there is no need for a mixture with better properties. The price of this mixture is around € 100/m³.

High Strength Concrete C90/105

Finer aggregates are used in a high strength concrete mixture. Consequently, the mixture requires more cement paste to ensure sufficient bond with the aggregates. While the strength of the material increases, the fire resistance is decreased. Synthetic fibers are added to ensure sufficient fire resistance. All these factors influence the total price of the mixture. On average, the price of a high strength concrete mixture is € 200-250/m³. This price is raised to € 400/m³ if steel fibers (€ 1.50/kg) are added to the mixture.

Ultra High Strength Concrete C180/200

The process of adding finer aggregates continues in developing UHSC. Apart from adding finer aggregates, more cement paste is required. The type of aggregates is also different. Aggregates like bauxite and silica fume, are used in large quantities and make the mixture a lot more expensive. The added steel fibers (high quantity) raise the mixture's price as well. The fiber amount of 235 kg/m³ raises the price with approximate € 350/m³. The total price of the mixture becomes € 1000/m³. This is a mixture with a Young's Modulus of 50.000 N/mm². To achieve a higher Young's Modulus, like applied in this thesis, more silica fume is added to the mixture. This mixture is priced at € 1200/m³.

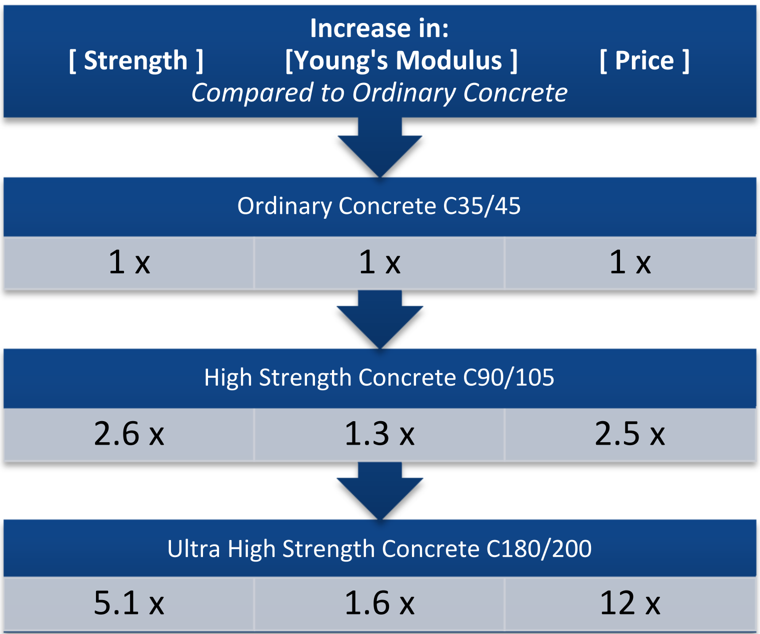


Figure 10.2 Strength versus Young's Modulus versus Price flow. Strength (left), Young's Modulus (middle), Price (right).

In practice, very little use of concrete mixtures like C120 or higher is found, unless strictly required. If the strength of the concrete goes beyond the point of 120 N/mm², expensive aggregates are required that raise the total mixture price to record high prices (17).

10.1.2 Reinforcement steel and fibers prices

The reinforcement steel used in the concrete is the B500B-type. The price of the reinforcement steel lies at around € 0.50/kg. The price of the fibers varies as many types and different quality fibers are on the market. For use in HSC and UHSC small fibers are used, which have better properties. These better properties come with a higher price. The price of the fibers is approximately € 1.50/m³.

The labour costs required for applying reinforcement steel is estimated at € 1.50/kg. This is three times more expensive than the price of the reinforcement steel itself. A total price of € 2.00/kg is used to take into account the purchase and labour costs of the reinforcement steel. No large influence on the total price is expected if the amount of reinforcement is reduced as many factors influence the labour costs (17).

10.1.3 Other costs associated with HSC & UHSC

To use a HSC or UHSC, some other parts of the construction process need more attention. The application of the stronger concretes requires more observation and attention during the construction process. It also requires a longer mixing time in the concrete factory. In the case of HSC and especially UHSC, sometimes more research is required before the specific type of concrete can be used.

All these factors make the application of higher strength concretes more expensive. Therefore, it is wise to observe these extra costs involved in using these concretes. In the end, the concrete mixtures are only used when they provide significant (costs) benefits.

10.2 Costs comparison

A result of the analysis was that a larger building height could be achieved when using HSC or UHSC. As stated in the previous section, these concretes are more expensive. However, the costs of the concrete structure only take up 12% of the total building costs. This section discusses the costs and benefits between the models.

The higher building height of the structure can eventually lead to an increase of sellable area. The use of HSC and UHSC makes this possible. At the same time, the structure becomes more expensive as the HSC and UHSC mixtures are more expensive than the OC mixture. The following (rough) costs comparison analysis finds out how the two relate to each other.

Variant one
In variant one, the maximum building height is 30, 35 and 40 levels for respectively OC, HSC and UHSC.

As illustrated in figure 10.1, the total costs of the concrete itself is 35% x 35% x 25% = 3% of the total building costs. This is an estimation for the analysed models and only serves to create an indication on the influence of expensive concrete mixtures. The numbers in the figure are based on current projects that have a similar type of structure (17).

The total amount of concrete in the variant one façade is 87.8 m³ per floor level. By multiplying this amount with the price of the concrete mixture, the total price of the concrete is calculated. In the case of HSC and UHSC, an extra cost factor is introduced to take into account the higher costs involved in using these types of concretes (as discussed section 9.1.3) (17). The building's price per floor level is then calculated by extrapolation using the assumption that the concrete's price is 3% of the total building costs. Table 10.1 presents the results of this calculation.

Concrete	m³ / floor level	# levels	€/m³	extra costs factor	costs of concrete structure	difference with OC model	total costs	costs per floor level	Difference
	[m³]	[-]	[€]	[-]	[€[	[€]	[€]	[€]	[-]
OC	87.8	30	100		263,424	0	8,608,627	286,954	-
HSC	87.8	35	250	1.2	921,984	777,728	9,386,355	268,182	-7%
UHSC	87.8	40	1200	1.4	5,900,698	5,756,442	14,365,069	359,127	25%

Table 10.1 Cost per floor comparison, variant one.

Variant two

The same analysis is performed in the variant two models. In variant two, the maximum building height is 20, 25 and 30 levels for respectively OC, HSC and UHSC. Due to the smaller columns, less concrete is applied. Consequently, the concrete's price is also less than in variant one, but the higher façade costs, due to larger window openings, compensates the total building costs. The concrete's price to the total building costs ratio is assumed lower: 2.5%. The results are that the average total costs per floor are roughly the same as in variant one. The comparison for variant two can be found in table 10.2.

Concrete	m³ / floor level	# levels	€/m³	extra costs factor	costs of concrete structure	difference with OC model	total costs	costs per floor level	Difference
	[m³]	[-]	[€]	[-]	[€[	[€]	[€]	[€]	[-]
OC	72.1	20	100		144,256	0	5,747,251	287,363	-
HSC	72.1	25	250	1.2	540,960	396,704	6,143,955	245,758	-14%
UHSC	72.1	30	1200	1.4	3,635,251	3,490,995	9,238,246	307,942	7%

Table 10.2 Cost per floor comparison, variant two.

Note: The models in table 10.1 and 10.2 are models with its maximum possible height. Therefore, the structure is fully utilised and the amount of reinforcement (in %) is the same in all models. The 20% reinforcement costs remains unchanged.

Conclusion costs comparison

Despite the fact that the HSC-mixture is a more expensive mixture, the results are very promising for this concrete type. The concrete costs are only a fraction of the total building costs. Therefore, the more expensive concrete price could be shadowed by the other costs, while the number of floor levels increases.

The result is that there are two structures possible, one in OC and one in HSC. The OC model has a height of 20 or 30 levels (depending on which variant is chosen), while the HSC model has a height of

25 or 35 levels. This means that in both variants, five more sellable floor levels are added to the structure by applying HSC. Consequently, the costs per floor are decreased, as seen in table 10.1 & 10.2. This is a price advantage of respectively 7% and 14%.

When it comes to UHSC, the benefits do not outweigh the increase of costs. The price disadvantage is 25% and 7% for respectively variant one and two. The concrete mixture is simply too expensive for this structure. The building needs to be re-designed to benefit from the great properties without

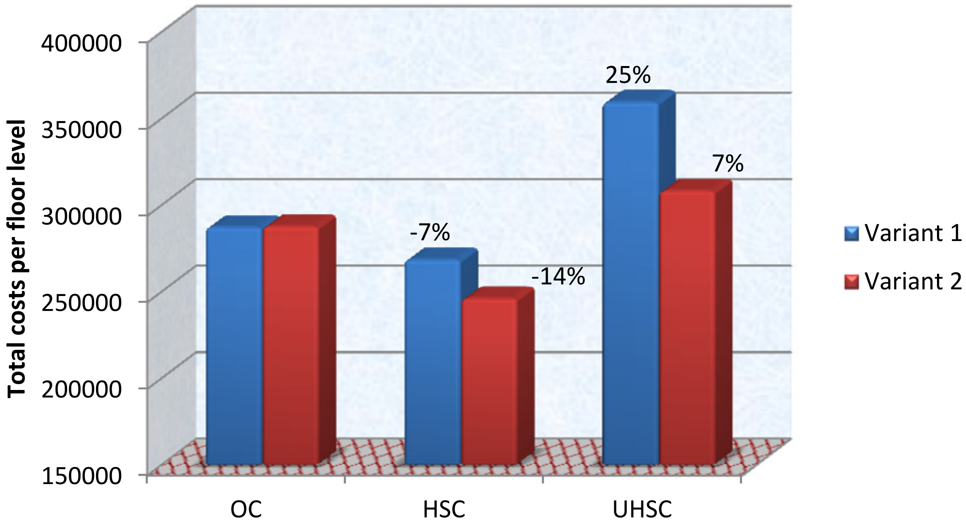


Figure 10.3 Total costs per floor level comparison

ending up in a too expensive structure. A reduced amount of concrete in the structure could make the use of UHSC more beneficial. This is already confirmed in the variant two results in the comparison. Due to the smaller element sizes, more benefit is gained in applying a higher strength concrete.

OC or HSC without geometry differences

The analysis points out that a structure with the same height can be built by applying all three types of concrete. The higher strength of the concrete requires less reinforcement steel, which reduces the reinforcement costs.

For instance, when the variant one, 30 level high model is analysed, the corner columns would require a large amount of steel reinforcements. This amount is significantly reduced by applying HSC. The reduced quantity of reinforcement steel saves costs (see section 10.3). However, this advantage comes with a concrete mixture that is twice the price of the OC mixture. The reduction in reinforcement steel is only present at the lower floor levels, which results in relatively small costs savings compared to the high costs of the concrete mixture.

A short calculation shows that the application of HSC costs € 15,800 extra per floor level compared to OC. The reinforcement costs per floor level are € 14,000 (at 1% reinforcement rate), which is slightly less than the increase of concrete costs. The expensive mixture is applied in the whole structure; consequently, a reduced amount of reinforcement steel does not outweigh the costs. A very large reduction in steel reinforcement or a hybrid model (section 10.3) is required to benefit from using a higher strength concrete in a model with the <u>same height</u>.

10.3 Alternatives Costs

Chapter 9 discusses several optimisations and alternatives. Not all of the mentioned changes in the structure are cheaper, but they improve the structure in various ways.

Reducing beam height
By reducing the beam height in the upper part of the structure, less concrete is used compared to the original structure. The reduction of beam height allows more daylight entry and improves the building's design with a more open façade. The building is not necessarily cheaper due to the lower beams, because larger window surfaces also mean higher costs. The lower beams could improve the building's design and can make the design more light or slender.

Steel columns
The main goal to replace the corner columns by steel members is to increase the building's quality compared to the original OC model. The concrete corner column in the OC model requires very large reinforcements, which increases the chance of brittle failure.

The steel alternative makes it able to stay within the columns original contour lines and is not intended to reduce costs. The replacement by steel columns is an expensive solution, as partly seen in the comparison with hybrid model two. The cheapest solution would be to apply changes in the concrete without applying any steel members.

The cheapest solution of the two steel alternatives is the solution with standard steel members. The building does not perform the same as the original model (top displacement is increased by max. 10% and still meets the displacement limit), but the costs are reduced to a minimum. The solution with the custom cross-section requires more steel and special welding. These two factors significantly increase the total costs of this solution (17). Consequently, contractors would avoid this solution.

Hybrid structure one
The hybrid structure benefits from the improved properties of fiber reinforced HSC (FRHSC). While the shear capacity of this concrete mixture is greatly increased, the mixture becomes more expensive. The reason for that is that the fibers are expensive and could almost double the price of the mixture, compared to non-fiber reinforced HSC.

The reduced amount of required shear reinforcement is not that significant that it can outweigh the increased costs of FRHSC. The cost of the FRHSC mixture is almost doubled compared to the HSC mixture. Consequently, the hybrid structure in this layout is more expensive than the normal HSC model.

Hybrid structure two
In this case, no fibers are added to the HSC mixture; the mixture is not as expensive as FRHSC. The mixture is only used in the lower eight floor levels of the structure, which make the structure less expensive. The less expensive OC mixture is applied in the rest of the structure. This hybrid structure provides a good performance to costs ratio. To acquire an indication on the difference in costs, a short calculation is performed, comparing the OC model with the hybrid structure two model.

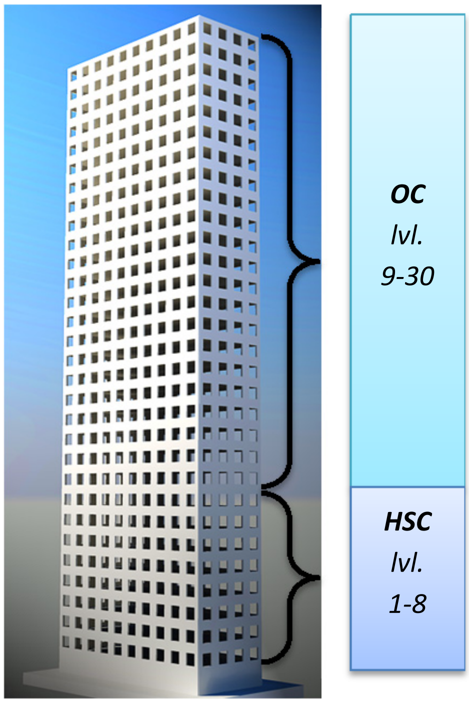


Figure 10.4 Hybrid Structure Two

The following approach is used to indicate the costs difference. The corner columns have a major influence on the costs of the structure. The rest of the structure is assumed to be more or less the same in both situations. The amount of reinforcement steel in the corner columns varies the most between both models and is illustrated as zones I and II in figure 10.4.

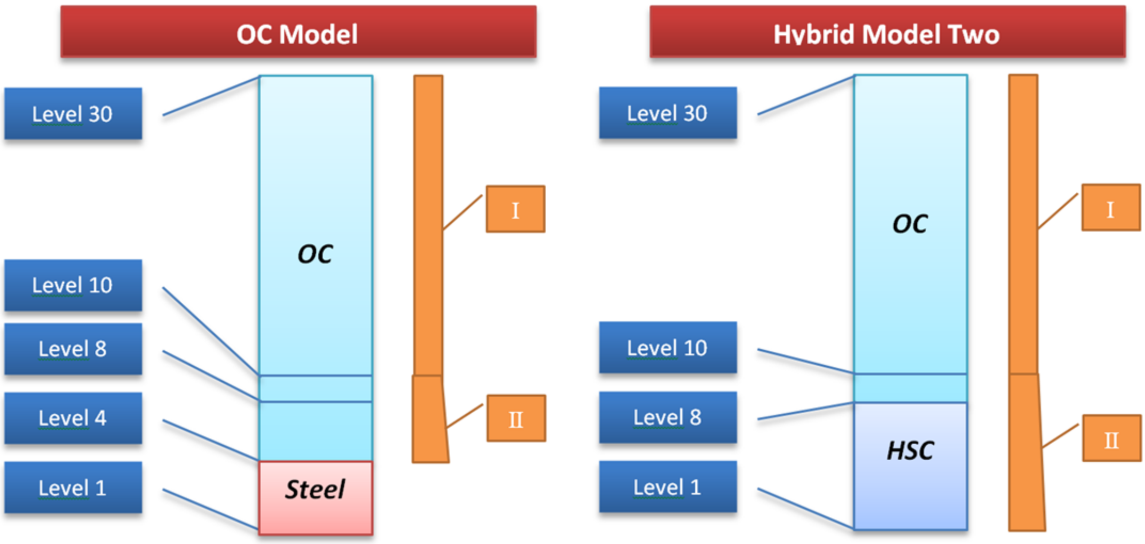


Figure 10.5 Composition of corner columns: reinforced concrete (OC/HSC) or steel replacement.

To calculate the costs of the structure the following is assumed:

- Concrete mixture prices based on section 9.1;
- Steel reinforcements are approximately 80 kg/m³ of concrete;
- Zone I contains 1% of steel reinforcement (including reinforcement loops);
- Zone II contains the required percentage of reinforcement steel originating from M-N-Kappa diagrams;

- Maximum amount of reinforcement steel of 4% is accepted; beyond this point steel columns are applied;
- Steel S355 is used for the columns, on average 7920 kg per floor level is required (in total 8 columns per floor level, two per corner);
- Price of reinforcement steel including labour is € 2,-/kg (without labour € 0,50/kg);
- Price of steel S355 including labour is € 2,30/kg (without labour € 1,60/kg).

The above assumptions are based on section 9.3, additional calculations, and on the interview with Grünewald (18) and van der Horst (17). The results of the calculation are listed in table 10.3 and 10.4.

Hybrid Model Two indeed saves costs, but the costs savings are relatively small and could be neglected. However, the structure performs better than the OC model. Reasons to choose for the hybrid model are:

- Better performing structure (less maximum displacement & better comfort);
- Cheaper solution;
- Better quality structure (no complex connections with steel members);
- No steel members / full concrete structure.

OC model							
Price of Concrete							
	m³	€/m³					€
C35/45	2600	100					€ 260,014
Price of Steel Reinforcement in corner columns including labour							
	lvls.		ave	kg/m³	m³	kg	€
Zone I	20	1%	1.0%	80	161.3	12902	€ 25,805
Zone II	6	1%-3.9%	2.5%	196	48.4	9483	€ 18,967
Price of Steel Reinforcement in rest of the structure including labour							
	lvls.		ave	kg/m³	m³	kg	€
Zone I	22		1.0%	80	1770.3	141626	€ 283,251
Zone II	4		1.5%	120	318.9	38273	€ 76,547
Steel part	4		2.2%	176	318.9	56134	€ 112,268
Price of Steel columns including labour							
	lvls.	kg/lvl	kg				€
S355	4	7920	31680				€ 72,864
Total							**€ 849,716**

Table 10.3 Costs indication of the OC model structure.

Hybrid Structure Two							
Price of Concrete							
	m³	€/m³	extra				€
C35/45	1932	100					€ 193,160
C90/105	702	250	1.2				€ 210,720
Price of Steel Reinforcement in corner columns including labour							
	lvls.		ave	kg/m³	m³	kg	€
Zone I	20	1%	1.0%	80	161.3	12902	€ 25,805
Zone II	10	1%-1.58%	1.3%	103.2	80.6	8322	€ 16,644
Price of Steel Reinforcement in rest of the structure including labour							
	lvls.		ave	kg/m³	m³	kg	€
Zone I	20	1%	1.0%	80	1594.7	127578	€ 255,155
Zone II	10	1%	1.0%	80	797.4	63789	€ 127,578
Price of Steel columns including labour							
	lvls.	kg/lvl	kg				€
S355	4	0	0				0
Total							**€ 829,062**

Table 10.4 Costs indication of the Hybrid Model Two structure.

11. Conclusion

As described in chapter two the objectives of this thesis are as follows:

- Researching the possibility of the application of HSC and UHSC in tubular structures.
- Summing up the advantages and disadvantages of the application of HSC and UHSC.
- Performing a comparison with similar OC structures, including a limited discussion on the costs of the various structures.
- Researching the possibility of achieving a greater building height while maintaining the same geometry and element sizes and still meet the requirement of maximum lateral deflection.

The thesis' analysis consisted of several models that all have the same geometries but differ in the used concrete types. The models have been developed and analysed and have provided many results. In many cases, these results give information that was unknown at the start of the thesis.

11.1 Researching the possibility of the application of HSC and UHSC in tubular structures

As stated in the literature study prior to this main study, the application of HSC and UHSC is possible. The structures with these higher strength concretes make total benefit of the better material properties: less reinforcement steel is required and elements become stiffer. Eventually, elements with smaller geometries become can be applied while still fulfilling the requirements to strength and stiffness. However, these benefits do not come without any disadvantages.

11.2 Advantages and disadvantages

The number one conclusion of the analysis is that structures from higher strength concrete, with the same dimensions as in ordinary concrete structures, have a better overall performance. The most notable increase in performance is recognised in the lateral displacements. At the same time, the elements exposed to high normal forces benefit from the high compressive strength of HSC and UHSC as less steel reinforcement is required.

The better performance was slightly noticeable in the magnitude of the shear-lag effect. The higher stiffness of the higher strength concrete has a small, but positive influence on reducing the shear-lag effect. However, the improvements are very small and, in most cases, can be neglected.

When it comes to reinforcing the elements, also better performance is noticeable. The corner columns benefit from the compressive strength of the higher strength concretes: the amount of reinforcement in corner columns or the column dimensions can be reduced. However, when it comes to the beams that are loaded with a smaller normal force but larger bending moments, there is no significant benefit in having a higher compressive strength.

From a performance point of view, there are almost no disadvantages. The use of UHSC allows the engineer to design a more slender structure. More slender elements mean an increased chance of instability issues. The design of the structure is required to deal with these issues. The other disadvantages are found in the higher material costs and issues in the building phase (see literature study).

Summary:

Advantages

- Less lateral displacements *(-44% in HSC model and -62% in UHSC model)*;
- Less reinforcement required in most heavily loaded elements *(up to -78% in same-height-models and 27% in fully utilised models / maximum height models)*;
- More slender design possible *(column size reduced by half when applying UHSC)*;
- Small decrease in magnitude of the shear-lag effect *(max. 8%)*.

Disadvantages

- High costs compared to OC *(HSC +150%, UHSC +1100%)*;
- Increased chance of instability issues due to slender design;
- Nearly no benefits in elements loaded with only bending moments.

11.3 Comparison with similar OC structures

The result from the comparison with similar OC structures is that when the geometry and the height of a structure remain the same, the structure will perform better and/or less reinforcement is required when applying a higher strength concrete.

In the corner columns of the structure, the required amount of reinforcement is that large, that a change in geometry or application of a higher strength concrete can be satisfying. Consequently, a lower amount of reinforcement is required and the chance on brittle failure is reduced, resulting in a better and more economical structural design.

The better properties of UHSC make it possible to build a 30 level structure with small columns (variant two). If a building of the same height is required when using OC, large columns need to be used (variant one). This is an advantage for UHSC, as the same building height can be achieved with a more slender structure.

From a costs perspective, the application of a higher strength concrete only has its advantages if changes are made in the structure. Increasing building height (compared to the OC model), or creating a hybrid structure increases the building's performance with minimal or less costs per floor level.

11.4 Increase of building height

A significant increase in maximum allowable building height is found when a higher strength concrete is applied to the structure. In both variants an increase in building levels of 5 and 10 is achieved when respectively HSC and UHSC is applied. In variant two, this means an increase of 50% if UHSC is applied.

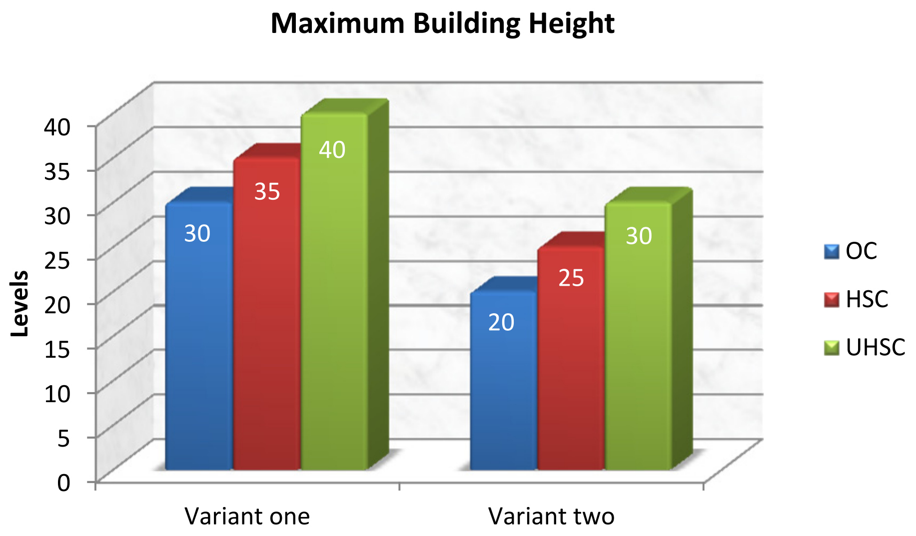


Figure 11.1 Maximum building height of structures with the various concrete types.

In this aspect, the better performing structure results in a building with more sellable area and thus more attractive to build for investors. The HSC model performs well in this analysis as the total costs per floor level reduces. The cost per floor level reduces by 7% and 14% in, respectively, variant one and two. In the case of the UHSC model, an increase is found instead of a reduction. The cost per floor level increases by 25% and 7% in respectively variant one and two. The extra costs in applying this concrete mixture are too high to make the better performing structure beneficial.

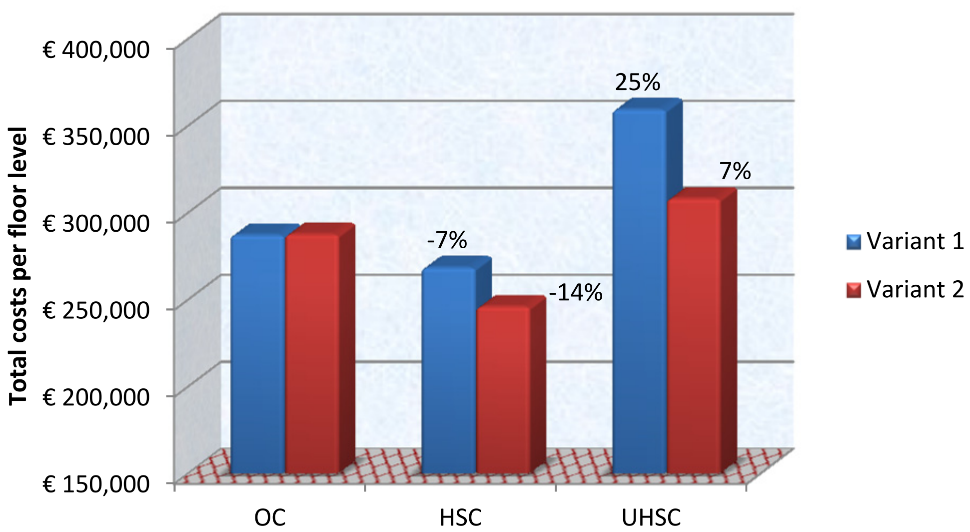


Figure 11.2 Total costs per floor level comparison.

Optimal height
Element geometries and applied materials (with its costs) influence the quest for an optimal building height. The optimal height is determined by taking into account the costs per floor level and the amount of sellable floor area. The HSC model dominates in all of the results thanks to its good price per floor level performance; the HSC model is a structure without a high amount of steel reinforcement and replacements by steel columns and does not require a too expensive concrete mixture.

The optimal building heights for a framed tubular structure with a typical floor plan of 36 by 14.4 m^2 are as follows:

- 35 levels (126 metres): Variant one geometries with the use of High Strength Concrete (Hybrid model possible).
- 25 levels (90 metres): Variant two geometries with the use of High Strength Concrete (Hybrid model possible).

Recommendation to achieve greater optimal building height: Increase element sizes (in lower levels).
Recommendations to achieve smaller optimal building height: Decrease the overall thickness of the structure or apply less high beams. Note: The advantages of HSC are not present if the building height is significantly reduced.

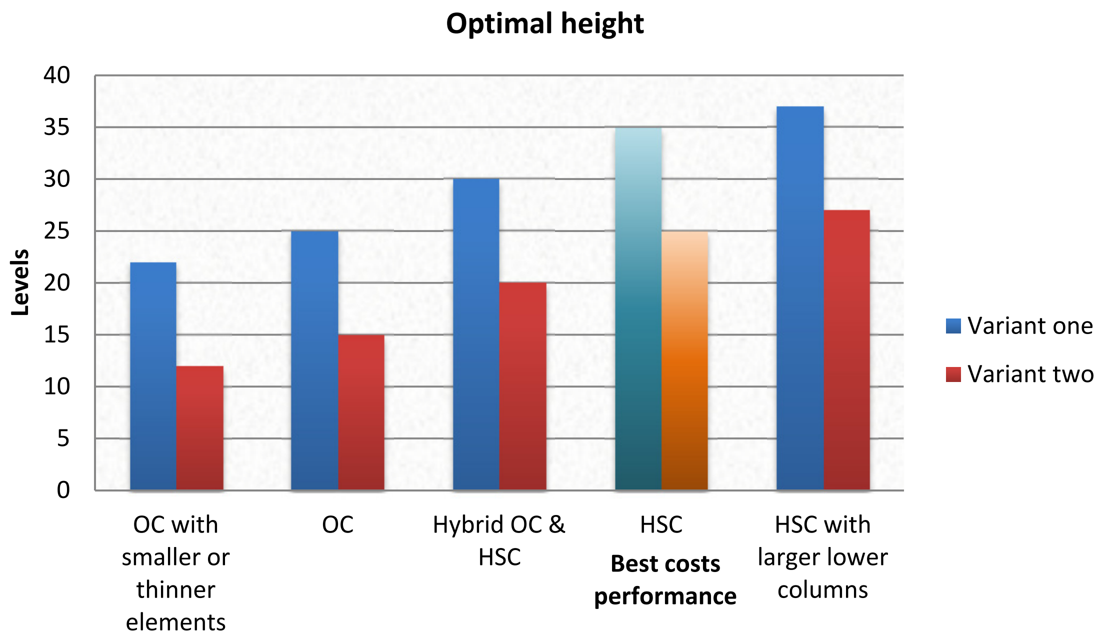


Figure 11.3 Optimal height, with recommended structure types.

11.5 Rules of Thumb

The use of the rules of thumb as provided by Faessen is not recommended when using higher strength concretes, as the results are often incorrect. The high Young's modulus of the higher strength concretes is not taken into account in determining the forces in the various elements of the structure. As confirmed in the analysis on the shear-lag effect, the higher Young's modulus in higher strength concretes changes the behaviour of the structure.

To correct the issue, a factor is required that takes into account the different structural behaviour. A simple factor in the form of a linear relationship with the Young's modulus will not correct the issue as no linear relationship is recognised in the results. To make sure that the rules of thumb are applicable when applying higher strength concretes, the following steps are recommended:

1. Introduce a factor to take into account the different behaviour of concrete mixtures with high Young's moduli in tubular structures.

2. Take into account the concrete’s stiffness (Young’s Modulus) in determining the forces in the structure.

11.6 End Conclusion

The application of higher strength concretes in framed-tubular structures is possible. It provides better performing structures and, in some cases, a reduction in costs. While the HSC models proved to provide a good performance to costs ratio, the UHSC models currently do not. Currently, the high price of the UHSC mixture does not outweigh the advantages.

UHSC suits itself best in application with a structure that is designed for it. The high strength allows the use of a slender structural design, which can justify the high price of the mixture. In the thesis’s analysis no costs advantages are found when using UHSC, because the design is not specifically based on the use of this concrete type.

Recommendations

- Fully utilise the high material properties of higher strength concretes to reduce the overall costs of a framed tube structure.
- Apply both high strength concrete as well as ordinary concrete in order to create and fully optimise a hybrid structure. Optimised hybrid structures are proven to provide best performance in strength, stiffness, and costs.
- The use of steel fibers to reduce shear reinforcement is not recommended as no cost benefit or performance benefits are found.
- Do not apply UHSC in framed tube structure variants as discussed in this thesis. No costs benefits are found unless the high properties of UHSC are utilised. To achieve this, a redesign of the structure is required.
- If only higher strength concretes are used: design the structure in a way to fully utilise the high material properties.
- If an element requires too much reinforcement: enlarge cross-section instead of replacement by steel. Replacement by steel is proven to be an expensive solution. Applying a higher strength concrete is also satisfactory (if possible).

11. Conclusion

Bibliography

1. **Balbaid, Heykal.** *Literature Study: Application of Higher Strength Concretes in Tubular Structures.* Amsterdam : TU Delft, 2011.

2. **Faessen, S.J.M.G.** *Master Thesis: Ontwerpregels voor Betonnen Gevelbuisconstructies.* Delft : Delft University of Technology, 2000.

3. **Eurocode 2: Concrete Structures, NEN-EN 1992.**

4. **Dycore Systeemvloeren.** [Online] http://www.dycore.nl.

5. **de Bruijn, H.J.** *Literature Study of Master Thesis: Voorgespannen Spoorbrug in Hogere Sterkte Beton.* Utrecht : Holland Railconsult BV & Delft University of Technology, 2005.

6. **Eurocode 0: Basis of Structural Design, NEN-EN 1990.**

7. **Eurocode 1: Actions on Structure, NEN-EN 1991.**

8. **Kerstens, K.** *Eindrapport: Constructief ontwerp voor woontoren Oostplein, Rotterdam.* Delft : Corsmit Raadgevend Ingenieursbureau & Delft University of Technology, 2004.

9. **Dicke, D.** *Stabiliteit voor ontwerpers.* 2005.

10. **prof.dr.ir. J.C. Walraven & dr.ir.drs. C.R. Braam.** *Lecture Notes: CT4160 Prestressed Concrete.* Delft : Delft University of Technology, 2011.

11. **Nederlands Normalistatie Instituut.** *NEN6720: VBC 1995 Voorschriften Beton.* Delft : s.n., 1995.

12. **C. Hartsuijker & J.W.Welleman.** *Lecture Notes: CT2031 Constructiemechanica 3 "Stabiliteit van het evenwicht".* Delft : Delft University of Technology, 2007.

13. **Nederlands Normalisatie-instituut.** *NTA Hoogbouw (03-A).* Delft : NEN - Convenant Hoogbouw, 2009.

14. *CUR Aanbeveling 103.*

15. **AFGC, SETRA.** *Bétons fibrés à ultra-hautes performances. Recommandations provisoires. Documents scientifiques et techniques.* Bagneux Cedex : Association Française de Génie Civil, Setra, 2002. pp. 1-152.

16. **Betonvereniging.** *Grafieken en tabellen voor gewapend-betonconstructies.* Gouda : s.n., 2010.

17. **Horst, A.C.Q. van der.** Koninklijke BAM Groep nv & Delft University of Technology. Bunnink, Delft : s.n., 2011.

18. **Grünewald, Dr.ir. S.** Delft : Delft University of Technology & Hurks Beton, 2011.

19. **Braam, dr.ir.drs. C.R.** s.l. : Delft University of Technology.

20. **Ketel, ir. J.A. RO, Willemse, ing. R.F. PMSE RC, van Rijen, ing. P., Koolen, ing. E.** *Toepassing van Vezelversterkt Ultra Hoge Sterkte Beton in bouwkundige constructies (1) & (2) (Preliminary article).* s.l. : Cement, 2011.

Appendix A Loads

A.1 Loads Overview (kN/m^2)

						kN/m^2
floor level	$q_{g;k}$	prefabricated concrete slab				5.00
		concrete layer d=	0.07	24		1.68
		ceiling				0.50
		finishing				1.00
						8.18
	$q_{q;k}$	live load				3.50
		seperation walls				0.80
		ψ =	0.50		extreme	4.30

facade (without inner structure)	$q_{g;k}$	2.00
facade openings	$q_{g;k}$	1.00

A.2 Line Loads (kN/m^1)

ψ table: momentane (0) or fully loaded (1)

q_1

	ψ	#	length	kN/m^2	kN/m^1		kN/m^2	kN/m^1
floor level	1	1.00	7.20	8.18	58.90	extr.	4.30	30.96
facade (without inner		0.69	3.60	2.00	4.97			
facade openings		0.31	3.60	1.00	1.12			
				$q_{g;k}$ =	64.98		$q_{q;k}$ =	30.96

q_2

	ψ	#	length	kN/m^2	kN/m^1		kN/m^2	kN/m^1
facade (without inner		0.69	3.60	2.00	4.97			
facade openings		0.31	3.60	1.00	1.12			
				$q_{g;k}$ =	6.08		$q_{q;k}$ =	0.00

Appendix B.1 Wind load in X-direction

Table NB.4 NEN-EN 1991-1-4

Height	Area II; built-up
1	0.58
2	0.58
3	0.58
4	0.58
5	0.58
6	0.58
7	0.58
8	0.62
9	0.65
10	0.68
15	0.8
20	0.9
25	0.97
30	1.03
35	1.09
40	1.13
45	1.17
50	1.21
55	1.25
60	1.28
65	1.31
70	1.34
75	1.37
80	1.39
85	1.42
90	1.44
95	1.46
100	1.48
105	1.50
110	1.52
115	1.54
120	1.55
125	1.57
130	1.59
135	1.61
140	1.62
145	1.64
150	1.65
155	1.66
160	1.67
165	1.69
170	1.70
175	1.71
180	1.72
185	1.74
190	1.75
195	1.76
200	1.77

b=14,4

level	**height**	**h/d**	**$c_{pe,D}$**	**$c_{pe,E}$**	**c_{pe}**	**z_e**	**q_p**	**corr fa**	**w_e**	**0.5b**	**q_{wind}**
1	3.6	0.1	0.8	0.50	1.30	14.4	0.79	0.85	0.87	7.2	6.3
2	7.2	0.2	0.8	0.50	1.30	14.4	0.79	0.85	0.87	7.2	6.3
3	10.8	0.3	0.8	0.50	1.30	14.4	0.79	0.85	0.87	7.2	6.3
4	14.4	0.4	0.8	0.50	1.30	14.4	0.79	0.85	0.87	7.2	6.3
5	18	0.5	0.8	0.50	1.30	18.0	0.86	0.85	0.95	7.2	6.8
6	21.6	0.6	0.8	0.50	1.30	21.6	0.92	0.85	1.02	7.2	7.3
7	25.2	0.7	0.8	0.50	1.30	25.2	0.97	0.85	1.07	7.2	7.7
8	28.8	0.8	0.8	0.50	1.30	28.8	1.02	0.85	1.12	7.2	8.1
9	32.4	0.9	0.8	0.50	1.30	32.4	1.06	0.85	1.17	7.2	8.4
10	36	1	0.8	0.50	1.30	36.0	1.10	0.85	1.21	7.2	8.7
11	39.6	1.1	0.8	0.51	1.31	39.6	1.13	0.85	1.25	7.2	9.0
12	43.2	1.2	0.8	0.51	1.31	43.2	1.16	0.85	1.29	7.2	9.3
13	46.8	1.3	0.8	0.52	1.32	46.8	1.18	0.85	1.32	7.2	9.5
14	50.4	1.4	0.8	0.52	1.32	50.4	1.21	0.85	1.36	7.2	9.8
15	54	1.5	0.8	0.53	1.33	54.0	1.24	0.85	1.40	7.2	10.1
16	57.6	1.6	0.8	0.53	1.33	57.6	1.27	0.85	1.43	7.2	10.3
17	61.2	1.7	0.8	0.54	1.34	61.2	1.29	0.85	1.46	7.2	10.5
18	64.8	1.8	0.8	0.54	1.34	64.8	1.31	0.85	1.49	7.2	10.7
19	68.4	1.9	0.8	0.55	1.35	68.4	1.33	0.85	1.52	7.2	11.0
20	72	2	0.8	0.55	1.35	72.0	1.35	0.85	1.55	7.2	11.2
21	75.6	2.1	0.8	0.56	1.36	75.6	1.37	0.85	1.58	7.2	11.4
22	79.2	2.2	0.8	0.56	1.36	79.2	1.39	0.85	1.60	7.2	11.5
23	82.8	2.3	0.8	0.57	1.37	82.8	1.41	0.85	1.63	7.2	11.8
24	86.4	2.4	0.8	0.57	1.37	86.4	1.43	0.85	1.66	7.2	12.0
25	90	2.5	0.8	0.58	1.38	90.0	1.44	0.85	1.68	7.2	12.1
26	93.6	2.6	0.8	0.58	1.38	93.6	1.45	0.85	1.71	7.2	12.3
27	97.2	2.7	0.8	0.59	1.39	97.2	1.47	0.85	1.73	7.2	12.4
28	100.8	2.8	0.8	0.59	1.39	100.8	1.48	0.85	1.75	7.2	12.6
29	104.4	2.9	0.8	0.60	1.40	104.4	1.50	0.85	1.78	7.2	12.8
30	108	3	0.8	0.60	1.40	108.0	1.51	0.85	1.80	7.2	13.0
31	111.6	3.1	0.8	0.61	1.41	111.6	1.52	0.85	1.82	7.2	13.1
32	115.2	3.2	0.8	0.61	1.41	115.2	1.54	0.85	1.84	7.2	13.3
33	118.8	3.3	0.8	0.62	1.42	118.8	1.55	0.85	1.86	7.2	13.4
34	122.4	3.4	0.8	0.62	1.42	122.4	1.56	0.85	1.88	7.2	13.6
35	126	3.5	0.8	0.63	1.43	126.0	1.57	0.85	1.91	7.2	13.7
36	129.6	3.6	0.8	0.63	1.43	129.6	1.59	0.85	1.93	7.2	13.9
37	133.2	3.7	0.8	0.64	1.44	133.2	1.60	0.85	1.95	7.2	14.0
38	136.8	3.8	0.8	0.64	1.44	136.8	1.61	0.85	1.97	7.2	14.2
39	140.4	3.9	0.8	0.65	1.45	140.4	1.62	0.85	1.99	7.2	14.3
40	144	4	0.8	0.65	1.45	144.0	1.65	0.85	2.03	7.2	14.6
41	147.6	4.1	0.8	0.66	1.46	147.6	1.66	0.85	2.05	7.2	14.8
42	151.2	4.2	0.8	0.66	1.46	151.2	1.66	0.85	2.06	7.2	14.9
43	154.8	4.3	0.8	0.67	1.47	154.8	1.66	0.85	2.07	7.2	14.9
44	158.4	4.4	0.8	0.67	1.47	158.4	1.67	0.85	2.08	7.2	15.0
45	162	4.5	0.8	0.68	1.48	162.0	1.68	0.85	2.10	7.2	15.1
46	165.6	4.6	0.8	0.68	1.48	165.6	1.69	0.85	2.12	7.2	15.3
47	169.2	4.7	0.8	0.69	1.49	169.2	1.70	0.85	2.14	7.2	15.4
48	172.8	4.8	0.8	0.69	1.49	172.8	1.71	0.85	2.16	7.2	15.6
49	176.4	4.9	0.8	0.70	1.50	176.4	1.71	0.85	2.18	7.2	15.7
50	180	5	0.8	0.70	1.50	180.0	1.72	0.85	2.19	7.2	15.8

Appendix B.2 Wind load in Y-direction

Table NB.4 NEN-EN 1991-1-4

Height	Area II; built-up
1	0.58
2	0.58
3	0.58
4	0.58
5	0.58
6	0.58
7	0.58
8	0.62
9	0.65
10	0.68
15	0.8
20	0.9
25	0.97
30	1.03
35	1.09
40	1.13
45	1.17
50	1.21
55	1.25
60	1.28
65	1.31
70	1.34
75	1.37
80	1.39
85	1.42
90	1.44
95	1.46
100	1.48
105	1.50
110	1.52
115	1.54
120	1.55
125	1.57
130	1.59
135	1.61
140	1.62
145	1.64
150	1.65
155	1.66
160	1.67
165	1.69
170	1.70
175	1.71
180	1.72
185	1.74
190	1.75
195	1.76
200	1.77

b=36

level	**height**	**h/d**	**$c_{pe,D}$**	**$c_{pe,E}$**	**c_{pe}**	**z_e**	**q_p**	**corr fa**	**w_e**	**0.5b**	**q_{wind}**
1	3.6	0.25	0.8	0.58	1.38	14.4	1.10	0.85	1.28	18	23.1
2	7.2	0.5	0.8	0.58	1.38	14.4	1.10	0.85	1.28	18	23.1
3	10.8	0.75	0.8	0.58	1.38	14.4	1.10	0.85	1.28	18	23.1
4	14.4	1	0.8	0.58	1.38	14.4	1.10	0.85	1.28	18	23.1
5	18	1.25	0.8	0.58	1.38	18.0	1.10	0.85	1.28	18	23.1
6	21.6	1.5	0.8	0.58	1.38	21.6	1.10	0.85	1.28	18	23.1
7	25.2	1.75	0.8	0.58	1.38	25.2	1.10	0.85	1.28	18	23.1
8	28.8	2	0.8	0.58	1.38	28.8	1.10	0.85	1.28	18	23.1
9	32.4	2.25	0.8	0.58	1.38	32.4	1.10	0.85	1.28	18	23.1
10	36	2.5	0.8	0.58	1.38	36.0	1.10	0.85	1.28	18	23.1
11	39.6	2.75	0.8	0.59	1.39	39.6	1.13	0.85	1.33	18	23.9
12	43.2	3	0.8	0.60	1.40	43.2	1.16	0.85	1.38	18	24.8
13	46.8	3.25	0.8	0.61	1.41	46.8	1.18	0.85	1.42	18	25.6
14	50.4	3.5	0.8	0.63	1.43	50.4	1.21	0.85	1.47	18	26.5
15	54	3.75	0.8	0.64	1.44	54.0	1.24	0.85	1.52	18	27.3
16	57.6	4	0.8	0.65	1.45	57.6	1.27	0.85	1.56	18	28.1
17	61.2	4.25	0.8	0.66	1.46	61.2	1.29	0.85	1.60	18	28.8
18	64.8	4.5	0.8	0.68	1.48	64.8	1.31	0.85	1.64	18	29.5
19	68.4	4.75	0.8	0.69	1.49	68.4	1.33	0.85	1.68	18	30.3
20	72	5	0.8	0.70	1.50	72.0	1.35	0.85	1.72	18	31.0
21	75.6	5.25	0.8	0.70	1.50	75.6	1.37	0.85	1.75	18	31.5
22	79.2	5.5	0.8	0.70	1.50	79.2	1.39	0.85	1.77	18	31.8
23	82.8	5.75	0.8	0.70	1.50	82.8	1.41	0.85	1.79	18	32.3
24	86.4	6	0.8	0.70	1.50	86.4	1.43	0.85	1.82	18	32.7
25	90	6.25	0.8	0.70	1.50	90.0	1.44	0.85	1.84	18	33.0
26	93.6	6.5	0.8	0.70	1.50	93.6	1.45	0.85	1.85	18	33.4
27	97.2	6.75	0.8	0.70	1.50	97.2	1.47	0.85	1.87	18	33.7
28	100.8	7	0.8	0.70	1.50	100.8	1.48	0.85	1.89	18	34.0
29	104.4	7.25	0.8	0.70	1.50	104.4	1.50	0.85	1.91	18	34.4
30	108	7.5	0.8	0.70	1.50	108.0	1.51	0.85	1.93	18	34.7
31	111.6	7.75	0.8	0.70	1.50	111.6	1.52	0.85	1.94	18	35.0
32	115.2	8	0.8	0.70	1.50	115.2	1.54	0.85	1.96	18	35.2
33	118.8	8.25	0.8	0.70	1.50	118.8	1.55	0.85	1.97	18	35.5
34	122.4	8.5	0.8	0.70	1.50	122.4	1.56	0.85	1.99	18	35.8
35	126	8.75	0.8	0.70	1.50	126.0	1.57	0.85	2.01	18	36.1
36	129.6	9	0.8	0.70	1.50	129.6	1.59	0.85	2.03	18	36.5
37	133.2	9.25	0.8	0.70	1.50	133.2	1.60	0.85	2.04	18	36.7
38	136.8	9.5	0.8	0.70	1.50	136.8	1.61	0.85	2.05	18	37.0
39	140.4	9.75	0.8	0.70	1.50	140.4	1.62	0.85	2.07	18	37.2
40	144	10	0.8	0.70	1.50	144.0	1.65	0.85	2.10	18	37.8
41	147.6	10.25	0.8	0.70	1.50	147.6	1.66	0.85	2.11	18	38.0
42	151.2	10.5	0.8	0.70	1.50	151.2	1.66	0.85	2.12	18	38.2
43	154.8	10.75	0.8	0.70	1.50	154.8	1.66	0.85	2.12	18	38.1
44	158.4	11	0.8	0.70	1.50	158.4	1.67	0.85	2.13	18	38.3
45	162	11.25	0.8	0.70	1.50	162.0	1.68	0.85	2.14	18	38.5
46	165.6	11.5	0.8	0.70	1.50	165.6	1.69	0.85	2.15	18	38.7
47	169.2	11.75	0.8	0.70	1.50	169.2	1.70	0.85	2.16	18	39.0
48	172.8	12	0.8	0.70	1.50	172.8	1.71	0.85	2.17	18	39.1
49	176.4	12.25	0.8	0.70	1.50	176.4	1.71	0.85	2.18	18	39.3
50	180	12.5	0.8	0.70	1.50	180.0	1.72	0.85	2.19	18	39.5

Appendix C.1 Displacements, Natural Frequencies & Second Order

Variant 1: Column width: 1600 mm

OC

levels	h	u_{max}	u_x	$u_{y;total}$	$u_{y;structu}$	$u_{y;foundat}$	%	f_e	T	F_d	$N_{total;SLS}$	$N_{total;ULS}$	N_{wind}	$EI_{structure}$	$n_{s;SLS}$	$n_{s;ULS}$	C	$n_{f;SLS}$	$n_{f;ULS}$	Total "Knikgetal" n_{SLS}	n_{ULS}	2nd-order factor SLS	ULS	End result u_y	u.c.	levels
	[m]	[mm]	[mm]	[mm]	[mm]	[mm]		[Hz]	[sec]	[kN]	[kN]	[kN]	[kN]	[kNm²]			[kNm/rad]							[mm]		
20	72	151.2	5.1	50.2	42.6	7.6	15.1%	0.85	1.18	1989	197987	310278	3895	4.27E+09	32.4	20.7	1.33E+09	186.4	118.9	27.6	17.6	1.04	1.06	52.1	0.34	20
30	108	223.2	18.7	189.4	171.6	17.8	9.4%	0.46	2.16	2572	284770	444810	6260	5.74E+09	13.5	8.6	2.05E+09	133.4	85.4	12.3	7.8	1.09	1.15	206.2	0.92	30
40	144	295.2	52.7	540.2	503.7	36.5	6.8%	0.29	3.47	3364	365642	570288	8927	6.62E+09	6.8	4.4	2.54E+09	96.3	61.8	6.4	4.1	1.19	1.33	641.1	2.17	40

HSC

levels	h	u_{max}	u_x	$u_{y;total}$	$u_{y;structu}$	$u_{y;foundat}$	%	f_e	T	F_d	$N_{total;SLS}$	$N_{total;ULS}$	N_{wind}	$EI_{structure}$	$n_{s;SLS}$	$n_{s;ULS}$	C	$n_{f;SLS}$	$n_{f;ULS}$	Total "Knikgetal" n_{SLS}	n_{ULS}	2nd-order factor SLS	ULS	End result u_y	u.c.	levels
	[m]	[mm]	[mm]	[mm]	[mm]	[mm]		[Hz]	[sec]	[kN]	[kN]	[kN]	[kN]	[kNm²]			[kNm/rad]							[mm]		
20	72	151.2	3.2	29.7	22.1	7.6	25.6%	1.07	0.94	2160	198108	310460	3895	8.22E+09	62.5	39.9	1.33E+09	186.3	118.9	46.8	29.8	1.02	1.03	30.3	0.20	20
30	108	223.2	11.0	106.2	88.6	17.6	16.6%	0.60	1.65	3005	284951	445082	6260	1.11E+10	26.1	16.7	2.07E+09	134.8	86.3	21.9	14.0	1.05	1.08	111.3	0.50	30
35	126	259.2	18.6	185.9	155.8	30.1	16.2%	0.48	2.10	3438	325388	507833	7555	1.21E+10	18.3	11.7	1.99E+09	97.2	62.3	15.4	9.9	1.07	1.11	198.8	0.77	35
40	144	295.2	29.2	296.0	259.7	36.3	12.3%	0.39	2.57	3324	365824	570583	8927	1.28E+10	13.2	8.5	2.55E+09	96.8	62.1	11.6	7.4	1.09	1.16	323.9	1.10	40

UHSC

levels	h	u_{max}	u_x	$u_{y;total}$	$u_{y;structu}$	$u_{y;foundat}$	%	f_e	T	F_d	$N_{total;SLS}$	$N_{total;ULS}$	N_{wind}	$EI_{structure}$	$n_{s;SLS}$	$n_{s;ULS}$	C	$n_{f;SLS}$	$n_{f;ULS}$	Total "Knikgetal" n_{SLS}	n_{ULS}	2nd-order factor SLS	ULS	End result u_y	u.c.	levels
	[m]	[mm]	[mm]	[mm]	[mm]	[mm]		[Hz]	[sec]	[kN]	[kN]	[kN]	[kN]	[kNm²]			[kNm/rad]							[mm]		
20	72	151.2	2.3	22.0	14.5	7.5	34.1%	1.18	0.84	2002	205640	322663	3895	1.25E+10	91.7	58.4	1.35E+09	181.8	115.9	61.0	38.8	1.02	1.03	22.4	0.15	20
30	108	223.2	7.8	75.8	58.1	17.7	23.4%	0.68	1.47	2595	294880	461719	6260	1.70E+10	38.5	24.6	2.06E+09	129.5	82.7	29.7	18.9	1.03	1.06	78.4	0.35	30
40	144	295.2	20.7	206.4	170.2	36.2	17.5%	0.43	2.32	3422	380381	594166	8927	1.96E+10	19.4	12.4	2.56E+09	93.4	59.8	16.0	10.3	1.07	1.11	220.1	0.75	40
50	180	367.2	49.6	494.3	430.7	63.6	12.9%	0.29	3.48	4655	464252	724524	11710	1.98E+10	10.3	6.6	2.98E+09	71.4	45.7	9.0	5.8	1.13	1.21	556.2	1.51	50

Appendix C.2 Displacements, Natural Frequencies & Second Order

Variant 2: Column width: 800 mm

OC

																				Total "Knikgetal"		2nd-order factor		End result		
levels	h	u_{max}	u_x	$u_{y;total}$	$u_{y;structui}$	$u_{y;foundat}$	%	f_e	T	F_d	$N_{total;SLS}$	$N_{total;ULS}$	N_{wind}	$EI_{structure}$	$n_{s;SLS}$	$n_{s;ULS}$	C	$n_{f;SLS}$	$n_{f;ULS}$	n_{SLS}	n_{ULS}	SLS	ULS	u_y	u.c.	levels
	[m]	[mm]	[mm]	[mm]	[mm]	[mm]		[Hz]	[sec]	[kN]	[kN]	[kN]	[kN]	[kNm²]			[kNm/rad]							[mm]		
20	72	151.2	-	103.4	95.7	7.7	7.4%	0.64	1.55	1962	190471	298089	3895	1.90E+09	15.0	9.6	1.31E+09	191.2	122.2	13.9	8.9	1.08	1.13	111.4	0.74	20
25	90	187.2	-	209.8	195.8	14	6.7%	0.47	2.14	2301	228837	357486	5022	2.34E+09	9.8	6.3	1.45E+09	141.1	90.3	9.2	5.9	1.12	1.20	235.4	1.26	25

HSC

																				Total "Knikgetal"		2nd-order factor		End result		
levels	h	u_{max}	u_x	$u_{y;total}$	$u_{y;structui}$	$u_{y;foundat}$	%	f_e	T	F_d	$N_{total;SLS}$	$N_{total;ULS}$	N_{wind}	$EI_{structure}$	$n_{s;SLS}$	$n_{s;ULS}$	C	$n_{f;SLS}$	$n_{f;ULS}$	n_{SLS}	n_{ULS}	SLS	ULS	u_y	u.c.	levels
	[m]	[mm]	[mm]	[mm]	[mm]	[mm]		[Hz]	[sec]	[kN]	[kN]	[kN]	[kN]	[kNm²]			[kNm/rad]							[mm]		
20	72	151.2	-	57.9	50.2	7.7	13.3%	0.84	1.18	2123	190550	298216	3895	3.62E+09	28.6	18.3	1.31E+09	191.1	122.1	24.9	15.9	1.04	1.07	60.3	0.40	20
25	90	187.2	-	116.7	102.3	14.4	12.3%	0.62	1.62	2468	228934	357641	5022	4.47E+09	18.8	12.0	1.41E+09	137.1	87.8	16.5	10.6	1.06	1.10	124.2	0.66	25
30	108	223.2	-	214.9	189.2	25.7	12.0%	0.47	2.12	2868	267480	417331	6260	5.21E+09	13.0	8.3	1.42E+09	98.3	63.0	11.5	7.4	1.10	1.16	235.4	1.05	30

UHSC

																				Total "Knikgetal"		2nd-order factor		End result		
levels	h	u_{max}	u_x	$u_{y;total}$	$u_{y;structui}$	$u_{y;foundat}$	%	f_e	T	F_d	$N_{total;SLS}$	$N_{total;ULS}$	N_{wind}	$EI_{structure}$	$n_{s;SLS}$	$n_{s;ULS}$	C	$n_{f;SLS}$	$n_{f;ULS}$	n_{SLS}	n_{ULS}	SLS	ULS	u_y	u.c.	levels
	[m]	[mm]	[mm]	[mm]	[mm]	[mm]		[Hz]	[sec]	[kN]	[kN]	[kN]	[kN]	[kNm²]			[kNm/rad]							[mm]		
20	72	151.2	-	40.8	33.2	7.6	18.6%	0.95	1.05	1968	196828	308387	3895	5.47E+09	41.8	26.7	1.33E+09	187.5	119.7	34.2	21.8	1.03	1.05	42.0	0.28	20
30	108	223.2	-	148.5	125.1	23.4	15.8%	0.53	1.89	2678	276606	432114	6260	7.88E+09	19.0	12.2	1.56E+09	104.5	66.9	16.1	10.3	1.07	1.11	158.3	0.71	30
35	126	259.2	-	247.8	213.5	34.3	13.8%	0.42	2.38	3139	322577	503280	7555	8.85E+09	13.5	8.6	1.75E+09	86.0	55.1	11.7	7.5	1.09	1.15	271.1	1.05	35

Appendix C.3 Displacements, Natural Frequencies & Second Order

Variant 1: Column width: 1600 mm

FRHSC & OC

levels	h	u_{max}	u_x	$u_{y;total}$	$u_{y;structui}$	$u_{y;foundat}$	%	f_e	T	F_d	$N_{total;SLS}$	$N_{total;ULS}$	N_{wind}	$EI_{structure}$	$n_{s;SLS}$	$n_{s;ULS}$	C	$n_{f;SLS}$	$n_{f;ULS}$	n_{SLS}	n_{ULS}	SLS	ULS	u_y	u.c.	levels
	[m]	[mm]	[mm]	[mm]	[mm]	[mm]		[Hz]	[sec]	[kN]	[kN]	[kN]	[kN]	[kNm²]			[kNm/rad]							[mm]		
30	108	223.2	-	120.8	103.0	17.8	14.7%	-	-	-	284877	444962	6260	9.57E+09	22.5	14.4	2.05E+09	133.3	85.4	19.2	12.3	1.05	1.09	127.4	0.57	30

Appendix D Spandrel Beams

Variant 1: Column width: 1600 mm

Beams l=2m b*h=350*1600

											Required reinforcements													
											Bending moments				Shear									
	N	V_z	M_y	M_z	2nd	N	V_z	M_y	M_z		A_s	A_s	M_u	u.c.	v_{Ed}	θ	$v_{Rd,c}$	$v_{Rd,max}$	v_f	A_{sw}/m		$v_{Rd,s}$	v_{Rd}	u.c.
	[kN]	[kN]	[kNm]	[kNm]	order	[kN]	[kN]	[kNm]	[kNm]		[mm²]	[diameters]	[kNm]		[N/mm²]	[°]	[N/mm²]	[N/mm²]	[N/mm²]	[mm²]		[N/mm²]	[N/mm²]	
30 levels																								
OC	-1387	-1760	-2015	12.95	1.15	**-1590**	**-2017**	**-2309**	**14.84**		2277	4R25+1R20	2308	1.00	3.73	21.8	0.33	3.74		1508	R12-150	4.22	**4.22**	0.88
HSC	-1444	-1899	-2076	9.86	1.08	**-1555**	**-2045**	**-2236**	**10.62**		1924	2R25+3R20	2314	0.97	3.78	21.8	0.53	7.15		1508	R12-150	4.22	**4.22**	0.90
UHSC	-1515	-2039	-2185	8.39	1.06	**-1599**	**-2153**	**-2307**	**8.86**		1924	2R25+3R20	2418	0.95	3.98	45.0	0.74	9.07	4.80	524	R10-300*	0.59	**5.39**	0.74
35 levels																								
HSC	-1679	-2228	-2475	13.42	1.11	**-1868**	**-2479**	**-2754**	**14.93**		3217	4R32	3132	0.88	4.58	21.8	0.53	7.15		1885	R12-125	5.28	**5.28**	0.87
40 levels																								
UHSC	-2074	-2842	-3083	13.68	1.11	**-2298**	**-3149**	**-3416**	**15.16**		2590	2R32+2R25	3629	0.94	5.82	40.0	0.74	8.94	5.72	524	R10-300*	0.70	**6.42**	0.91

* practical reinforcement

Variant 2: Column width: 800 mm

Beams l=2.8m b*h=350*1600

											Required reinforcements													
											Bending moments				Shear									
	N	V_z	M_y	M_z	2nd	N	V_z	M_y	M_z		A_s	A_s	M_u	u.c.	v_{Ed}	θ	$v_{Rd,c}$	$v_{Rd,max}$	v_f	A_{sw}/m		$v_{Rd,s}$	v_{Rd}	u.c.
	[kN]	[kN]	[kNm]	[kNm]	order	[kN]	[kN]	[kNm]	[kNm]		[mm²]	[diameters]			[N/mm²]	[°]	[N/mm²]	[N/mm²]	[N/mm²]	[mm²]		[N/mm²]	[N/mm²]	
20 levels																								
OC	-672	-1085	-1758	6.61	1.13	**-757**	**-1223**	**-1981**	**7.45**		2590	2R32+2R25	2055	0.96	2.26	21.8	0.33	3.74		905	R12-250	2.53	**2.53**	0.89
HSC	-662	-1151	-1773	4.01	1.07	**-706**	**-1228**	**-1892**	**4.28**		2454	5R25	2007	0.94	2.27	21.8	0.53	7.15		905	R12-250	2.53	**2.53**	0.90
UHSC	-760	-1228	-1846	2.74	1.05	**-796**	**-1287**	**-1935**	**2.87**		2277	4R25+1R20	2077	0.93	2.38	45.0	0.74	9.07	4.80	524	R10-300*	0.59	**5.39**	0.44
25 levels																								
HSC	-910	-1527	-2382	5.75	1.10	**-1005**	**-1686**	**-2630**	**6.35**		3217	4R32	2874	0.92	3.12	21.8	0.53	7.15		1131	R12-200	3.16	**3.16**	0.99
30 levels																								
UHSC	-1132	-1739	-2682	4.17	1.11	**-1254**	**-1926**	**-2970**	**4.62**		3217	4R32	2986	0.99	3.56	45.0	0.74	9.07	4.80	524	R10-300*	0.59	**5.39**	0.66

* practical reinforcement

Appendix E.1 Corner Columns

Variant 1: Column width: 1600 mm

Required reinforcements

	Compr./ Tension	N [kN]	V_y [kN]	V_z [kN]	M_y [kNm]	M_z [kNm]	2nd order	N [kN]	V_y [kN]	V_z [kN]	M_y [kNm]	M_z [kNm]	M [kNm]	Bending moments A_s [mm²]	A_s [diameters]	M_u [kNm]	u.c.	Shear	$v_{Ed,max}$ [N/mm²]	θ [°]	$v_{Rd,c}$ [N/mm²]	$v_{Rd,max}$ [N/mm²]	$v_{Rd,f}$ [N/mm²]	A_{sw}/m [mm²]		$v_{Rd,s}$ [N/mm²]	v_{Rd} [N/mm²]	u.c.
30 levels																												
OC	**C**	-17802	631	510	-1080	-253	1.15	-20404	723	585	-1238	-290	1271	37699	30R40	1354	0.94	y	1.34	21.8	0.37	3.74		628	R10-200	2.20	2.20	0.61
	T	736	550	843	640	438		844	630	966	734	502	889			3374	0.26	z	1.79	21.8	0.37	3.74		628	R10-200	2.20	2.20	0.81
HSC	**C**	-18101	-684	508	-1068	-325	1.08	-19493	-737	547	-1150	-350	1202	8875	4R25+22R20	1732	0.69	y	1.36	21.8	0.59	7.15		628	R10-250	1.76	1.76	0.77
	T	491	639	846	645	496		529	688	911	695	534	876			953	0.92	z	1.68	21.8	0.59	7.15		628	R10-250	1.76	1.76	0.96
UHSC	**C**	-18984	700	501	-1097	-357	1.06	-20042	739	529	-1158	-377	1218	8168	26R20	5194	0.23	y	1.37	45.0	0.74	9.07	4.80	335	R8-300*	0.37	5.17	0.26
	T	114	611	771	-864	-635		120	645	814	-912	-670	1132			1181	0.96	z	1.50	45.0	0.74	9.07	4.80	335	R8-300*	0.37	5.17	0.29
35 levels																												
HSC	**C**	-18223	829	669	2297	-530	1.11	-20277	922	744	2556	-590	2623	37699	30R40	2645	0.99	y	1.92	21.8	0.59	7.15		628	R10-200	2.20	2.20	0.87
	T	1770	-931	563	-793	714		1970	-1036	626	-882	794	1187			1664	0.71	z	1.38	21.8	0.59	7.15		628	R10-200	2.20	2.20	0.63
40 levels																												
UHSC	**C**	-23243	-1005	-807	2902	-660	1.06	-24538	-1061	-852	3064	-697	3142	26389	5R40+25R32	5588	0.56	y	1.96	45.0	0.74	9.07	4.80	335	R8-300*	0.37	5.17	0.38
	T	3065	-986	633	-1057	644		3236	-1041	668	-1116	680	1307			1340	0.98	z	1.58	45.0	0.74	9.07	4.80	335	R8-300*	0.37	5.17	0.30

* practical reinforcement

Variant 2: Column width: 800 mm

Required reinforcements

	Compr./ Tension	N [kN]	V_y [kN]	V_z [kN]	M_y [kNm]	M_z [kNm]	2nd order	N [kN]	V_y [kN]	V_z [kN]	M_y [kNm]	M_z [kNm]	M [kNm]	Bending moments A_s [mm²]	A_s [diameters]	M_u [kNm]	u.c.	Shear	$v_{Ed,max}$ [N/mm²]	θ [°]	$v_{Rd,c}$ [N/mm²]	$v_{Rd,max}$ [N/mm²]	$v_{Rd,f}$ [N/mm²]	A_{sw}/m [mm²]		$v_{Rd,s}$ [N/mm²]	v_{Rd} [N/mm²]	u.c.
20 levels																												
OC	**C**	-9539	415	226	-422	-228	1.13	-10748	468	255	-476	-257	540	23323	16R40+4R32	573	0.94	y	0.86	21.8	0.43	3.74		402	R8-250	1.12	1.12	0.77
	T	-56	280	529	-615	-215		-63	315	596	-693	-242	734			1406	0.52	z	1.10	21.8	0.43	3.74		402	R8-250	1.12	1.12	0.98
HSC	**C**	-9568	432	239	-413	-233	1.07	-10211	461	255	-441	-249	506	9817	20R25	801	0.63	y	0.85	21.8	0.69	7.15		402	R8-200	1.41	1.41	0.60
	T	-191	364	577	-551	-319		-204	388	616	-588	-340	679			719	0.94	z	1.14	21.8	0.69	7.15		402	R8-200	1.41	1.41	0.81
UHSC	**C**	-10075	449	235	-413	-254	1.05	-10559	471	246	-433	-266	508	6794	10R25+6R20	2081	0.24	y	0.87	45.0	0.74	9.07	4.80	335	R8-300*	0.37	5.17	0.17
	T	-411	382	557	-546	-325		-431	400	584	-572	-341	666			669	1.00	z	1.08	45.0	0.74	9.07	4.80	335	R8-300*	0.37	5.17	0.21
25 levels																												
HSC	**C**	-13281	569	324	-591	-306	1.10	-14666	628	358	-653	-338	735	19704	8R40+12R32	754	0.97	y	1.16	21.8	0.69	7.15		524	R10-250	1.76	1.76	0.66
	T	596	452	740	-699	-424		658	499	817	-772	-468	903			1124	0.80	z	1.51	21.8	0.69	7.15		524	R10-250	1.76	1.76	0.86
30 levels																												
UHSC	**C**	-16529	602	359	-585	-397	1.11	-18304	667	398	-648	-440	783	16939	9R40+7R32	2093	0.37	y	1.23	45.0	0.74	9.07	4.80	335	R8-300*	0.37	5.17	0.24
	T	1438	415	683	-701	-442		1592	460	756	-776	-489	918			944	0.97	z	1.40	45.0	0.74	9.07	4.80	335	R8-300*	0.37	5.17	0.27

* practical reinforcement

Appendix E.2 Corner Column Slenderness

Variant 1: Column width: 1600 mm

	Compr./ Tension	Input							Determination of λ_{lim}						Determination of k_1 & k_2										Column Slenderness			
															Adjacent beam		Adjacent column					Calculated column						
		N [kN]	M [kNm]	A_s [mm²]	f_{yd} [N/mm²]	A_c [mm²]	f_{cd} [N/mm²]	E_c [N/mm²]	A [-]	ω [-]	B [-]	C [-]	n [-]	λ_{lim} [-]	l_{adj-b} [mm]	EI_{adj-b} [mm⁴]	l_{adj-c} [mm]	EI_{adj-c} [mm⁴]	M_{adj-b} [kNm]	M_{adj-c} [kNm]	θ [rad]	l_{column} [mm]	EI_{column} [mm⁴]	$k_1 = k_2$ [-]	l_0 [mm]	i [-]	λ [-]	u.c.
30 levels																												
OC	**C**	-20404	1271	37699	435	560000	23.3	13333	0.7	1.26	1.87	0.7	1.56	**14.69**	1000	1.59E+15	1000	5.64E+14	939	332	1.18E-03	2000	5.64E+14	0.261	3038	274.8	**11.06**	0.75
HSC	**C**	-19493	1202	8875	435	560000	60.0	26087	0.7	0.11	1.11	0.7	0.58	**14.27**	1000	3.12E+15	1000	1.10E+15	888	314	5.70E-04	2000	1.10E+15	0.261	3038	274.8	**11.06**	0.77
UHSC	**C**	-20042	1218	8168	435	560000	120.0	40000	0.7	0.05	1.05	0.7	0.30	**18.87**	1000	4.78E+15	1000	1.69E+15	900	318	3.76E-04	2000	1.69E+15	0.261	3038	274.8	**11.06**	0.59
35 levels																												
HSC	**C**	-20277	2623	37699	435	560000	60.0	26087	0.7	0.49	1.41	0.7	0.60	**17.73**	1000	3.12E+15	1000	1.10E+15	1937	686	1.24E-03	2000	1.10E+15	0.261	3038	274.8	**11.06**	0.62
40 levels																												
UHSC	**C**	-24538	3142	26389	435	560000	120.0	40000	0.7	0.17	1.16	0.7	0.37	**18.78**	1000	4.78E+15	1000	1.69E+15	2321	821	9.71E-04	2000	1.69E+15	0.261	3038	274.8	**11.06**	0.59

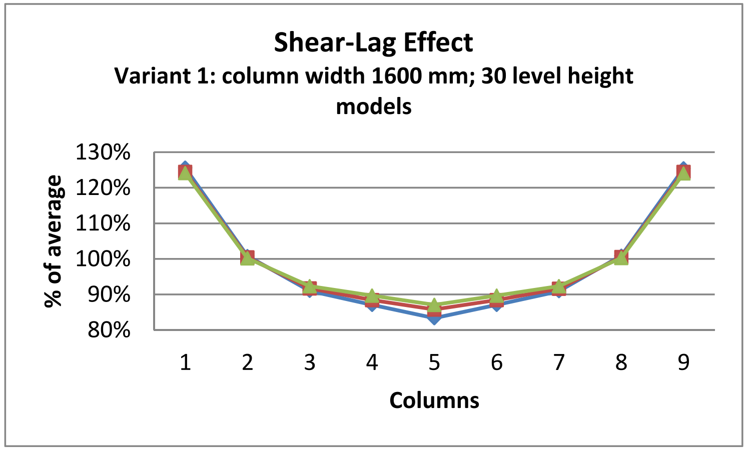
Variant 2: Column width: 800 mm

	Comp Tensi	Input							Determination of λ_{lim}						Determination of k_1 & k_2										Column Slenderness			
															Adjacent beam		Adjacent column					Calculated column						
		N [kN]	M [kNm]	A_s [mm²]	f_{yd} [N/mm²]	A_c [mm²]	f_{cd} [N/mm²]	E_c [N/mm²]	A [-]	ω [-]	B [-]	C [-]	n [-]	λ_{lim} [-]	l_{adj-b} [mm]	EI_{adj-b} [mm⁴]	l_{adj-c} [mm]	EI_{adj-c} [mm⁴]	M_{adj-b} [kNm]	M_{adj-c} [kNm]	θ [rad]	l_{column} [mm]	EI_{column} [mm⁴]	$k_1 = k_2$ [-]	l_0 [mm]	i [-]	λ [-]	u.c.
30 levels																												
OC	**C**	-10748	540	23323	435	280000	23.3	13333	0.7	1.55	2.03	0.7	1.65	**15.48**	1400	1.59E+15	1000	5.64E+14	361	179	6.35E-04	2000	5.64E+14	0.331	3260	388.6	**8.39**	0.54
HSC	**C**	-10211	506	9817	435	280000	60.0	26087	0.7	0.25	1.23	0.7	0.61	**15.44**	1400	3.12E+15	1000	1.10E+15	338	168	3.04E-04	2000	1.10E+15	0.331	3260	388.6	**8.39**	0.54
UHSC	**C**	-10559	508	6794	435	280000	120.0	40000	0.7	0.09	1.08	0.7	0.31	**18.96**	1400	4.78E+15	1000	1.69E+15	340	168	1.99E-04	2000	1.69E+15	0.331	3260	388.6	**8.39**	0.44
35 levels																												
HSC	**C**	-14666	735	19704	435	280000	60.0	26087	0.7	0.51	1.42	0.7	0.87	**14.91**	1400	3.12E+15	1000	1.10E+15	491	244	4.41E-04	2000	1.10E+15	0.331	3260	388.6	**8.39**	0.56
40 levels																												
UHSC	**C**	-18304	783	16939	435	280000	120.0	40000	0.7	0.22	1.20	0.7	0.54	**15.93**	1400	4.78E+15	1000	1.69E+15	523	259	3.07E-04	2000	1.69E+15	0.331	3260	388.6	**8.39**	0.53

Appendix F Shear-lag Effect

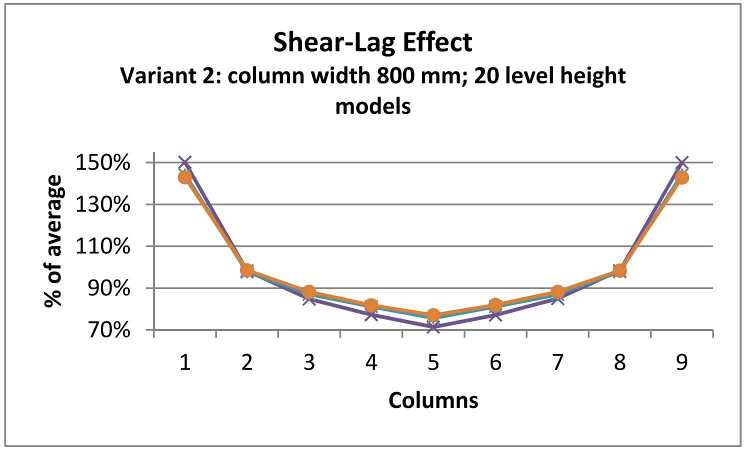
Variant 1: Column width: 1600 mm

Concrete	Levels	Forces in Columns 1	2	3	4	5	6	7	8	9
OC	30	1412	1133	1023	979	937	979	1022	1134	1410
	% of average	125.6%	100.8%	91.0%	87.1%	83.4%	87.1%	90.9%	100.9%	125.4%
HSC	30	1399	1129	1031	994	964	994	1030	1130	1399
	% of average	124.5%	100.4%	91.7%	88.4%	85.8%	88.4%	91.6%	100.5%	124.5%
UHSC	30	1395	1126	1038	1008	979	1008	1038	1127	1394
	% of average	124.1%	100.2%	92.3%	89.7%	87.1%	89.7%	92.3%	100.3%	124.0%



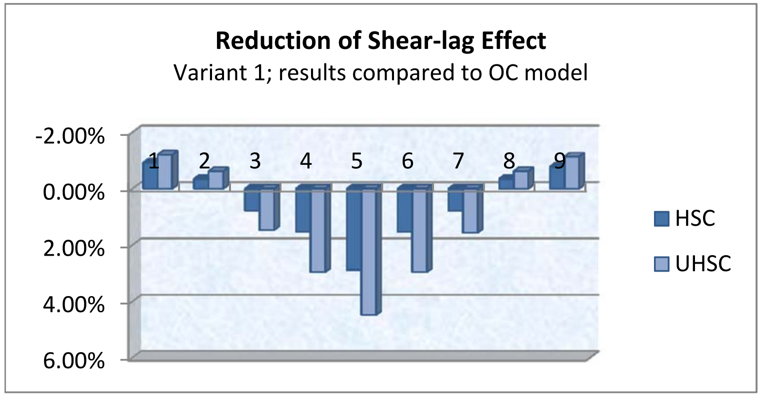
Variant 2: Column width: 800 mm

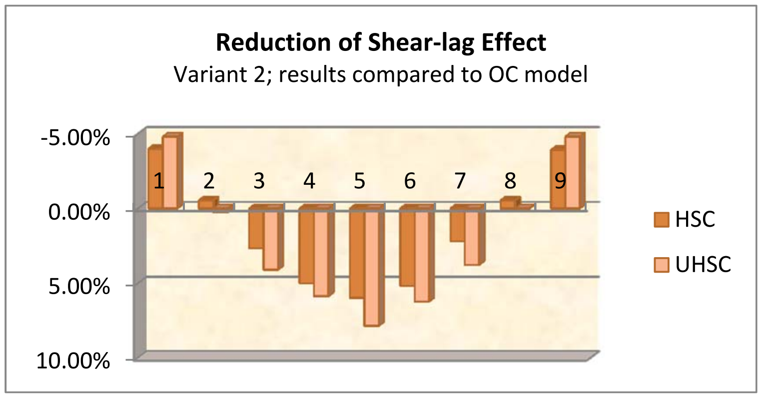
Concrete	Levels	Forces in Columns 1	2	3	4	5	6	7	8	9
OC	20	1125	736	635	579	535	578	637	737	1124
	% of average	150.2%	98.3%	84.8%	77.3%	71.4%	77.2%	85.0%	98.4%	150.1%
HSC	20	1080	732	652	608	567	608	651	733	1080
	% of average	144.2%	97.7%	87.0%	81.2%	75.7%	81.2%	86.9%	97.9%	144.2%
UHSC	20	1071	738	661	613	577	614	661	737	1070
	% of average	143.0%	98.5%	88.3%	81.8%	77.0%	82.0%	88.3%	98.4%	142.9%



Reduction of Shear-lag Effect

Concrete	Levels	1	2	3	4	5	6	7	8	9
HSC	30	-0.92%	-0.35%	0.78%	1.53%	2.88%	1.53%	0.78%	-0.35%	-0.78%
UHSC	30	-1.20%	-0.62%	1.47%	2.96%	4.48%	2.96%	1.57%	-0.62%	-1.13%
HSC	20	-4.00%	-0.54%	2.68%	5.01%	5.98%	5.19%	2.20%	-0.54%	-3.91%
UHSC	20	-4.80%	0.27%	4.09%	5.87%	7.85%	6.23%	3.77%	0.00%	-4.80%





Appendix G Comfort

Variant 1: Column width: 1600 mm

								$S_L(z,n)$				K_s															
	z	c_f	ρ	$I_v(z_s)$	$v_m(z_s)$	$v^2_m(z_s)$	δ_s	$L(z)$	$f_L(z,n)$	f_e	$S_L(z,n)$	$\phi_y = \phi_z$	G_y	G_z	K_s	R^2	R	K_y	K_z	$N_{total;SLS}$	$A_{structure}$	μ_{ref}	σ_a	k_p	$\hat{a}$	$\hat{a}_{max}$	u.c.
	[m]	[-]	[kg/m³]	[-]	[-]	[-]	[-]	[-]	[-]	[Hz]	[-]	[-]	[-]	[-]	[-]	[-]	[-]	[-]	[-]	[kN]	[m²]	[kN/m²]	[-]	[-]	[m/s²]	[m/s²]	[-]
30 levels																											
OC	108	1.075	1.25	0.0930	19.4	376.4	0.10	202.81	4.81	0.46	0.0159	9.82	0.50	0.28	0.0893	0.0701	0.265	1.00	1.67	284770	518.4	549.3	0.038	4.0	**0.151**	0.27	0.56
HSC	108	1.075	1.25	0.0930	19.4	376.4	0.10	202.81	6.27	0.60	0.0134	12.80	0.50	0.28	0.0580	0.0383	0.196	1.00	1.67	284951	518.4	549.7	0.028	4.0	**0.112**	0.25	0.45
UHSC	108	1.075	1.25	0.0930	19.4	376.4	0.10	202.81	7.11	0.68	0.0123	14.51	0.50	0.28	0.0468	0.0285	0.169	1.00	1.67	294880	518.4	568.8	0.023	4.0	**0.093**	0.24	0.39
35 levels																											
HSC	126	1.075	1.25	0.0904	19.4	376.4	0.10	223.68	5.53	0.48	0.0145	10.24	0.50	0.28	0.0835	0.0599	0.245	1.00	1.67	325388	518.4	627.7	0.030	4.0	**0.119**	0.27	0.44
40 levels																											
UHSC	144	1.075	1.25	0.0883	19.4	376.4	0.10	243.49	5.40	0.43	0.0148	9.18	0.50	0.28	0.0989	0.0721	0.269	1.00	1.67	380381	518.4	733.8	0.027	4.0	**0.109**	0.28	0.39

Variant 2: Column width: 800 mm

								$S_L(z,n)$				K_s															
	z	c_f	ρ	$I_v(z_s)$	$v_m(z_s)$	$v^2_m(z_s)$	δ_s	$L(z)$	$f_L(z,n)$	f_e	$S_L(z,n)$	$\phi_y = \phi_z$	G_y	G_z	K_s	R^2	R	K_y	K_z	$N_{total;SLS}$	$A_{structure}$	μ_{ref}	σ_a	k_p	$\hat{a}$	$\hat{a}_{max}$	u.c.
	[m]	[-]	[kg/m³]	[-]	[-]	[-]	[-]	[-]	[-]	[Hz]	[-]	[-]	[-]	[-]	[-]	[-]	[-]	[-]	[-]	[kN]	[m²]	[kN/m²]	[-]	[-]	[m/s²]	[m/s²]	[-]
20 levels																											
OC	108	1.075	1.25	0.0930	19.4	376.4	0.10	202.81	6.69	0.64	0.0128	13.66	0.50	0.28	0.0519	0.0329	0.181	1.00	1.67	190471	518.4	367.4	0.039	4.0	**0.155**	0.23	0.67
HSC	108	1.075	1.25	0.0930	19.4	376.4	0.10	202.81	8.78	0.84	0.0107	17.93	0.50	0.28	0.0320	0.0170	0.130	1.00	1.67	190550	518.4	367.6	0.028	4.0	**0.111**	0.21	0.53
UHSC	108	1.075	1.25	0.0930	19.4	376.4	0.10	202.81	9.93	0.95	0.0099	20.27	0.50	0.28	0.0255	0.0125	0.112	1.00	1.67	196828	518.4	379.7	0.023	4.0	**0.092**	0.20	0.46
25 levels																											
HSC	126	1.075	1.25	0.0904	19.4	376.4	0.10	223.68	7.15	0.62	0.0123	13.23	0.50	0.28	0.0549	0.0333	0.182	1.00	1.67	228934	518.4	441.6	0.031	4.0	**0.126**	0.23	0.55
30 levels																											
UHSC	144	1.075	1.25	0.0883	19.4	376.4	0.10	243.49	6.65	0.53	0.0129	11.31	0.50	0.28	0.0713	0.0453	0.213	1.00	1.67	276606	518.4	533.6	0.030	4.0	**0.119**	0.26	0.46

Appendix H Comparison with Rules of Thumb

Variant 1: Column width: 1600 mm

		Displacements			Shear Force in Spandrel Beam			Normal Force in Corner Column		
		RoT	Model		RoT	Model		RoT	Model	
	levels	$u_{y;structure}$	$u_{y;structure}$	Δ	V_{beam}	V_{beam}	Δ	N_{column}	N_{column}	Δ
		[mm]	[mm]		[kN]	[kN]		[kN]	[kN]	
OC	20	49.0	42.6	-13.1%	549	438	-20.3%	1438	1230	-14.4%
OC	30	144.4	171.6	18.9%	875	711	-18.7%	2755	2778	0.8%
HSC	30	83.4	88.6	6.2%	875	744	-15.0%	2755	2751	-0.1%
HSC	35	127.2	155.8	22.5%	1053	894	-15.1%	3537	3113	-12.0%
UHSC	40	100.9	170.2	68.7%	1242	1102	-11.3%	4400	4185	-4.9%

Variant 2: Column width: 800 mm

		Displacements			Shear Force in Spandrel Beam			Normal Force in Corner Column		
		RoT	Model		RoT	Model		RoT	Model	
	levels	$u_{y;structure}$	$u_{y;structure}$	Δ	V_{beam}	V_{beam}	Δ	N_{column}	N_{column}	Δ
		[mm]	[mm]		[kN]	[kN]		[kN]	[kN]	
OC	20	114.6	95.7	-16.5%	549	443	-19.4%	1584	1355	-14.5%
HSC	20	66.2	50.2	-24.2%	549	472	-14.1%	1584	1323	-16.5%
HSC	25	117.9	102.3	-13.3%	705	629	-10.8%	2232	2084	-6.6%
UHSC	30	103.0	125.1	21.5%	875	699	-20.1%	2977	2799	-6.0%