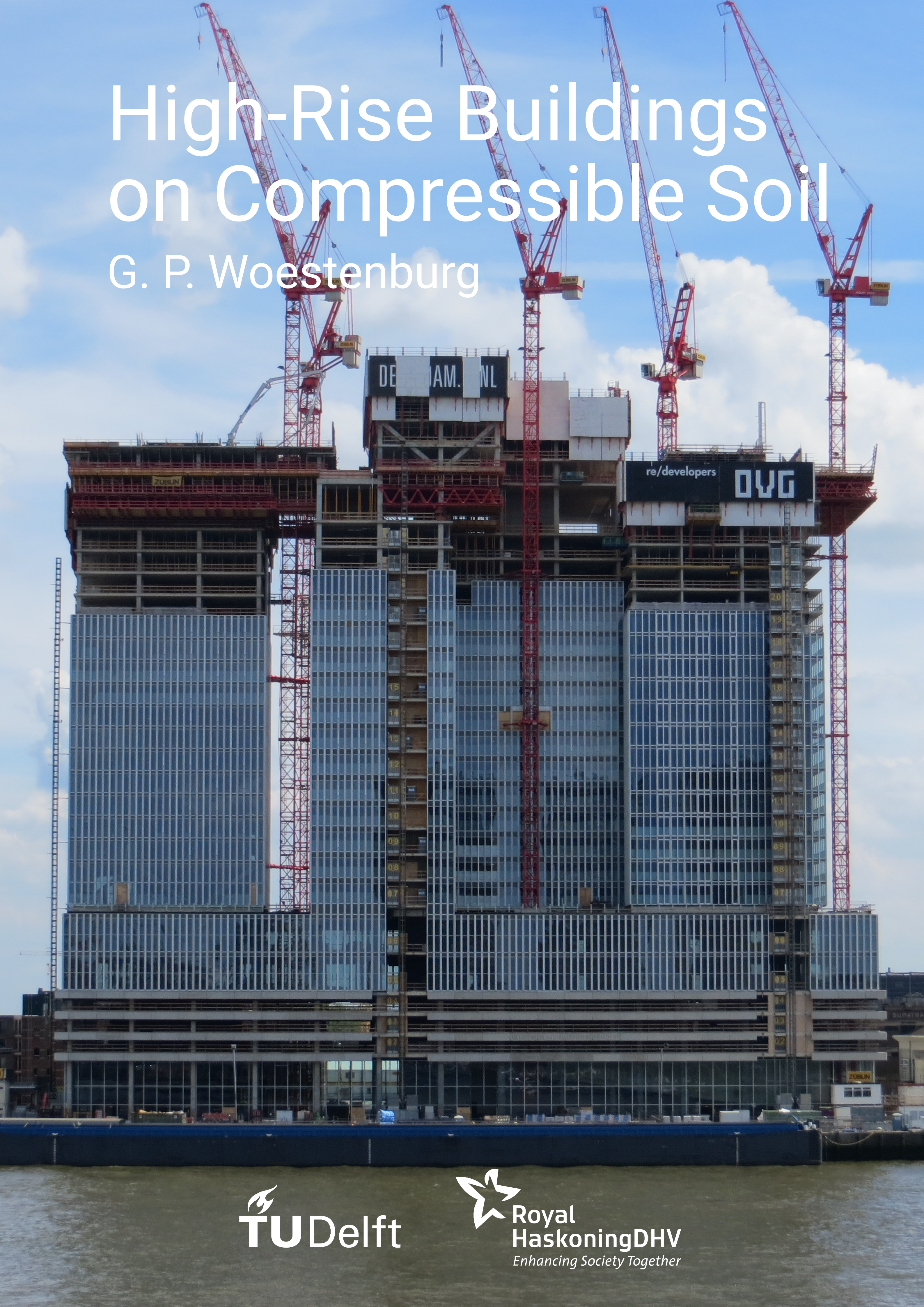
High-Rise Buildings on Compressible Soil

G. P. Woestenburg



TU Delft

Royal HaskoningDHV
Enhancing Society Together

Front Page: De Rotterdam in Aanbouw, June 2012, Digital Photograph,
https://nl.m.wikipedia.org/wiki/Bestand:De_rotterdam_in_aanbouw_juni_2012.JPG

High-Rise Buildings on Compressible Soil

Research on the Structural Behavior and Substantiation of the Mitigating Measures

by

G. P. Woestenburg

to obtain the degree of Master of Science
at the Delft University of Technology

Student number:	4207432	
Thesis committee:	Prof. ir. R. Nijsse	TU Delft, Chairman
	Dr. ir. K. C. Terwel	TU Delft
	Dr. ir. M. Korff	TU Delft
	D. Snijders, MSc	Royal HaskoningDHV

Summary

Cities like Amsterdam and Rotterdam are located on top of a compressible clay layer. The compressible layer referred to is situated under the first foundation layer. Most of the buildings in these cities are founded on the foundation layer above the compressible layer. Therefore this compressible layer can induce problems when the soil pressure increases significantly due to the weight of the high-rise buildings. This deformation can cause inclination of the floors in the superstructure, damage to the structural and non-structural elements of the building. Therefore the structural design of a building must appropriately be designed to respond or resist these deformations.

This research aims to get a better understanding of the structural behavior of high-rise buildings on compressible soil and to get acquainted with the possible mitigating measures to exclude unacceptable deformations in the structural design. Whether the mitigating measures are possible depends on several factors, and all of the mitigating measures have advantages and disadvantages.

The deformation, which causes the problems in the building itself, is the differential deformation (sagging). This deformation is unequal over the entire plot of the building. The uniform deformation of the building does not induce problems in the structure itself, but it can damage the adjacent buildings because the soil under these buildings will deform (hogging). Column shortening and the construction sequence do also affect the vertical deformation in the structure. The column shortening only decreases the problem due to differential deformation because it causes rotation of the horizontal elements in the opposite direction compared to the rotation due to differential deformation of the soil. The construction sequence decreases the problem for the higher levels because the deformation of the compressible layer start occurring during construction, and these levels are constructed horizontally independent on the amount of deformation of the soil.

For a better understanding of the problem, the deformation of the substructure is investigated for three locations (Amsterdam, Rotterdam, and Utrecht). The weight of the building is related to the amount of settlement of the compressible layer. For this research, a building is designed with three alternatives for the primary structure; one in concrete, one in steel, and the third is a combination of both (hybrid). For these alternatives at the three locations, the deformations of the substructure are calculated, taking into account the Soil-Structure Interaction (SSI) and the construction sequence. The total deformation consists of deformation of the foundation piles and deformation of the soil under the foundation piles. The maximum total settlement occurs at Rotterdam, for the concrete alternative, the total settlement is 0.21 m under the core. This settlement is comparable to the calculated settlement of the building De Rotterdam. The settlement of the compressible layer has a time-dependent behavior, which takes decennia to reach the final settlement.

For this research, the structural behavior is analyzed with the program SCIA Engineer. The settlement of the compressible soil due to the weight of the building are calculated with the program D-Settlement and back introduced into the SCIA model to investigate the deformations in the building. In the SCIA model, the construction stages of the high-rise building are modeled because the soil will start to deform during the construction of the building. This model shows that, especially for the lower levels of the buildings, the deformation exceeds the limits for deformation. The problem is amplified by the geometry of the building, because of the setback. The deformations at the higher levels are not exceeding the assumed limits.

For this research, the following mitigating measures are considered:

- Changing the construction sequence
- Increase the length of the foundation pile
- Application of a camber
- Increase the stiffness of the foundation slab
- Increase the stiffness of the superstructure
- Application of a jack-down system

The primary factors which are affecting the applicability of the mitigating measures are the amount of settlement, the soil structure, and the time-dependent behavior. By a significant settlement of the soil, the mitigating measures of applying a camber and increasing the stiffness of the structure are not possible or less favorable. The soil structure determines if increasing the length of the foundation piles is possible, this mitigating measure put the foundation piles in the second foundation layer, which is situated under the compressible layer. With this measure, the problem of the compressible layer is solved, but it is an expensive mitigating measure. Adjusting the construction sequence depends on the time-dependent behavior of the settlement. It can be an option when the settlement happens mostly during construction. When the settlement of the soil takes longer than the construction time, the jack-down system can be applied to adjust the structure when needed, even when the building is finished. When comparing the applicability of the mitigating measures, the jack-down system is the most favorable. This measure can be applied in all cases. The other mitigating measures do have particular boundary conditions or increase the total cost of the project.

The advantages and the disadvantages of the mitigating measures are pointed out based on a multi-criteria analysis. The mitigating measures are ranked for the considered aspects; material use, the functionality of the building, effect on the surroundings, construction time, conventionality, and adaptability of the measures. The mitigating measures of changing the construction sequence, application of a camber, and the jack-down system are the most favorable based on these six aspects.

The application of the jack-down system is definitely worth considering when a high-rise building is built on compressible soil. The other mitigating measures can be applied too, however, only with very specific conditions. This makes the jack-down system the most favorable mitigating measure.

Contents

Acronyms

CPT Cone Penetration Test

DSP Drilling Sample Profile

OCR Over Consolidation Ratio

SLS Serviceability Limit State

SSI Soil-Structure Interaction

ULS Ultimate Limit State

List of Figures

List of Tables

Acknowledgements

To finalize my master Building Engineering, I present to you the result of my graduation project. The subject of high-rise buildings on compressible soil is an exciting and challenging topic. After a difficult start defining the research, I enjoyed the structural calculations. I only hoped for another ending of my master, the lock-down due to the coronavirus did not make it any easier. Nevertheless, I am looking forward to start working as a structural engineer.

I could not be able to finish this thesis without the support of many people, therfore I want to highlight a few.

First of all, I want to express my gratitude to my graduation committee for their knowledge and enthusiasm, which guided me through the graduation project. I want to thank you for the helpful discussions and answers to all my questions during my research, and especially for the meetings we had. After each meeting, I found the motivation to go on with the research. I want to thank my daily supervisor Dennis Snijders from Royal HaskoningDHV, for introducing me to the subject and for your expertise. Also, Karel Terwel and Mandy Korff, thank you for your help and guidance throughout this research and Rob Nijsse for being the chairman of the committee.

Secondly, I am thankful for all my friends in and outside Delft. I enjoyed the distractions from this research. I want to thank Jeffrey for the support during this project and also my roommates, Geert and Jurre, thank you for the time we lived together and especially for the delicious dinners to end the working days during this period.

This acknowledgement is not complete without thanking my family. I want to thank my parents and my sister for their undoubtedly support and confidence in me. I thank you for all the opportunities and support you gave me to finish this master. Last but not least, I want to thank my girlfriend, Annabella, for the time we spend together and also for the support you gave me during this graduation.

G. P. Woestenburg
Delft, April 2020

1 Introduction

In cities of the Netherlands, more and more high-rise buildings are being built. The demand for a living- and working space in these cities is becoming higher, and therefore the high-rise buildings are the solution. One of the reasons why this country is behind on the world market for high-rise buildings is the soil conditions on which some of our cities are built. Amsterdam and Rotterdam are built on top of a compressible clay-layer. Problems arise when the compressible layer is situated underneath the foundation layer.

Deformation of the compressible layer could result in inclination of the floors and damage to structural and non-structural elements. These consequences will occur when the structure of a building is not sufficiently designed with mitigating measures to reduce or adapt to the deformations of the underlying soil. The building De Rotterdam (shown in Figure 1.1) is constructed on top the foundation layer with a compressible layer underneath it. The structural engineers of Royal HaskoningDHV designed the structure in a way that the structure can adapt the deformation of the soil with a jacking system which is integrated in the structural design. Buildings of First Rotterdam, De Zalmhaventoren and New Orleans are constructed with mitigating measures in the structural design to resist or adapt the deformation of the compressible layer too.

The high-rise visions of the cities Amsterdam, Rotterdam, and Utrecht show that the municipalities of these cities have the desire for more high-rise buildings [9] [13] [23]. Amsterdam and Rotterdam are cities which are located on top of a compressible clay-layer and the soil structure of Utrecht is mostly sand-layers. For the future buildings, it would be of help to have a better understanding of how high-rise buildings are affected by a compressible layer underneath the foundation layer.

Figure 1.1: Picture of De Rotterdam [5]

1.1. Problem Statement

Designing a high-rise building on compressible soil is a complicated task. The deformations of the soil can have a significant effect on the structural behavior. Usually, the structural engineer focuses on the structure itself, but when designing a high-rise building on compressible soil, it is essential to understand what is happening and how the structure is affected by the deformations.

The complexity of the problem is due to the Soil-Structure Interaction (SSI), which implies the interaction between the stiffnesses of the soil and the structure. Next to the SSI, the construction sequence influences the problem and discontinuities in the shape of the building can amplify the deformations. The combination of these aspects makes the design process difficult. For the design process of a building it is favorable for the structural engineers to make a good initial design. In the beginning of the design process changes in the design are relative easily made. The further the design process is, the more difficult it becomes to make significant changes in the structural design. The initial design of a structure must be designed in a short period and without many specifics known about the soil conditions, the design of the foundation and the structural behavior.

1.2. Objective

The objective of this research is to get a better understanding of the problem of a high-rise building on compressible soil and to investigate what possible mitigating measures are to ensure that the deformation limits of the building are not exceeded. With this knowledge better decisions on the structural design can be made in the early design phase of the project, which can improve the final structural design.

1.3. Research Questions

The main research question is:

*What are possible mitigating measures in the structural design for a **high-rise building** with **discontinuities in the geometry** to reduce the risk of **unacceptable deformations** induced by settlement of compressible soil and what are the advantages and disadvantages of these mitigating measures?*

The key aspects of the above-mentioned research question are explained in detail:

- This research focusses on a **high-rise building** with a height of around 150 m.
- **Discontinuities in the geometry** are amplifying the problem of deformations. In this research the discontinues of a setback and a cantilevering part are included in the design of the building.
- **Unacceptable deformations** are deformations where the limits are exceeded.

This research first answers two sub-questions, which will help to understand the problem and provides an indication of how significant the deformations of the soil are. Therefore the proposed mitigating measures can be understood better. The sub-questions are:

1. *What type of deformation is affecting the building due to deformation of the soil, and are there additional factors influencing this deformation?*
2. *How big is the total deformation of the substructure for a high-rise building when the building is constructed in Amsterdam, Rotterdam, and Utrecht?*

1.4. Methodology

In this section the outline of this research is described. Per chapter is described what questions will be answered.

Sub-question 1

Chapter 2 - *Literature Research*

In this chapter the literature research is explained. The type of deformations inducing the problems is determined and the limits for the deformations are established.

Section 2.1: What type of deformations in the building are induced by soft soil?
Section 2.2: What causes these type of deformations?
Section 2.3: What are the limits for these deformations?
Section 2.4: How is the construction sequence affecting these deformations?
Section 2.5: How big are the soil deformations at De Rotterdam?

Sub-question 2

Chapter 3 - *Structural Design*

In this chapter the structures are designed for which the deformations of the substructure are calculated in the next chapter.

Section 3.1: What is the geometry and what are the starting points of the building?
Section 3.2: What are the loads and materials used for the structure?
Section 3.3: What is the structural design of the building?
Section 3.4: What is the design of the foundation?
Section 3.5: What is the building sequence of the structure?

Chapter 4 - *Deformations of the Substructure*

In this chapter the deformations of the substructure are investigate and calculated.

Section 4.1: What is the subsoil structure of the locations?
Section 4.2: What is the theory behind settlement of the soil?
Section 4.3: How does the Soil-Structure Interaction work and how is this modeled?
Section 4.4: How big are the deformations of the substructure?

Main question

Chapter 5 - *Mitigating Measures*

In this chapter the deformation of the structure are analyzed due to the deformation of the substructure and the considered mitigating measures are explained.

Section 5.1: Are the deformations of the structure exceeding the limits?
Section 5.2: How are the considered mitigating measures working?

Chapter 6 - *Multi-criteria Analysis*

In this chapter the advantages and disadvantages of the mitigating measures based on a multi-criteria analysis are discussed .

Section 6.1: What are the aspects of the multi-criteria analysis?
Section 6.2: How are the mitigating measure ranked on the multi-criteria analysis?

Final Remarks

Chapter 7 - *Discussion, Conclusion and Recommendation*

In this chapter the final conclusion of the research is stated, together with the discussion and recommendation for further research.

1.5. Demarcation of the Research

This research focuses on the long-term deformations of the soil and is done with publicly available information. Therefore the following aspects are left out or simplified in this research:

1. For this research **wind** is taking into account for the structural design of the superstructure. The windload is neglected for the design of the foundation, the calculations of the settlement of the subsoil, and the design of the mitigating measures. For these aspects only the vertical loading of the building is taking into account. Wind vibrations and the therefore needed damping in the foundations are out of the scope of this research.

2. Unequal deformation of the soil can cause **tilting** of the building. Tilting of the building depends on the center, the stiffness of the foundation, and the stiffness of the soil. Tilting is left out of scope because it will increase the difficulty of this research, and in addition, it is very project-specific.

3. **Subsoil structures** used in this research are the result of combining an available and suitable Cone Penetration Test (CPT) and Drilling Sample Profiles (DSP) to one soil profile. This means that the locations of the used CPT and DSP can deviate. Therefore the subsoil structures used are fictive.

4. In this research, **mitigating measures for the structural system** are investigated. Any adjustments to the finishes of the building to increase the deformations limits are out of scope for this research.

2

Literature Research

2.1. Deformations of the Structure

The primary structure of a building transfers all the applied loads to the underlying soil. The definition in the Eurocode defines that a structure will sustain all the actions and influences during its intended lifetime and meet the specified requirements [18]. The Eurocode makes a distinction between the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS).

First of all, the safety of people and the structures need to be guaranteed. The limit state which regards the safety is called the ULS. The structure has to be designed to exclude loss of stability, collapse due to excessive deformations of the structure or the foundation and collapse due to time-dependent effects in the ULS [18]. Additionally, the function of the building under normal use, the comfort of the people, and the appearance of the building need to be guaranteed [18]. The limit state regarding these aspects is the SLS. In the SLS, the structure needs to resist deformations which negatively affect the functioning of the building and will damage structural elements, non-structural element, and finishes of the building.

In buildings deformations are inevitable, but when the structure is adequately designed, the deformations are so small that it will happen unnoticed. The following types of deformation can be distinguished:

- **Elastic deformation** is a deformation which deforms back to its initial position when the load is removed. The elastic deformations will happen instantly when the load is applied on the structure.
- **Plastic deformation** is a deformation which remains when the load is removed.
- **Time-dependent deformation** is a deformation that happens over time. The structural material of concrete is susceptible to time-dependent deformations like shrinkage, relaxation, and creep.

These type of deformations can happen in the structural elements of the building under loading. The deformation of the subsoil has a time-dependent behavior as well and will be explained in Subsection 4.2.2.

For the total building, five types of movement can be distinguished. The possible movements of the building are explained below and shown in Figure 2.1:

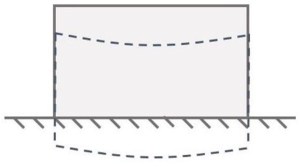
- **Uniform** means that the entire structure moves vertically. This type of movement does not influence the building itself, only the connection with the surroundings of the building will experience this deformation.
- **Shear** is movement in horizontal direction. Shear movement can be the result of the wind-load on the building. The structure must have enough stiffness to resist the wind-load and keep the movement between the corresponding limits.

- **Differential Deformation (Sagging)** is unequal vertical deformation of the building. Differential deformation can occur when, for example, the stability core of the building is heavier than the rest of the building. Therefore the middle of the building settles more compared to the outside of the building.
- **Tilting** means that the whole structure tilts. The structure does not experience this deformation, but the second-order effect can significantly increases, which can damage the structure.
- **Rotation** can happen under the wind load. The building will rotate around the vertical axis.

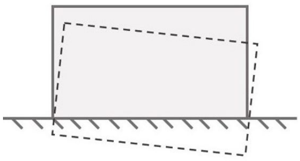
For this research the vertical deformations due to the compressible subsoil are of interest. The deformations which occur due to the compressible subsoil and induce the problems in the structure is the differential deformation. Shear and rotation movements of the building are induced by wind, these deformations are out of the scope of this research. Settlement of the soil can also induce tilting of the building, the design of the foundation must have enough rotation stiffness to resist tilting of the building. In the demarcation of this research in Section 1.5 is explained that tilting is left out of the scope for this research. Therefore the focus is on the uniform and differential deformation.

(a) Uniform

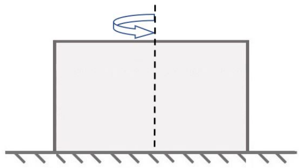
(b) Shear



(c) Differential (Sagging)



(d) Tilting



(e) Rotation

Figure 2.1: Movements of the building

2.2. Causes of the Deformations

In the previous section is explained that the focus is on differential and uniform deformation of the building. The deformation of the foundation induces these vertical deformations. The vertical deformation in the superstructure is influenced by column shortening and by the geometry of the building. In the following subsections these three causes are explained more in detail.

2.2.1. Deformation of the Foundation

The deformation of the foundation influences the entire building. The function of the foundation is to transfer the forces of the structure to the soil. The total deformation of the foundation consist of two parts. The first part considers the settlement and deformation of the foundation piles of the building. The second part regards the deformation of the soil under the foundation of the building. The total deformation of the foundation is the sum of the two parts, as shown in Formula 2.1

$$s_{tot} = s_1 + s_2 \tag{2.1}$$

In Formula 2.1 is s_{tot} the deformation of the foundation, s_1 is the part of the deformation initiated by the displacement and deformation of the foundation piles, s_2 is the part of the deformation due to the deformation of the soil under the foundation of the building.

Figure 2.2 shows the differential settlement. In this schematic drawing is the settlement of the core bigger compared to the rest of the building, which results in rotation of the floors.

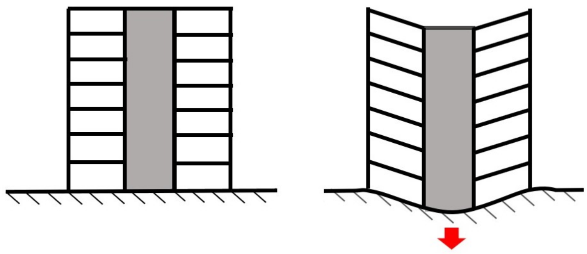


Figure 2.2: Schematic drawing of deformation of the foundation

The following aspects can cause the differential deformation of the foundation [14]:

- **Difference in foundation pressure**: The shape of the building can induce an inhomogeneous soil pressure at the foundation, an example is the stability cores in high-rise buildings, which results in a more heavily loaded area underneath the core compared to the rest of the foundation.
- **Difference in stiffness of the soil**: Differential deformations can be caused by difference in stiffness of the soil over the area of the building.

Further in this research, the soil structure is assumed to be similar over the building plot, resulting in an equal stiffness of the subsoil. The difference in foundation pressure for the high-rise building is considered in this research because of the concrete stability core.

2.2.2. Column Shortening

The structural element shortening can also cause differential deformation in the building. The phenomenon of shortening of the structural element is proportional to the force in the structural element. However, also properties non-related to the force like shrinkage can induce a shortening of the structural element. In Figure 2.3 is the differential deformation caused by column shortening shown. In this Figure can be observed that the floor is inclined due to the column shortening.

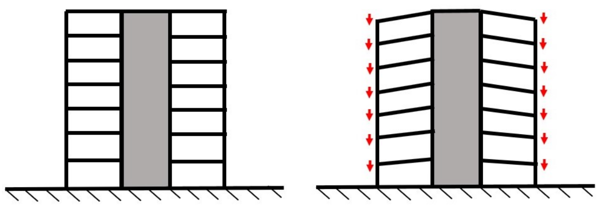


Figure 2.3: Schematic drawing of column shortening

The components which induce the shortening of the structural elements are:

- **Elastic deformation**: This component of the deformation is only dependent on the amount of stress in the structural element. The elastic deformation of the structural element can be explained by Hooke's Law combined with the spring stiffness of a structural element [17]:

$$\Delta L = \frac{F}{EA} * L \tag{2.2}$$

  In Formula 2.2 is the shortening or elongation of the element denoted by ΔL, F is the internal force in the element, L is the length of the element, A is the cross-sectional area of the element, and E is the E-modulus of the material. This formula can be used in the low deformation elastic range [17].

- **Creep**: Creep can be described as the tendency of concrete to continue to compress over time under a sustained load [17]. The creep component of the deformation of a concrete structural element is dependent on the stress, loading history, ratio of reinforcement and volume-to-surface ratio [10]. The creep deformation of concrete $\varepsilon(\infty, t_0)$ at time $t = \infty$ for a constant compressive stress σ_c applied at the concrete age t_0 is given by:

$$\varepsilon(\infty, t_0) = \varphi(\infty, t_0) * \frac{\sigma_c}{E_c} \tag{2.3}$$

  The creep deformation becomes non-linear when the compressive stress exceeds the value $0.45 f_{ck}(t_0)$. In this case the non-linear notional creep coefficient should be obtained with the following formula:

$$\varphi_{nl}(\infty, t_0) = \varphi(\infty, t_0) * e^{1.5(k_\sigma - 0.45)} \tag{2.4}$$

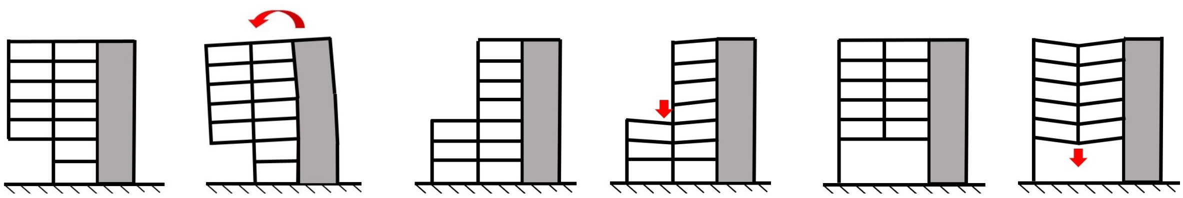
  In Formula 2.4 is k_σ noted as the stress-strength ratio $\sigma_c / f_{ck}(t_0)$ with $f_{ck}(t_0)$ as the characteristic concrete compressive strength at the time of loading.

- **Shrinkage**: This component represents the decrease in the volume of concrete during hardening and drying [17]. The shrinkage component of a concrete structural element is dependent on the ratio of reinforcement, volume-to-surface ratio, and the environmental conditions [10]. The shrinkage behavior of the material concrete is high in the early stages and will decrease to the final value of shrinkage over time. The component shrinkage is not dependent on the stress-level in the structural element.

- **Temperature difference**: The temperature difference has also impact on the elongation.

2.2.3. Discontinuities of the Building

Discontinuities in the geometry of the building can amplify the differential deformation. The geometry of the building determines the load distribution, which affects the deformation of the foundation and the column shortening. These discontinuities can amplify the problem but can not be considered as a cause. The structural engineer will design the construction with the geometry known so that the structure will have enough strength and stiffness. Buildings with these discontinuities are more susceptible to differential column shortening and deformation of the foundation when not adequately considered. Figure 2.4 shows the discontinuities which can influence the vertical deformation of the building.



(a) Cantilever (b) Setback (c) Change in Grid

Figure 2.4: Schematic drawing of discontinuities in the geometry

Due to a cantilevering part of the building, the building tends to lean towards the cantilever, as shown in Figure 2.4a. In the setback, differential deformations can occur since the middle column is more loaded, as shown in Figure 2.4b. The change of grid can amplify unequal deformations in the structure too, as shown in Figure 2.4c. In Chapter 3 the discontinuities of a setback and a cantilever are incorporated in the design of the building for this research.

2.3. Limitations of the Displacements

The limitations for deformations of the structure are agreed upon in the design process of a building. In this research, the focus is on the vertical deformation of the primary structure like the floor deformation and the facade support deformation. The deformations can damage multiple parts of the building, all the parts have their limit. The following limits are considered for the structural design: the limit for lateral deformation, the floor deflection, the deflection of the facade supports, the relative rotation of the foundation, and the allowed deformations of the surroundings. In the next subsections the limitations are explained.

For this research, the deformations are regarded as unacceptable when the deformation limits are exceeded.

2.3.1. Lateral Deflection

The lateral deformation limit is essential for the design of the stability cores for the alternatives of the primary structure. This limitation are used in order to determine the dimensions of the stability core, and therefore the weight of the core. The definition of the lateral movement is shown in Figure 2.5.

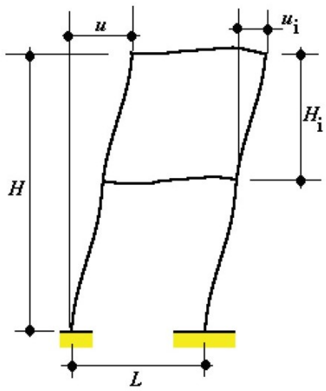


Figure 2.5: Definition of lateral deformation [6]

The limit of the maximum lateral deflection is assumed as:

$$u_{max,lateral} = \frac{H}{500} \tag{2.5}$$

In Formula 2.5 is $u_{max,lateral}$ the limit for the lateral deflection and H is the total height of the building.

2.3.2. Floor Deformation

In the Eurocode, a definition of the vertical deformation is stated. In this definition is the vertical deformation divided into w_1, as the short-term part due to the long-term loading, w_2, as the long-term part due to permanent loading, and w_3, as the deformation due to variable loading. The summation of w_1, w_2 and w_3 is the total deformation w_{tot}. These deformation are shown in Figure 2.6. In the schematic drawing is the camber of the floor w_c shown too. The w_{max} is the vertical deformation with considering the camber of the floor.

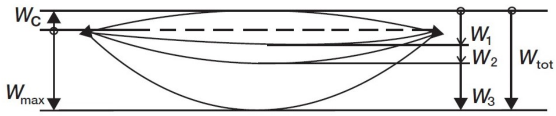


Figure 2.6: Schematic drawing of the deformation of a beam [18]

For simplification the total floor deflection is assumed as:

$$w_{floor} = \frac{l_{rep}}{500} \tag{2.6}$$

In Formula 2.6 is l_{rep} the span of the floor.

2.3.3. Facade Deformation

The primary structure supports the facade elements. For this interface, no strict limit for the facade movement is stated. This limit will be established in the design meeting and can vary between projects.

For this research is the limit for the deflection of the facade supports assumed to be 10 mm in the vertical direction. This limit is used for the project of De Rotterdam where Royal HaskoningDHV was acting as the structural engineer.

$$u_{facade} = 10mm \tag{2.7}$$

u_{facade} is the maximal vertical deflection of the facade supporting element. This limit is independent of the length of the span of the supporting element.

2.3.4. Differential Deformation

In the previous two subsections are limits for the vertical deformations of the floors and the facade supporting elements stated. As described in Chapter 3, the superstructure and the finishes of a building will only experience the differential deformations of the structure. In literature and applicable regulations for buildings, no limit for differential deformation in the superstructure are stated. Therefore, the limits for vertical deformations are used to establish limits for differential deformations. In this research, the differential deformation of the structure will be measured with the rotations of the floor elements and the facade supporting elements.

The limits for vertical deformation are used to derive the rotation limit with the help of the beam formulas. The beam formulas are depending on the structural system, further in this research becomes clear that the floor will be clamped to the stability core and columns. Therefore the floor can be schematized as followed:

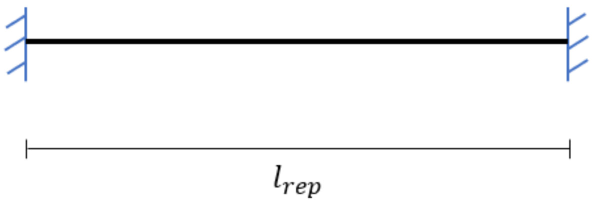


Figure 2.7: Schematic drawing of the floor system

Figure 2.8 shows how the floor will deform.

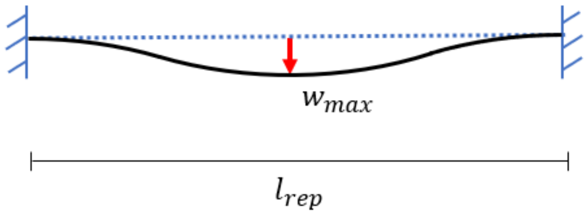


Figure 2.8: Schematic drawing of the deformation of the floor system

Figure 2.9 shows the floor deflection of the left side of the beam with an imposed deformation.

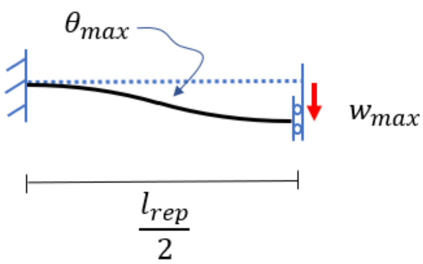


Figure 2.9: Schematic drawing of the floor deflection corresponding to the beam formula

The allowed deflection of the floor and facades are known, so the limit for the rotation can be calculated with Formula 2.8:

$$\theta_{max} = \frac{3}{2} * \frac{w_{max}}{0.5 * l_{rep}} \tag{2.8}$$

In this formula is w_{max} the maximal allowed vertical deformation.

2.3.5. Relative Rotation of the Foundation

As explained in Section 2.2, the deformation of the foundations is one of the causes for the vertical deformations in the primary structure. The NEN9997 provides a limit for the differential deformation of the foundation. This limitation regards the relative rotation of the foundation [20]. The definition of the relative rotation α is shown in Figure 2.10.

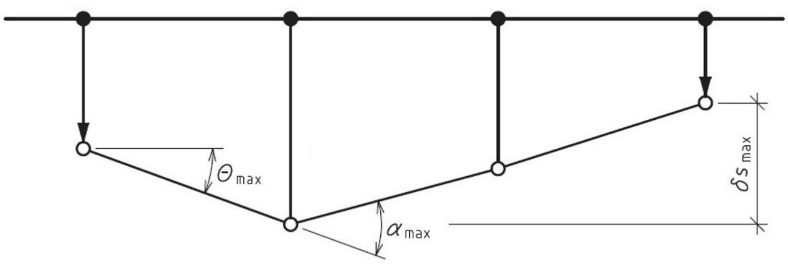


Figure 2.10: Definition of the relative rotation of the foundation [20]

The NEN9997 states that the maximum allowed relative rotation for an open skeleton construction, skeleton construction with walls or construction with load-bearing walls is not likely to be the same, but varies between 1:200 and 1:300 to prevent the limits of the SLS to exceed. The relative rotation is likely to lead to an exceedance of the+ ULS when the rotation is 1:150. These values are applicable for downward bending. Downward bending is when the center of the building settles more than the perimeter of the building, as shown in Figure 2.10. Upward bending is the opposite, then the perimeter of the building settles more than the center of the building. For the upward bending of the foundation, the limits for relative rotations are half of the values mentioned above (as mentioned in the NEN9997).

For this research the following limits are assumed:

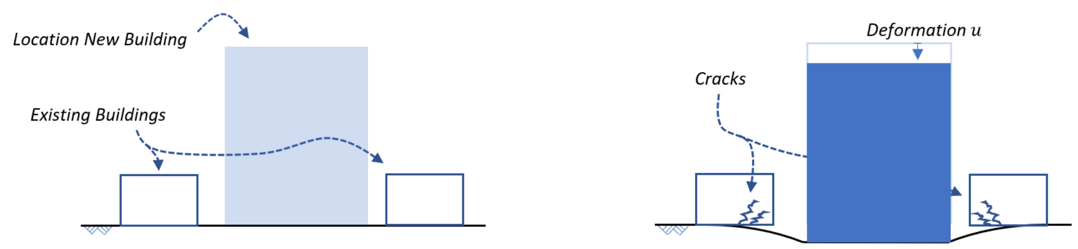
$$\alpha_{SLS} = \frac{1}{300} \tag{2.9}$$

$$\alpha_{ULS} = \frac{1}{150} \tag{2.10}$$

Further research is needed to investigate if a link can be made between the limit for relative rotation and the limit for floor rotation and facade rotation.

2.3.6. Settlement of Adjacent Buildings

In this subsection, the limit is explained for the maximal rotation allowed for adjacent buildings provided by the CUR166. The limit is for relative rotation of the soil next to the new building (under the adjacent buildings). The problem for adjacent buildings is shown in Figure 2.11 with a schematic drawing. In this figure, the initial position is shown (Figure 2.11a) and the deformed position after the completion of the building (Figure 2.11b) where the cracks in the surrounding buildings are shown as a result of the deformation u of the newly constructed building. The soil under the adjacent buildings settles, also called hogging.



(a) Initial Position

(b) Deformed Position

Figure 2.11: Schematic drawing to explain the problem for the adjacent buildings

The values imply for the maximal allowed relative rotation are shown in Table 2.1. These limits are for upward bending of the foundation, as explained previously.

Table 2.1: Limits for the relative rotation under the adjacent buildings

Type of Damage	Skeleton Construction	Masonry
Architectural Damage	1:300 (general) 1:600 (High-rise buildings) 1:1000 (Warehouse)	1:1200
Structural Damage (Cracks of 15 -25 mm)	1:150	1:600
Possibility of Collapse	1:75	1:300

2.4. Construction Sequence

In the Eurocode is mentioned that the construction sequence must be considered. The Eurocode states that when the functioning or damage of the structure and/or finishes are considered, the verification of deformation should take into account the part of the deformation which occurs after the execution of the member and/or finishes concerned [18].

For high-rise buildings on compressible soil it is essential to take the construction sequence into account. During construction the soil will start the deform and affects the structure immediately. However, the building constructed gradually and these two aspects will influence the amount of deformation which is experienced by the floors and the finishes. In Figure 2.12 a schematic drawing shows that the deformations are happening during construction sequence. For the explanation, a cross-section of a building with three levels is used with three construction stages. The deformations are the result of the self-weight of the structure itself. The red building sequence shows the deformations only due to column shortening, and the green building sequence shows only the deformation due to deformation of the foundation. In the schematic drawing is shown that the deformations start to happen during construction. During construction the columns will already shorten, and the foundation under the core starts to settle. The lower floors are affected by these deformations, but the higher floors are constructed in a later stage, which means that they are constructed horizontally as shown in the figure. The top levels of the construction stages are all horizontally, as a conclusion can be said that the construction sequence decreases the problem for the higher levels.

In the figure can also be seen that the rotation of the floors due to the column shortening is the opposite of the rotation of the floors due to the deformation of the foundation.

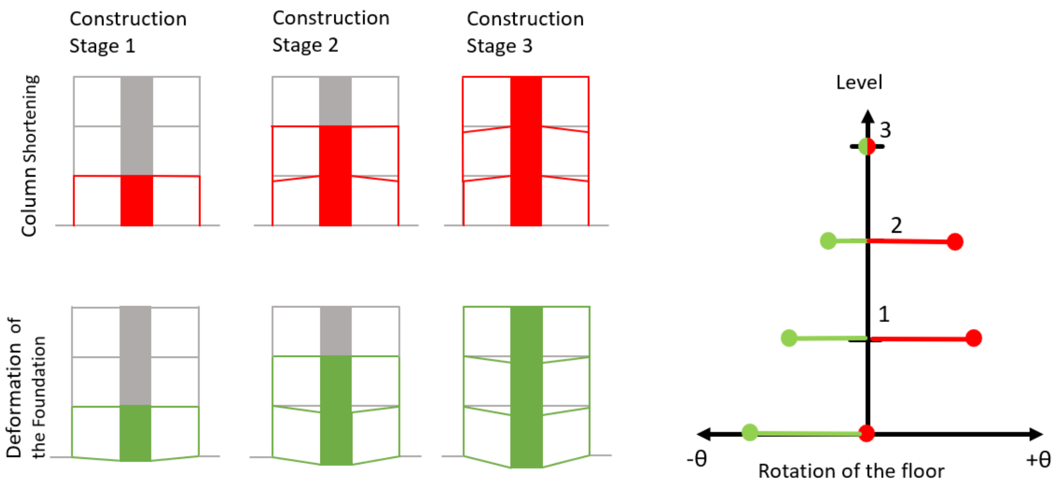


Figure 2.12: Schematic drawing of the rotation of the floors due to column shortening and displacements of the foundation

2.5. Case-study: De Rotterdam

De Rotterdam is a building located at the Wilhelminapier in Rotterdam (see Figure 1.1). The building has a height of 150 m and is constructed with a primary structure in concrete. De Rotterdam is a building that is built on top of a compressible layer.

The calculated deformations of the soil due to the weight of the building are shown in Figure 2.13.

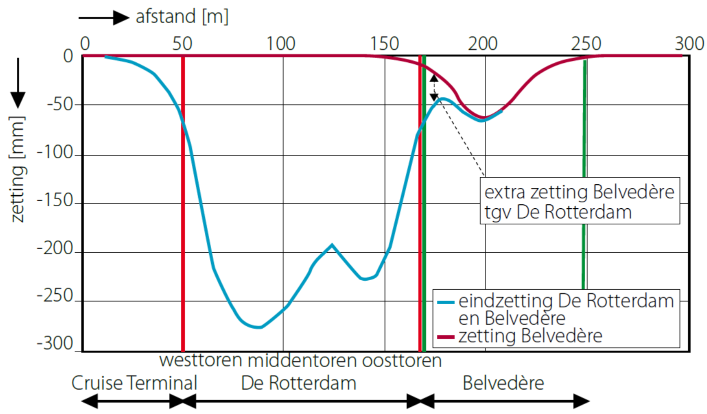


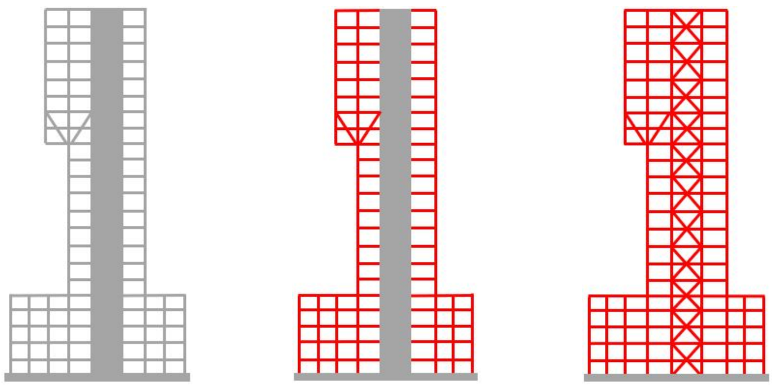
Figure 2.13: Calculated deformation of the soil for the building De Rotterdam [11]

The maximal deformation of the soil due to the weight of the building is 0.275 m, as shown in Figure 2.13.

3
Structural Design

In this chapter the geometry of the building and the dimensions of the structural elements are established. This design and those dimensions are used for the calculation of the deformation of the substructure.

For this research, the building will be constructed in three different options. The amount of settlement of the compressible soil is influenced by the weight and stiffness of the structure, in Section 4.3 this interaction is explained more in detail. For this reason, three alternatives are designed for the structural design to investigate the influence of the structural design. In Figure 3.1 the three building systems are shown, with the color gray as the structural material concrete and in red the structural material steel. These three building systems are chosen because they are commonly used.



(a) Concrete Alternative (b) Hybrid Alternative (c) Steel Alternative

Figure 3.1: Schematic drawing of the three structural design for the building

3.1. Geometry and Starting Points of the Building

For this research a building is designed to investigate the influence of the compressible soil in the structure. As explained in Subsection 2.2.3, the geometry of the building will amplify the problem. For this reason the building is designed with a cantilevering part and setbacks on the front and the back of the building. The geometry of the building is shown in Figure 3.2.

Figure 3.2: Impression of the structure of the building

The dimensions of the building are:

- Height is 144 m
- Storyheight is 3.6 m
- Griddistance is 7.2 m

The starting points for this structure are:

- Consequences Class 3
- Design lifespan of 50 years
- Fire resistance is 120 minutes

The dimensions of the building are chosen in order to comply with regular buildings in the Netherlands. The starting points of the structure are determined according to the Eurocode. The building is regarded as CC3 because for a high-rise building in the city center, the consequence of failure is high. The design working life of 50 years is set in line with the indicative design working lifespans in the Eurocode. The fire resistance of a building higher than 13 m must be 120 minutes, which is in line with the requirements of the Bouwbesluit [6].

In Figure 3.3, a side view of the structure is shown. The structure is kept simple in order to understand the flow of forces and to dimension the structural elements with simple calculations. In this figure, the definition of the low-rise and the high-rise part of the building are pointed out. The functions are shown in the figure too, the ground floor and the first level are for public space, the 20th level is a level for installations, and the rest of the levels is for offices. In Figure 3.4 the typical floor plans are shown. The floor plans show that the structure consists of a stability core and columns as the vertical load-bearing structure.

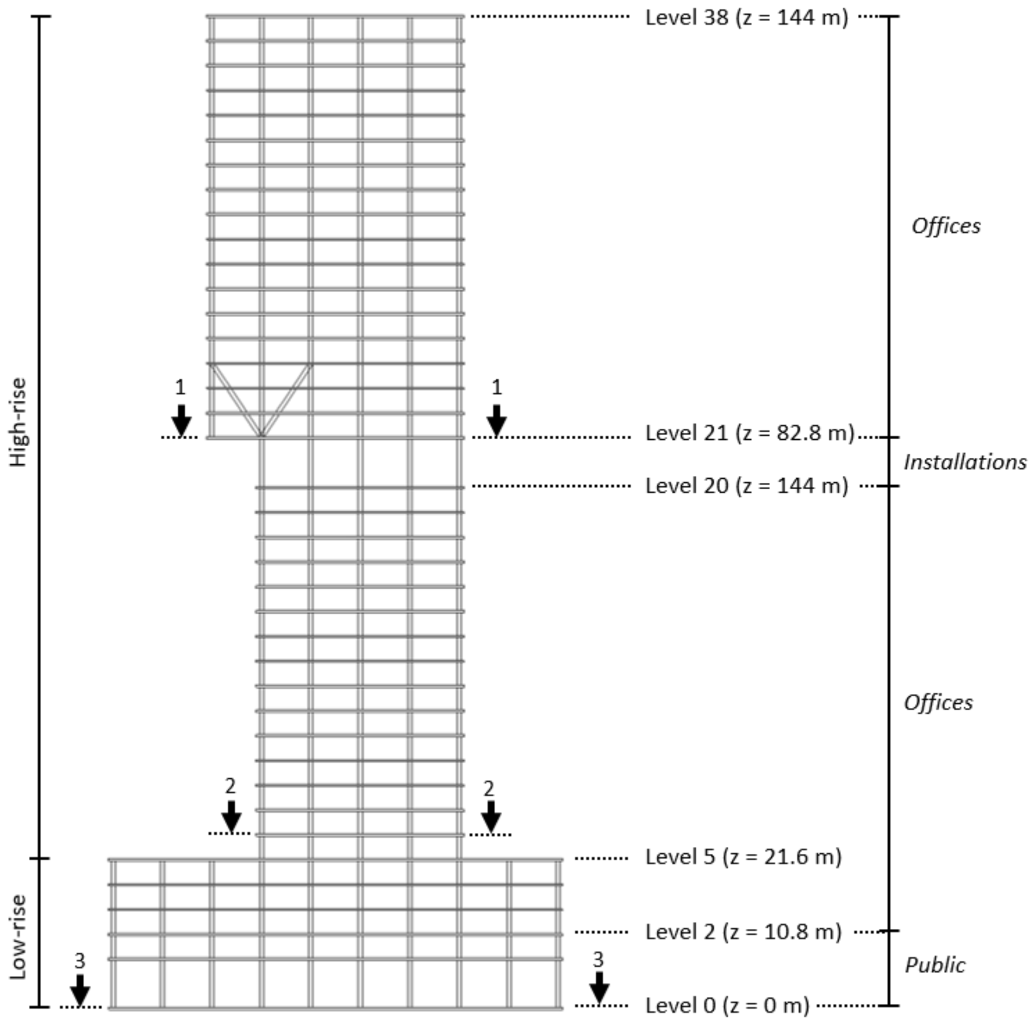
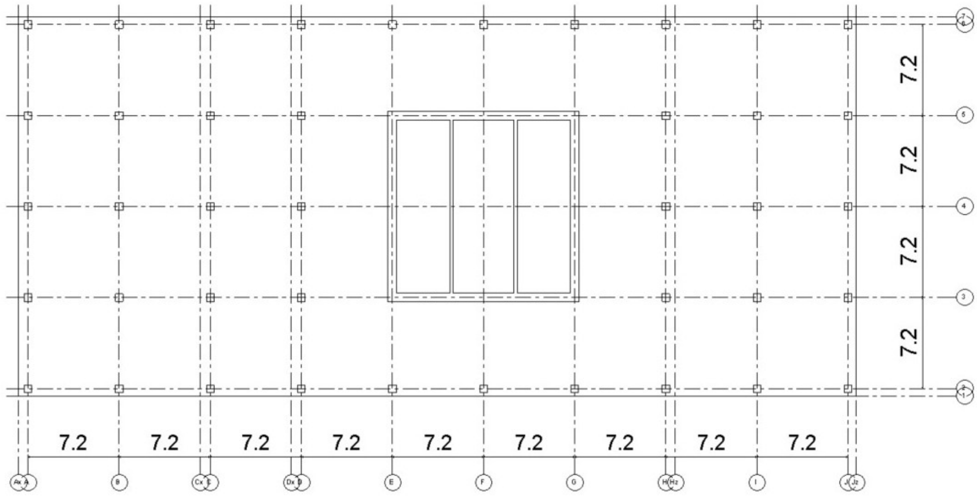
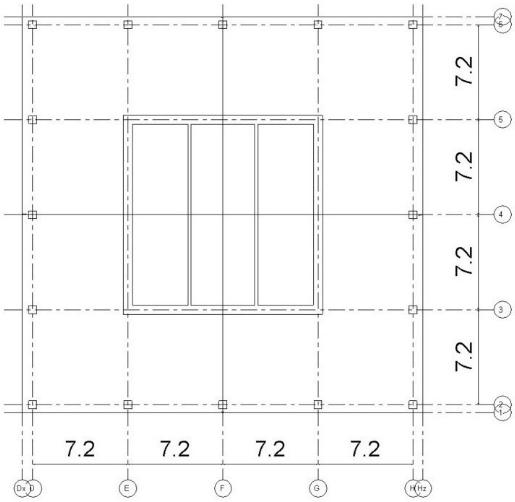


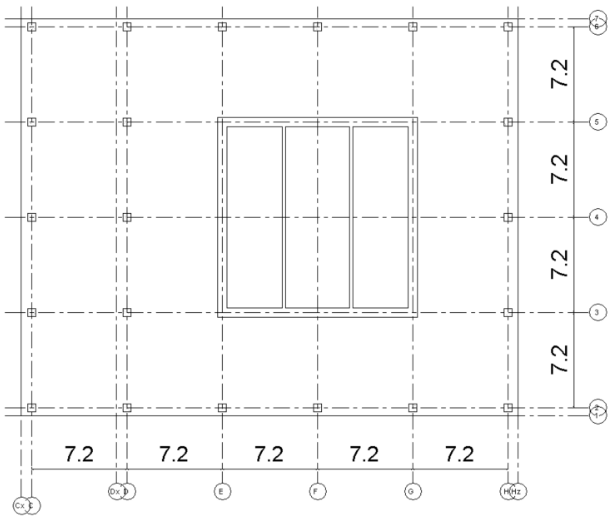
Figure 3.3: Sideview of the structure



(a) Floorplan of the low-rise part, cross section 3-3



(b) Floorplan of the high-rise part, cross section 2-2



(c) Floorplan of the high-rise part, cross section 1-1

Figure 3.4: Floorplans

3.2. Loads and Material Properties

In this section, the loads which are applied to the structure and the materials used are stated. The loads are divided into permanent and variable loads on the structure.

Floor loads

The permanent loads on the structure are assumed based on the estimation of the weight. The permanent loads on the structure used are:

Table 3.1: Permanent floor loads

Function	Load (kN/m^2)
Finishing floor	1.0
Ceiling and ducts	0.25
Partition	1.0
Facade	1.0

The variable loads on the floor per function, together with the partial factors are stated in the Eurocode. The variable loads and partial factors used in the calculations are:

Table 3.2: Variable floor loads

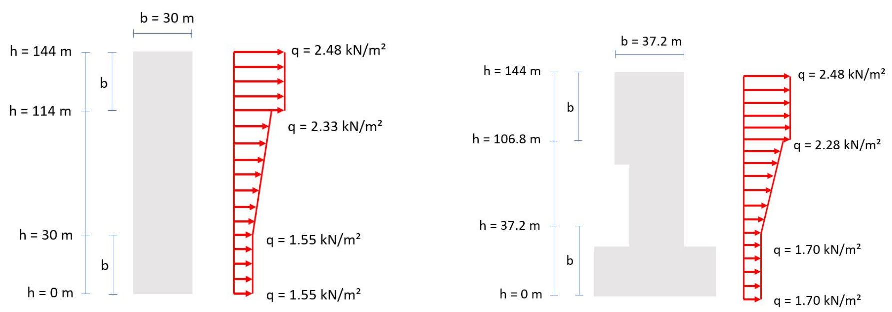
Function	Category	Ψ_0	Ψ_2	Load (kN/m^2)
Offices	B	0.5	0.3	2.5
Public	C	0.6	0.6	5.0
Installation	E	1.0	0.8	7.5
Roof	H	0.0	0.0	1.0

Wind-load

The building is also subjected to wind load, this load is calculated according to the Eurocode. In this section the final values for the wind load are stated. The calculation of the wind-pressure is explained in Appendix 3. The wind-pressure is different for the two considered wind direction, which depends on the width b of the building as shown in Figure 3.5. The wind pressure on the facade over the height of the building is as followed:

Table 3.3: Wind loads

	Height (m)	Wind load X-direction(kN/m^2)	Wind load Y-direction(kN/m^2)
q_1	144	2.48	2.48
$q_{2,max}$	144 - b	2.33	2.28
$q_{2,min}$	0 + b	1.55	1.70
q_3	0	1.55	1.70



(a) Windpressure in x-direction

(b) windpressure in y-direction

Figure 3.5: Windpressure on the building over the height

Load combinations

The load combinations used for the structural calculations are according to the Eurocode. In the load combinations represents G the sum of the self-weight and the permanent loads on the structure, Q_{floor} is the variable floor load and Q_{wind} is the variable load due to the wind. A distinction is made between the load combinations to design the structure and a load combination for the analysis of the vertical deformations.

The following load combinations are used for the dimensioning of the horizontal structural elements (as the floors):

$$ULS : 1.49 * G + 1.65 * \Psi_0 * Q_{floor} \tag{3.1}$$

$$SLS : 1.0 * G + 1.0 * Q_{floors} \tag{3.2}$$

The following load combination is used for the dimensioning of the vertical structural elements (as the columns):

$$ULS : 1.49 * G + 1.65 * \Psi_0 * Q_{floor} \tag{3.3}$$

For the analysis of the lateral deformation of the structure due to the wind load is the following load combination used:

$$SLS : 1.0 * G + 1.0 * Q_{wind} \tag{3.4}$$

For the calculation of the vertical deformation of the soil and the analysis of the corresponding deformations in the structure is the following load-case used:

$$SLS : 1.0 * G + 1.0 * \Psi_2 * Q_{floors} \tag{3.5}$$

The quasi-permanent combination is used to calculate the long-term effects of the deformations.

Structural materials

The superstructure of the building is constructed with the following structural materials:

Concrete C35/45		
Compressive strength	f_{ck} =	35 MPa
E-modulus	E_{cm} =	34000 MPa
Density	$\rho_{concrete}$=	25.0 kN/m^2
Concrete C90/105		
Compressive strength	f_{ck} =	90 MPa
E-modulus	E_{cm} =	44000 MPa
Density	$\rho_{concrete}$=	25.0 kN/m^2
Reinforcement Steel B500B		
Yield strength	f_{yk} =	435 MPa
E-modulus	E_{steel} =	210000 MPa
Density	ρ_{steel}=	78.5 kN/m^2
Steel S355		
Yield strength	f_{yk} =	355 MPa
E-modulus	E_{steel} =	210000 MPa
Density	ρ_{steel}=	78.5 kN/m^2

In the structural calculation the displacement and deformations of structural elements are regularly normative. The crack of the structural material concrete will influence the stiffness of the structural elements. To take into account the crack, the Young's modulus of the concrete is multiplied by a factor depending on the type of structural element. The E-modulus of concrete for the structural elements as the core, floors, and columns are:

$E_{cm,core}$ = 0.70 * 34000 MPa = 23800 MPa

$E_{cm,floors}$ = 0.30 * 34000 MPa = 10200 MPa

$E_{cm,columns}$ = 1.0 * 44000 MPa = 44000 MPa

3.3. Structural System

The structural systems of the three alternatives are identical regarding the load-bearing structure. The three structures consist of a stability core and a skeleton structure around it, as shown in Figure 3.6. The stability core is used for the vertical transportation of the people and as the structural element to resist the horizontal forces. The stability core is placed in the center to exclude significant torsional forces. The primary function of the stability core is to resist the horizontal wind loads on the facade of the building.

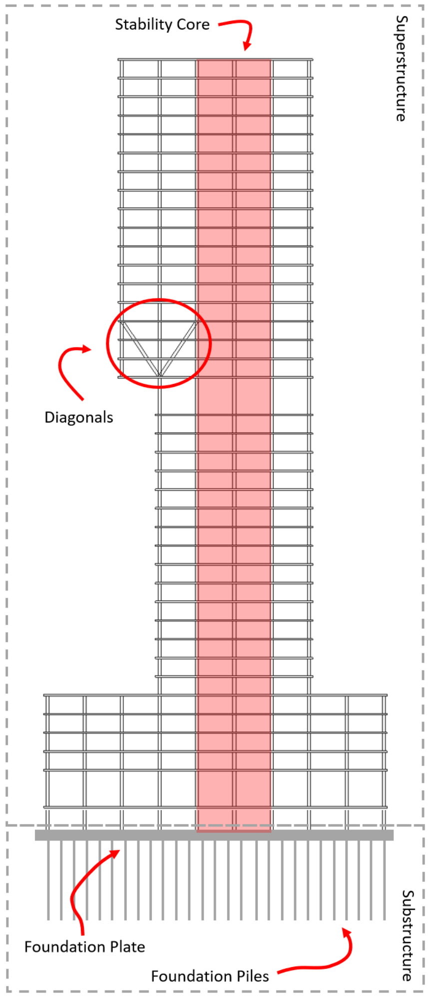


Figure 3.6: Schematic drawing of the structural system

The stability core is also used to support the floors which are attached to the stability core. The floor loads are transferred to the foundation by the stability core and the columns. The structural system of the floor elements is different for the three alternatives, as will be explained in Subsection 3.3.2.

The diagonals support the cantilevering part of the building. The diagonals are used to redistribute the forces of the cantilevering columns to the supporting columns. In this way, the moment of the cantilevering part does not have to be supported by the horizontal structural elements.

The connections between the vertical elements and the horizontal elements are rigidly connected in the three alternatives. The facade elements are assumed to be a light-weighted facade system which are supported by the floor elements of the structure.

The forces of the construction are transferred to the soil by the substructure of the building. The substructure consists of a foundation slab and foundation piles.

In the following subsections the dimensions of the structural elements are established.

3.3.1. Stability Core

For the stability core two options are designed. The first one is the concrete core for the concrete and hybrid alternatives, and the second one is the steel truss for the steel alternative. The two options are subjected by the same wind load because of the dimensions.

The dimensions of the core are calculated based on the maximal lateral deflection at the top of the core. The lateral deflection is not allowed to exceed the limit for lateral deflection as stated in Subsection 2.3.1. Figure 3.7 shows that the total deflection of the top of the core consists of two parts, the deflection of the stability core and the rotation of the foundation.

$$u_{hor,tot} = u_{def} + u_{rot} <= \frac{H}{500} \tag{3.6}$$

In Formula 3.6 is $u_{hor,tot}$ the maximal horizontal deflection at the top of the core, u_{def} is the horizontal deflection at the top due to deflection of the stability core, u_{rot} is the horizontal deflection at the top due to rotation of the foundation and H is the height of the building.

For this research, the allowed deflection is equally assumed over the two parts. The rotation stiffness of the foundation for short-term loadings is out of the scope for this research, so the lateral deflection due to the rotation is furthermore assumed to be 0.5 * u_{max}.

$$u_{def} <= 0.5 * \frac{144}{500} = 0.144m \tag{3.7}$$

$$u_{rot} <= 0.5 * \frac{144}{500} = 0.144m \tag{3.8}$$

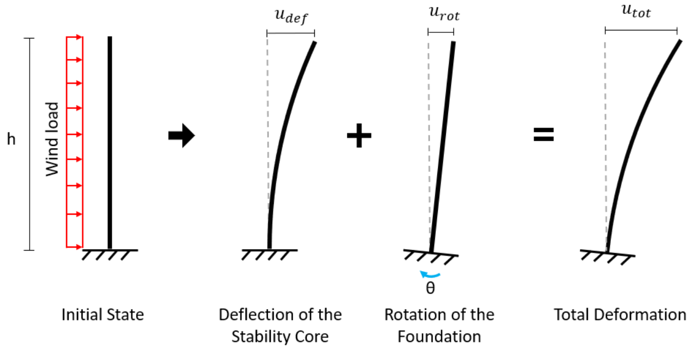


Figure 3.7: Deformation of the stability core under wind load

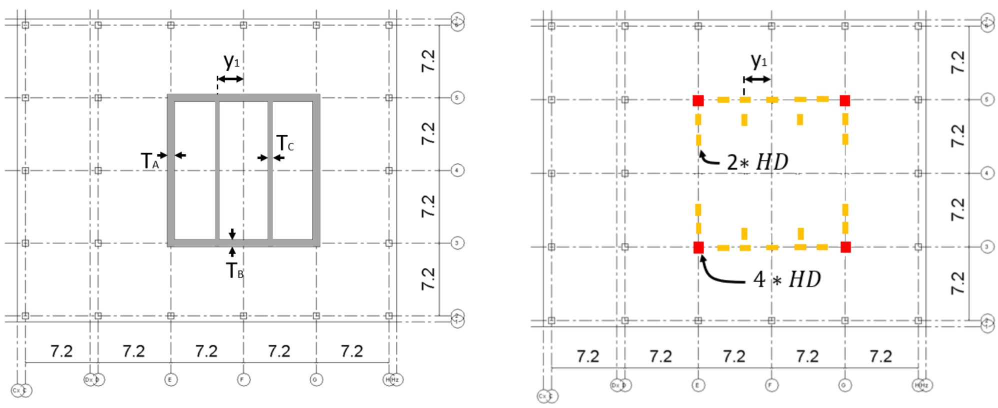
With the calculated limit for the horizontal deflection due to deflection of the core itself, the required moment of inertia for the stability core is calculated with the following beam equation:

$$u_{def} = \frac{1}{8} * \frac{q * h^4}{E * I_{req}} \tag{3.9}$$

Dimensions of the core

The dimension of the core is determined to meet the required moment of inertia.

Figure 3.8 shows the cross section to show the dimensions of the stability core for the two alternatives. The dimensions of the concrete core are shown in Table 3.4. A detailed calculation can be found in Appendix A.



(a) The concrete alternative

(b) The steel alternative

Figure 3.8: Horizontal cross-section of the stability cores

Table 3.4: Dimensions of the concrete core

	Dimensions (m)
T_A	0.75
T_B	0.80
T_C	0.50
y_1	3.0

Check of the stability core

The check of the stability core consists of two parts: the deflection due to the wind load and the deflection caused by the second-order effect. The definitions are shown in Figure 3.9.

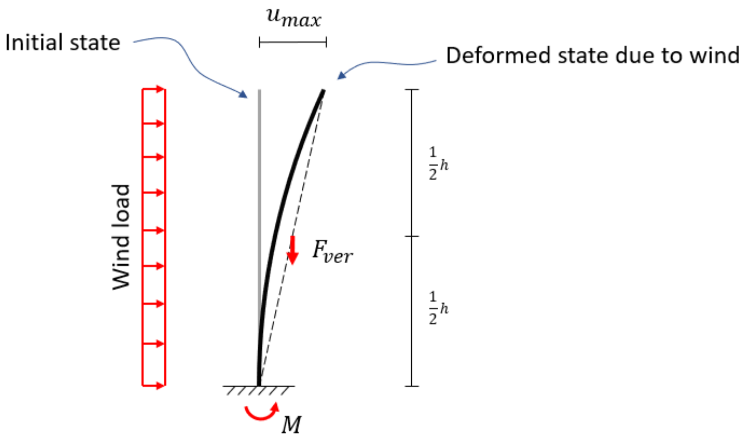


Figure 3.9: Deflection of the stability core

First, the deflection due to the wind load is calculated with the program SCIA engineer. Due to this deflection, the vertical force of the building will shift, which induce an extra bending moment at the support. This bending moment can be estimated by multiplying the vertical force by half of u_{def}. Due to the second-order deflection the building will deflect some more, which induces a third-order bending and therefore, an extra deflection etc. This accumulation can be estimated with a magnification factor, as shown in Formula 3.10:

$$N = \frac{n}{n-1} \tag{3.10}$$

In Formula 3.10 is n the ratio between the bending moment due to the wind and the bending moment of the second order calculation.

$$n = \frac{M_{wind}}{M_{secondorder}} \tag{3.11}$$

The total deflection due to the wind load and the second order deflection taken into account is calculated with Formula 3.12.

$$u_{tot} = N * u_{wind} \tag{3.12}$$

Results

The results are shown in the Table 3.5:

Table 3.5: Results of the total deflection of the stability core

Building	M_{wind} [kNm]	u_{def} [mm]	u_{rot} [mm]	u_{tot} [mm]	F_G [kN]	M_{second} [kNm]	n	M	u_{total} [mm]
Concrete	$8.81 * 10^5$	94.0	144	238	$6.83 * 10^5$	$8.13 * 10^4$	10.85	1.102	103.55
Hybdrid	$8.81 * 10^5$	95.4	144	239.4	$5.58 * 10^5$	$6.68 * 10^4$	13.20	1.082	103.22
Steel	$8.80 * 10^5$	85.6	144	229.6	$4.09 * 10^5$	$4.70 * 10^4$	18.73	1.056	90.43

3.3.2. Floor Systems

The three alternatives are designed with different floor systems. The systems are designed to resist loads of the office floors as a simplification because this function has occupied the most levels. The floors for the public and installation levels are modelled the same. A detail calculation of the floor systems including the deformation check can be found in Appendix A.

Concrete Alternative

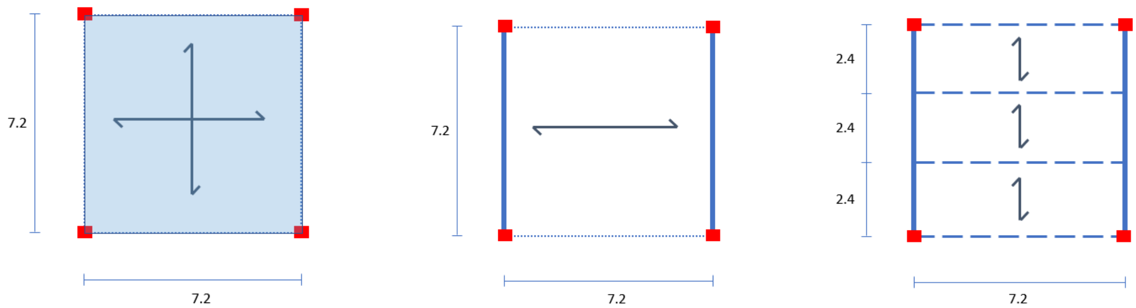
For the concrete alternative, the floor system is constructed of a concrete flat slab floor with a thickness of 250 mm is sufficient for all the functions in the building. The floor system is shown in Figure 3.10a.

Hybrid Alternative

For the hybrid alternative, the floor system is constructed with a Comflor 210 with a height of 300 mm. The primary beam which supports the floor is a HEB450 profile. This floorsystem is shown in Figure 3.10b.

Steel alternative

For the steel alternative, the floor is designed to be as light as possible, therefore the floor is constructed with a Comflor 46 with a height of 120 mm. The secondary beam which will support the Comflor is an IPE400 profile. The primary beam which will support the secondary beams is an IPE550 profile. The floorsystem is shown in Figure 3.10c.



(a) The concrete alternative (b) The hybrid alternative (c) The steel alternative

Figure 3.10: Floor systems for the different alternatives

3.3.3. Dimensions of the Columns

The dimensions of the columns are determined according to a weight calculation made for the building. The properties of the structural materials, as stated in Section 3.2, are used to calculate the required dimensions. To simplify the design, the columns are divided into groups depending on the strength needed according to load combination shown in Formula 3.1. The column groups are shown in Figure 3.11.

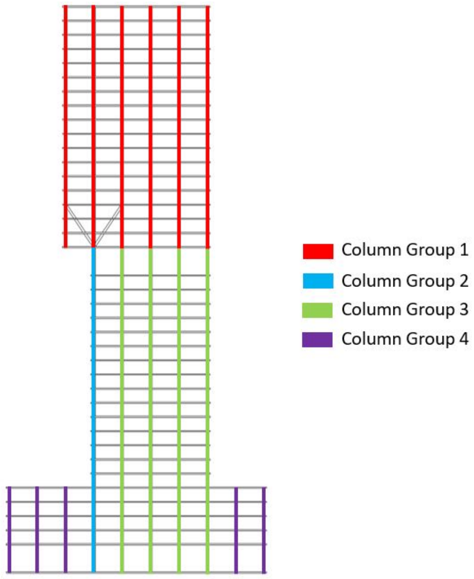


Figure 3.11: The column groups used to calculate the dimensions of the columns

The dimensions of the columns are designed based on the normal force in the columns and the buckling load of the columns. The area required to resist the normal force is calculated with Formula 3.13, the buckling load for the concrete columns is calculated with Formula 3.14 and for the steel columns with Formula 3.15.

$$A_{req} = \frac{N_{Ed}}{f_y} \tag{3.13}$$

For the concrete columns the buckling load is calculated with the formula of Euler.

$$N_{b,Rd} = N_{cr} = \frac{\pi^2 * EI}{(K * L)^2} \tag{3.14}$$

In Formula 3.14 is L the storey height and k is the buckling constant. For this research $k = 0.5$ is assumed, as the columns are continuous and rigid connected to the floor.

For the steel columns the formula of Euler can be too positive, therefore the buckling load of the steel columns is calculated according to the Eurocode. The following formulas are used:

$$N_{b,Rd} = \frac{\chi * A * f_y}{\gamma_{M1}} \tag{3.15}$$

In Formula 3.15 is γ_{M1} the partial factor and is equal to 1.

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \tag{3.16}$$

$$\Phi = 0.5 * (1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2) \tag{3.17}$$

In Formula 3.17 is α the imperfection factor for the buckling curve depending on the properties of the used steel profile.

$$\bar{\lambda} = \sqrt{\frac{A * f_y}{N_{cr}}} \tag{3.18}$$

The dimensions used in the structural designs of the columns are shown in Table 3.6:

Table 3.6: Dimensions of the columns

Column Group	Concrete Alternative	Hybrid Alternative	Steel Alternative
Column Group 1	$450 * 450$ mm^2	HD 400 x 216	HD 400 x 216
Column Group 2	$850 * 850$ mm^2	HD 400 x 818	HD 400 x 744
Column Group 3	$600 * 600$ mm^2	HD 400 x 347	HD 400 x 347
Column Group 4	$400 * 400$ mm^2	HEM 300	HEM 300

The dimensions of the column groups 1, 2, and 3 are based on the calculations explained above. The dimensions of the columns of column group 4 are also based on the calculations, but the results of the calculations allow smaller dimensions than the chosen dimensions. In practice these dimensions are used as a minimum for a column supporting this number of levels.

3.3.4. Dimension of the Diagonals

In the structural design the diagonals are constructed to support the cantilevering part of the building. The diagonals are shown in Figure 3.6. The normative loads on the diagonals are the result of the load combination of Formula 3.1. The diagonals have to resist the axial force and are susceptible to buckling. For buckling of the diagonals, it is assumed that the diagonals are laterally supported by the floors and that the columns are rigid connected ($k = 0.5$). The dimensions are calculated in the same way as the dimensions of the columns are established, as explained in the previous subsection.

The top of the diagonals is connected to a tension beam in order to create equilibrium, as shown in Figure 3.12, without overstressing the floor elements. The diagonals are constructed over the height of 3 levels to resist the compression force.

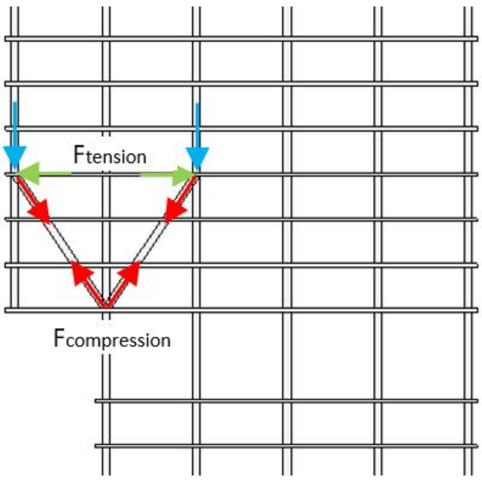


Figure 3.12: Diagonals and tension beam which support the cantilevering part of the building

The calculated dimensions for the diagonals and the tension beam are shown in table 3.7:

Table 3.7: Dimensions of the diagonals and the tension beams

	Concrete Alternative	Hybrid Alternative	Steel Alternative
Diagonal	$550 * 550\ mm^2$	HD 400 x 509	HD 400 x 463
Tension Beam	HEM 300	HEM 300	HEM 300

3.3.5. Foundation Slab

The stiffness of the foundation slabe is also considered. The foundation slab will increase the stiffness of the building and is therefore of influence on the settlement. In the initial design a foundation slab is constructed with a thickness of 1500 mm.

3.4. Forces on the Foundation

The reaction forces on the foundation are associated to the deformation in the compressible soil. For this research, the building is supported by 5 * 10 supports in the SCIA model, as shown in Figure 3.13. The locations of the supports are directly under the columns and the stability core. It is essential to understand that per support, the number of piles is still to be determined. So, one support in the SCIA model represents a group of piles under the vertical structural element.

In order to determine the number of piles needed per support, the supports are divided into groups based on the range of reaction forces of the supports. The groups are shown in Figure 3.13.

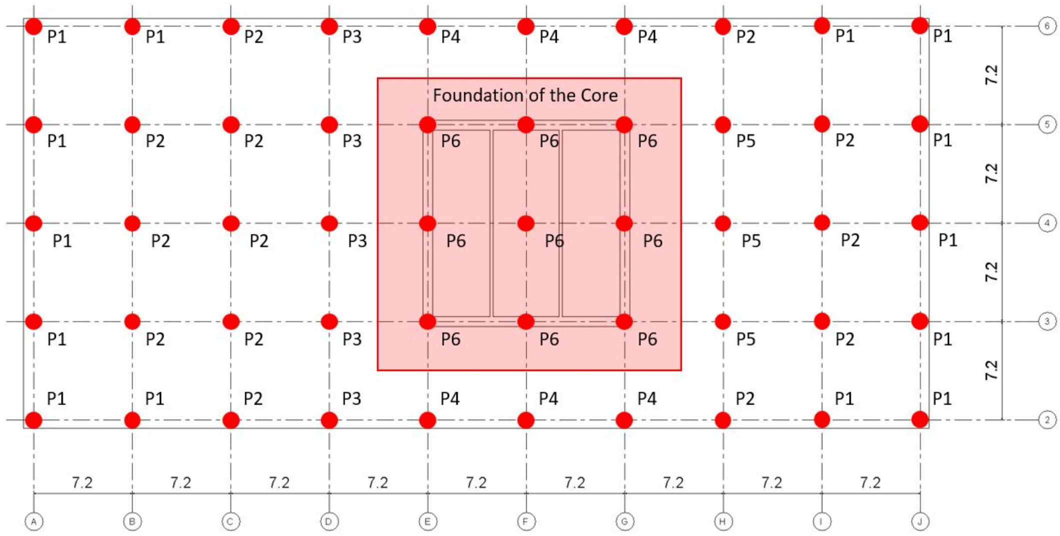


Figure 3.13: Locations of the supports with mentioning the groups used for the calculation

3.4.1. Total Weight of the Structures

The alternatives for the structures are designed to investigate the influence of the weight of the buildings. The total weight of the structures are shown in Table 3.8

Table 3.8: Total weight of the different structures in ULS

Alternative	Total Weight [kN]	Ratio
Concrete	$1122.8 * 10^3$	1
Hybrid	$929.6 * 10^3$	0.83
Steel	$697.9 * 10^3$	0.62

The results of the total weight show that the concrete alternative is the heaviest and the steel option is the lightest alternative.

3.4.2. Bearing Capacity of the Foundation Piles

In order to design the structure, the load-bearing capacity of the foundation piles needs to be known. The bearing capacity of the foundation piles is determined based on the CTP of the locations. The method used for this calculation is the Koppejan method. The strength is determined with a hand calculation and for verification also calculated with the program D-Foundations.

Load bearing calculation

The characteristic load-bearing capacity of the pile consists of two parts as shown in Formula 3.19

$$R_{c,k} = R_{b,k} + R_{s,k} = \frac{R_{b,cal} + R_{s,cal}}{\xi} \tag{3.19}$$

In Formula 3.19 is $R_{b,k}$ the characteristic load-bearing capacity of the pile tip, $R_{s,k}$ is the characteristic load-bearing capacity of the shaft, ξ is 1.39 as this is the correlation-factor depending on the number of CPT done, see Table A.10a in NEN 9997 [20]. $R_{b,cal}$ is the maximal load bearing capacity of the pile tip in this formula and $R_{s,cal}$ is the maximal load bearing capacity of the shaft friction.

The load bearing capacity can be decreased when negative shaft friction occurs, for this research the negative shaft friction is neglected. The positive shaft friction, which will increase the load-bearing capacity of the foundation pile, is only considered over the length of the pile in the load-bearing sand layer. Any positive friction due to other layers is neglected.

The design value of the load-bearing capacity of the foundation pile can be calculated with Formula 3.20.

$$R_{c,d} = \frac{R_{c,k}}{\gamma_t} \tag{3.20}$$

In Formula 3.20 is $R_{c,d}$ the design value and γ_t is 1.2 according to the partial factors in table A.6 in NEN 9997.

The maximal load bearing capacity of the pile tip is calculated with Formula 3.21.

$$R_{b,cal} = A_b * q_{b,max} \tag{3.21}$$

In Formula 3.21 is A_b the area of the pile tip and q_b, max is the pile tip resistance.

The maximal load bearing capacity of the shaft friction is calculated with Formula 3.22.

$$R_{s,cal} = O_s * \int q_{s,z} dz \tag{3.22}$$

In Formula 3.22 is $q_{s,z}$ the average shaft friction over the depth z.

The maximal pile tip resistance is calculated according to the CPT with the Koppejan method. The resistance of the pile tip is calculated with Formula 3.23.

$$q_{b,max} = \frac{1}{2} * \alpha_p * \beta * s * (\frac{q_{c;I;gem} + q_{c;II;gem}}{2} + q_{c;III;gem}) \tag{3.23}$$

In Formula 3.23 is $p_{b,max}$ the maximal pile tip resistance with a maximum of 15 MPa , α_p is a reduction-factor depending on the method of inserting the foundation pile, β is a reduction-factor for piles with a larger pile tip, s is a reduction-factor depending on the cross section of the pile, $q_{c;I;gem}$ is the minimal average value of the cone resistance over a depth of 0.7 till 4.0 times the equivalent pile diameter (trajectory 1), $q_{c;II;gem}$ is the average value of the cone resistance over a depth from the bottom of trajectory 1 to the pile tip (trajectory 2), and $q_{c;III;gem}$ is the average cone resistance over a depth from the pile tip depth to 8 times the equivalent pile diameter above the pile tip (trajectory 3).

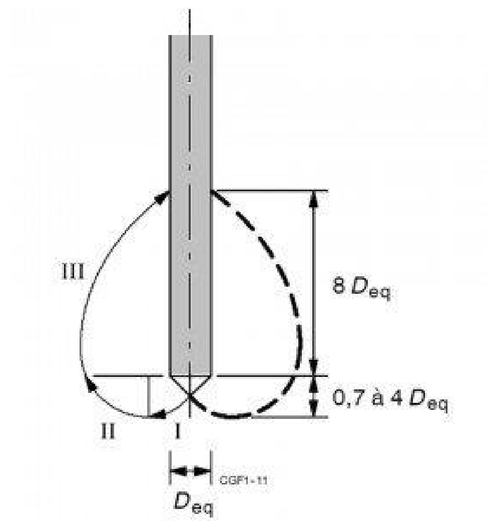


Figure 3.14: Trajectories of the method of Koppejan [1]

Which pile type to use

For this research, three different types of foundation piles were considered. The types of foundation piles were prefab concrete foundation piles, vibropiles type HBF, and concrete screw piles. The load-bearing capacity of the piles is related to the type of foundation pile and how the foundation pile is inserted. The values for these properties are shown in Table 3.9. Calculation of the load-bearing capacity of the foundation piles shows that the vibropile has the highest load-bearing capacity. For this reason, the vibropile type HBF is chosen to be used in the structural design for the buildings in this research. A detailed calculation can be found in Appendix A.

Table 3.9: Properties of the three types of foundation piles [2] [3] [4]

	Prefab Pile	Vibropile type HBF	Schroefpaal
α_p	0.70	0.70	0.56 (traject III reduced)
β	1.0	1.0	1.0
α_s	0.010	0.014	0.006
s	1.0	1.0	1.0

For this research the spring stiffnesses are needed in order to calculate the deformation of the foundation. The spring stiffnesses for the vibropile are determined for the three locations with the program D-Foundations. The vibropiles are modelled in the program D-Foundation with the CPT of the three locations. To verify if the calculation was done as expected, the results of the load bearing capacity are compared to the results of the hand calculation. The results are shown in table 3.10.

Table 3.10: Comparison of the results of the load bearing capacity of the hand calculation and the program D-Foundations

		Load Bearing Capacity (kN)	
Location	Depth Pile Tip (m)	Hand calculation	Program DFoundation
Amsterdam	-23	2860	3002
Rotterdam	-25.5	2200	2196
Utrecht	-15	3239	2948

The differences between the results of the hand calculation and the results of the program D-Foundations are related to the accuracy of the estimation of the cone resistance values $q_{c;I;gem}$, $q_{c,II,gem}$, $q_{c,III,gem}$, and $q_{s,z}$ for the hand calculation.

3.4.3. Number of Piles

The number of foundation piles per support is calculated by dividing the support reaction by the load-bearing capacity of the foundation piles. The SSI affects the support reactions as explained in Subsection 4.3.2. The support reactions need to be determined by an iterative process. However, for this research, a simplification is made to exclude this iterative calculation process for calculating the number of foundation piles.

3.5. Construction Sequence

In Section 2.4 is the influence of the construction sequence explained and concluded that the construction sequence needs to be taken into account for measuring the deformations in the building. Further in the research, where the construction sequence is considered to measure the deformations, the focus is already narrowed down to the concrete alternative of the structures. Therefore, in this section, the construction sequence of the concrete alternative is explained.

The construction sequence is estimated to be able to investigate the deformations of the finishes. The assumed construction time per level is stated in table 3.11.

Table 3.11: Construction time per level of the building for the concrete alternative

Part of the building	Construction time [days/level]
Foundation	3 months
Levels of the low-rise part	14
Levels of the high-rise part	8
Cantilever (4 levels)	3 months

Figure 3.15 shows the buildings sequence of a typical levels. The graph shows the height of the building with on the vertical axis the level number and on the horizontal axis the construction time in days. So, with increasing days of construction time, the height of the constructed primary structure, facade, and finishings will increase. In the figure, the sequence of the primary structure is shown per level, which is followed by the sequence of attaching the facade elements and the finishes inside. The figure shows that the construction sequence is happening per level and that it, for example, takes eight days takes to construct one level of the primary structure. The total days between constructing the primary structure and placing the facade elements is depending on the number of levels in between. It is assumed that the facade is placed six levels below the constructing of the primary structure, and for the finishes are three levels assumed after placing the facade elements.

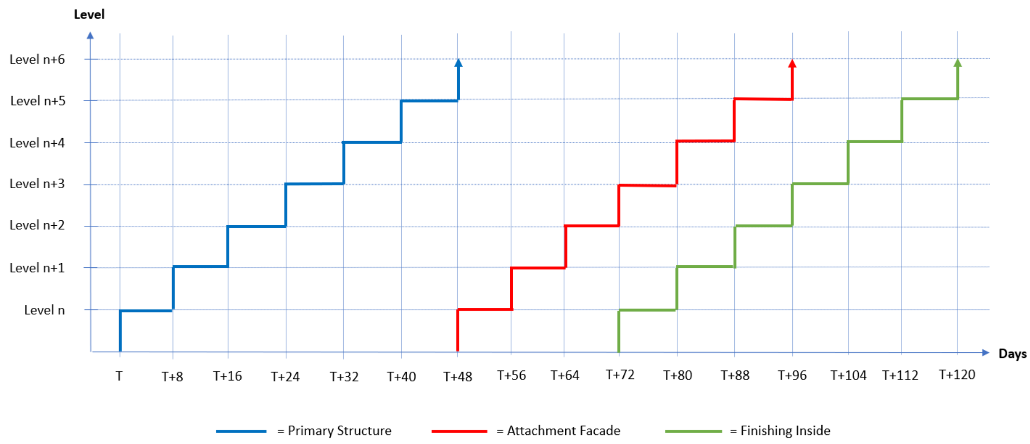


Figure 3.15: Construction sequence for a typical floor in the high-rise part of the building

4

Deformations of the Substructure

In order to take into account different soil structures three locations in the Netherlands are considered, as shown in Figure 4.1. This are locations where high-rise buildings are built or expected to be built. The locations considered are in line with the high-rise visions of the municipality of the cities [9] [13] [23]. Between the soil structures of the locations are significant differences, as will be explained in Section 4.1. In order to compare the behavior of the soil structures, the soil is loaded by the structures designed in the previous chapter.

1. Amsterdam, Overhoeks (IJ-oever)
2. Rotterdam, De Wilhelminapier
3. Utrecht, Centraal Station/Jaarbeursplein

Figure 4.1: Overview of the locations in the Netherlands

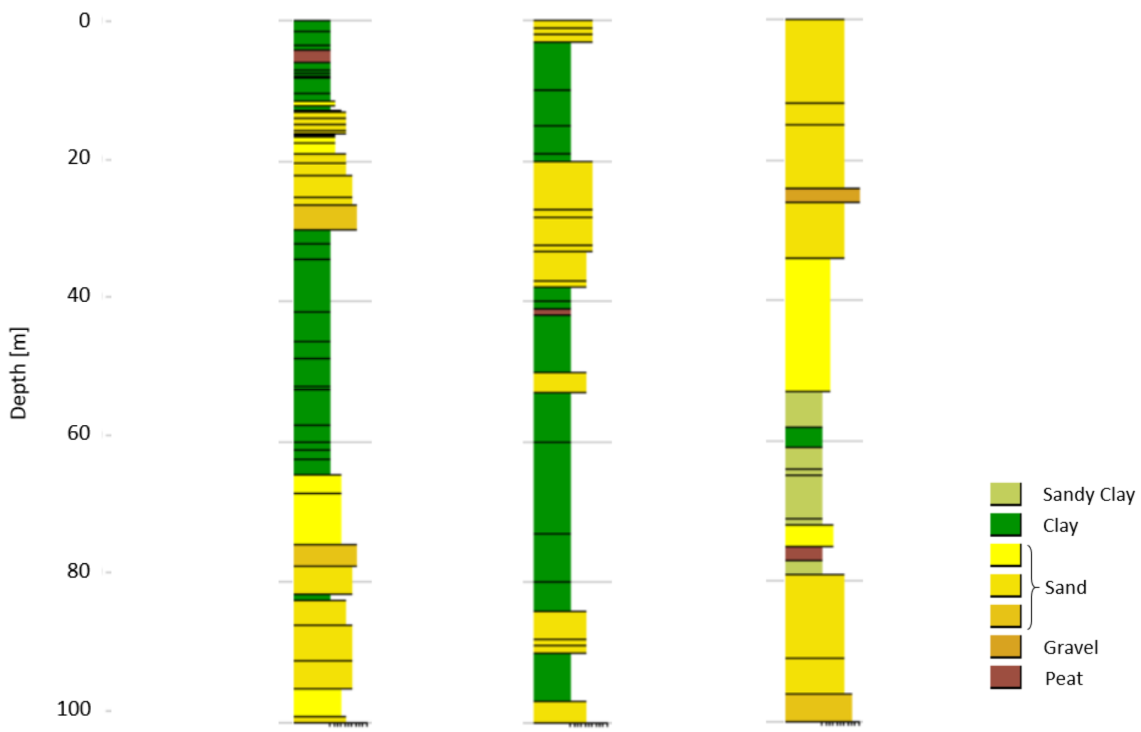
Retrieved information

The soil properties are specific per location and can vary significantly over an area of the building. In order to investigate the problems regarding the settlement of the soil, simplifications are made in this research for the properties and structures of the soil. The soil structure used in this research are based on Cone Penetration Tests (CPT) and Drilling Sample Profiles (DSP) retrieved from the DINOloket [25]. The locations of the used DSP and CPT for this research deviate from the initially wanted locations, as a consequence of the available information and because a certain depth of the profiles is needed. The second significant simplification made, is that the profiles are considered representative over the total area of the building. In reality the soil can vary over the total building plot, but for this research this is acceptable because a general impression of the deformations between the locations is of interest.

4.1. Soil Structure of the Locations

The soil structure of the three locations is different per location. The soil structure is the result of the geological sequence during the past. In the past, the Netherlands was flooded on a regular interval, because a large part of the Netherlands is below sea level. As a result of the flooding of the main rivers and the influence of the North Sea, the soil structure is constructed of different sediments [24]. Today the main rivers and the North Sea are controlled by the people, which prevents flooding of the dense populated area. Additional to the floodings, the Ice Ages did also affect the soil structure of the Netherlands. During the Ice Ages, a large part of the Netherlands was covered with a thick layer of ice. Therefore the soil is compressed by larger forces than the loads now applied on the soil, which is the result of the Over Consolidation Ratio (OCR) of the lower layers in Amsterdam and Rotterdam.

The soil structures used for this research are shown in Figure 4.2. The considered domain of the soil used for this research is set to 100 m. The DSP of the three locations shown in Figure 4.2 shows that at a depth of 100 m a sand layer is present. Further down as 100 m there will also be sand layers and no significant compressible layer. At a depth of 100 m, it is assumed that the sand layer will act very stiff and therefore be the rigid boundary conditions of the domain of 100 m for the considered soil for this research.



(a) Amsterdam (b) Rotterdam (c) Utrecht

Figure 4.2: Soil structure per location

In the Appendix B the same DSP are shown but with the total available depth. The depth of the DSP of Amsterdam is 102 m in total. Accordingly, it is assumed that the soil underneath 100 m consists of stiff sand layers. For the locations of Rotterdam and Utrecht can be observed that deeper than 100 m is sand.

Figure 4.3 shows the soil structure of the city of Amsterdam. This is a cross-section from the financial district of the 'Zuidas' to the station 'Buikslotermeerplein' in the north of Amsterdam. The dark brown color represents the Eemclay layer, which varies in thickness over the cross-section. It is essential to do extensive research in the soil structure before constructing a high-rise building by CPT and DSP. In the cities of Rotterdam and Utrecht the variation of the soil structure is also present.

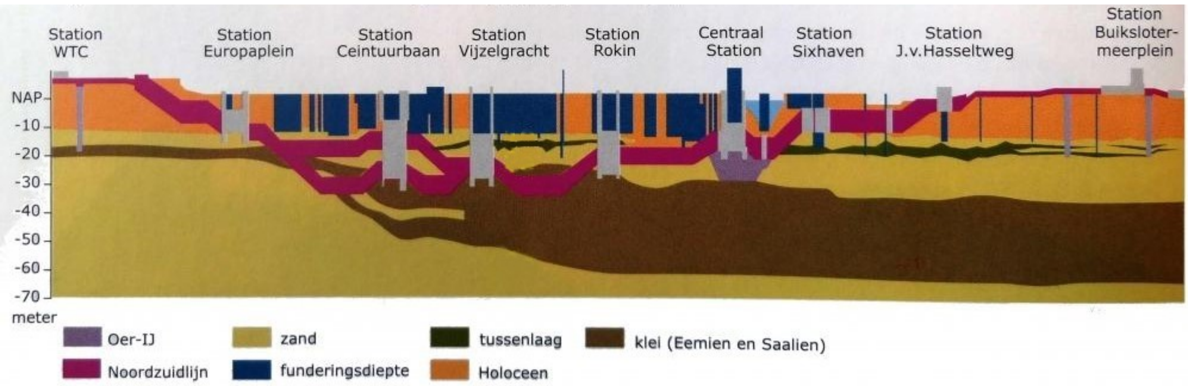


Figure 4.3: Cross section of the soil structure of Amsterdam [7]

In the following paragraphs the soil structures of the three locations are explained more in detail.

Amsterdam

The Eemclay layer characterizes the soil underneath the city of Amsterdam. Figure 4.2a shows the soil structure of Amsterdam with the Eemclay situated at a depth of -30 m to -64 m. This clay layer is an over-consolidated. Above and under the Eemclay sand layers are situated. The foundation of the building in this research is founded on the first sand layer, which has a depth from 13 m to 30 m. The depth of the soil layers are mentioned in Table 4.1.

Table 4.1: Soil structure of Amsterdam

Amsterdam				
Layer	Depth [m]			soil
I	0	-	4	Clay
II	4	-	6	Peat
III	6	-	13	Clay
IV	13	-	30	Sand
V	30	-	64	Eem-Clay
VI	64	-	81	Sand
VII	81	-	82	Clay
IX	82	-	100	Sand

Rotterdam

The clay layer of Kedichem characterizes the soil of the city of Rotterdam. The layer of Kedichem is situated under the first sand layer and begins at a depth of -38 m. In Figure 4.2b the soil structure of Rotterdam is shown. In this figure can be seen that the clay layer is much thicker than the Eemclay layer in Amsterdam. In the layer of Kedichem two small sand layers are situated, which are not noteworthy to act as a load-bearing sand layer. The load-bearing layer is the thick sand layer at a depth of 20 m to 28 m. The depth of the soil layers are mentioned in Table 4.2.

Table 4.2: Soil structure of Rotterdam

Rotterdam				
Layer	Depth [m]			soil
I	0	-	3	Sand
II	3	-	20	Clay
III	20	-	38	Sand
IV	38	-	41	Kedichem-Clay
V	41	-	42	Peat
VI	42	-	50	Kedichem-Clay
VII	50	-	53	Sand
VIII	53	-	84	Kedichem-Clay
IX	84	-	90	Sand
IX	90	-	97	Clay
IX	97	-	100	Sand

Utrecht

The soil structure at the city of Utrecht is characterized by mostly sand layers, as shown in Figure 4.2c. Underneath this thick sand layer are small layers of clay, loam, and peat. These layers are situated deeper than the Eemclay layer and the layer of Kedichem in Rotterdam. Therefore it is expected that the soil of Utrecht will react much stiffer than the other two locations. The depth of the soil layers are mentioned in Table 4.3

Table 4.3: Soil structure of Utrecht

Utrecht				
Layer	Depth[m]			soil
I	0	-	53	Sand
II	53	-	58	Sandy Clay
III	58	-	61	Clay
IV	61	-	72	Sandy Clay
V	72	-	75	Sand
VI	75	-	77	Peat
VII	77	-	79	Sandy Clay
VIII	79	-	100	Sand

4.2. Settlement of the Soil

For the understanding of the results, the basic principles of the soil mechanics are explained in this section. Soil mechanics differs from structural mechanics, the fundamental difference is the material properties, and the structural engineer has to deal with more significant uncertainties.

4.2.1. Deformations in the Soil

Deformations in the soil are the result of an increase in the ground pressure by the weight of the building. The increase in the ground pressure causes compaction of the soil particles and results in settlement of the soil.

For low stress the soil follows the stress-strain relation of Hooke's law, which is the linear stress-strain relation of most of the structural materials. However for increasing load the soil will plasticaly deform, the stress-strain relation for soil is for low stress stiff, and with increasing stress, the stiffness will decrease as shown in Figure 4.4. The behavior of soil in loading and unloading is different from structural materials in the elastic range. After unloading, the soil will regain little of its lost volume due to compaction, as shown in Figure 4.4. The compaction of the soil grains is an irreversible process, and therefore the permanent deformations occur. The figure shows that when the soil is unloaded, a part of the strain is permanent. This is what happened due to the formation of glaciers on top of the soil.

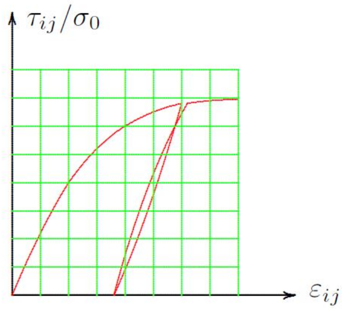


Figure 4.4: Loading and unloading of soil [24]

The stiffness of the soil depends on the compressive stress it is subjected to, for example, surface sand will slip through fingers, but under certain compressive stress it gains an ever-increasing stiffness and strength [24]. For this reason, high-rise buildings are founded on deeper situated sand layers.

4.2.2. Time-Dependent Behavior

The deformation of the soil depends on the characteristics of the soil. The settlement calculations shall include both immediate and time-dependent settlement. The settlement of the soil can be divided into the following three components [20]:

- s_0: Immediate settlement caused by shear deformation and volume reduction
- s_1: Settlement caused by consolidation
- s_2: Settlement caused by creep

The settlements s_1 and s_2 are only happening when the soil is considered as partially or fully saturated. These settlements are time-dependent and can continue after the construction of the building. The time-settlement line is shown in Figure 4.5.

The time-dependent behavior is related to the permeability of the soil. Fine grained soils like clay are particular soils with low permeability. The permeability of the soil is related to the size of the grains, so when the size of the grains is small (clay), the time-dependent behavior will go on for a long time. When soil with low permeability is loaded, the water pressure inside the voids will increase due to the weight of the building instead of the ground pressure. In the beginning, the load is carried by the water inside the voids, the reaction is that the water will leave the voids and transfer the load to the ground pressure. For sand the permeability is high, so the water can escape very quickly when the pressure is increased which results in a rapid transfer of the water pressure to the ground pressure.

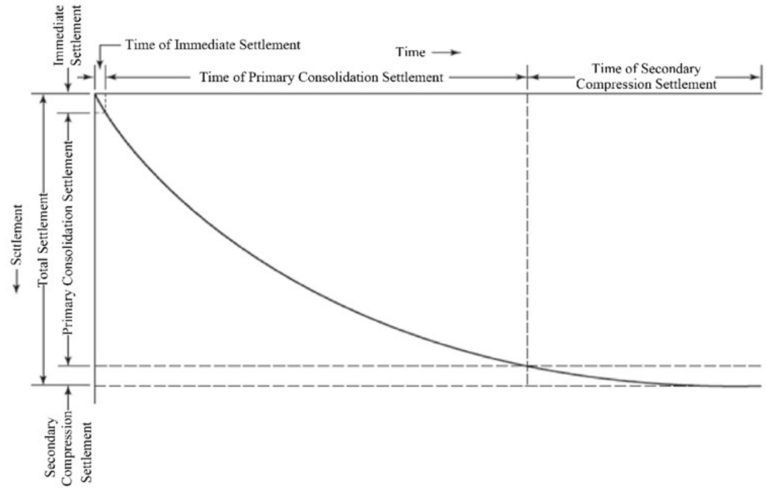


Figure 4.5: Time dependent behavior of the settlement of the soil [16]

Over-consolidation ratio

In Section 4.1 is explained that soil structures of Amsterdam and Rotterdam consist of layers that are over-consolidated. The OCR influences the time-dependent behavior of the soil. As already explained, the consolidation factor is the result of previous loads. The OCR is defined as the ratio of the maximum stress of the past and the present existing stress [21]. The soil of Amsterdam and Rotterdam are over-consolidated from a certain depth. This results in a stiffer behavior of the soil.

Figure 4.6 shows the stress-strain behavior of loading of pre-consolidated soil. As long as the ground pressure is below the maximum stress occurred in the past, the soil will act stiff but the stiffness will decrease when the load is above the maximum past stress.

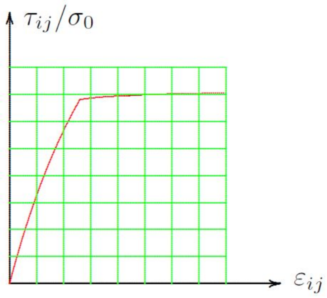


Figure 4.6: Loading of a over-consolidated soil [24]

4.2.3. Properties of the Soil Layers

The properties of the soil layers need to be determined in order to estimate the strength and the behavior of the soil under loading. Prior to constructing a building, geotechnical engineers investigate the location of the building and look into the soil conditions of the site. The soil properties are location specific and can vary significantly over the building area. Therefore, in reality investigation is needed for every building and every location. Determination of the soil properties of the location by preforming tests is out of scope for this research. The soil properties are retrieved from Table 2.b of the NEN9997, where characteristic values for the soil-properties for the main soil types are stated [20]. This table provides a range of the characteristics values for the soil-property, for this research the high values for stiffness are assumed otherwise the settlements of the soil are exceeding the expected settlements.

The soil structures of Rotterdam and Utrecht, as explained in Section 4.1, show both a small layer of peat under the foundation layer. The stiffness property of peat as mentioned in NEN9997 has a very low stiffness. This stiffness will increase the total deformations of the soil significant, and in addition it is not likely that a layer of uncompressed peat is situated at these depths. Therefore the stiffness properties of these layers are set equal to the stiffness properties of clay (mentioned in red in Table 4.4).

Table 4.4: Properties of the soils with in red the changed stiffness compared to the values of NEN9997

Soil type	γ_{dry} [kN/m³]	γ_{sat} [kN/m³]	C_v [m²/s]	OCR [-]	C_p [-]	C'_p [-]	C_s [-]	C'_s [-]
Gravel	19	21	0.1	1	4800	1500	∞	∞
Sand	19	21	0.1	1	4000	1500	∞	∞
Sandy Clay (Loam)	21	21	10^{-6}	1	480	100	3000	2500
Clay	19	19	10^{-8}	1	100	30	750	500
Eem-clay	19	19	10^{-8}	2.0	100	30	750	500
Kedichem-clay	19	19	10^{-8}	1.5	100	30	750	500
Peat	12	12	10^{-4}	1	100	30	750	500

4.2.4. Uncertainty of the Soil

The properties of the soil materials are measured and estimated by performing onsite and laboratory tests. Based on the results of these test the behavior of the soil is estimated and therefore variability of material properties is inevitable. The structural engineer has to deal with uncertainty for the settlement of the soil. For this research, the uncertainty is assumed to be 30%.

4.3. Soil-Structure Interaction

The deformation of the substructure is the result of the soil structure-interaction (SSI). The SSI is the interaction between the stiffness of the soil and the stiffness of the superstructure which makes the assignment to design a building on soft soil complicated. The stiffness of the soil and the superstructure both influence the final settlement and, therefore also the reaction forces of the supports. To explain the SSI, a simple structural calculation is made for a beam supported by multiple springs. Figure 4.7 shows the schematic drawing of the structure where the stiffness of the soil is represented by the springs, and the stiffness of the structure is represented by the bending stiffness of the beam.

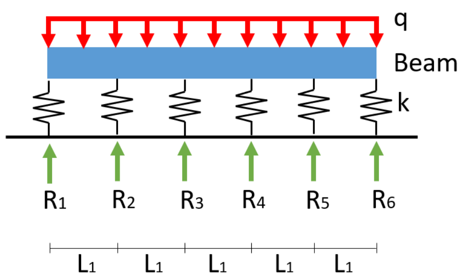


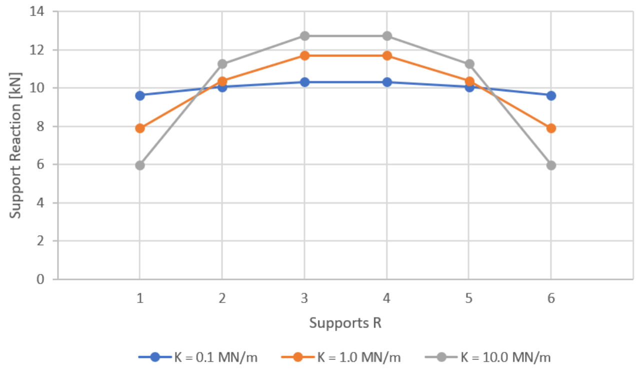
Figure 4.7: Example structure to explain the change in reaction force

Table 4.5 shows the properties used for the calculation. The results of the support reactions are shown in the graphs of Figure 4.8. The graphs show the variation in reaction forces for the changing stiffnesses of the beam and the springs.

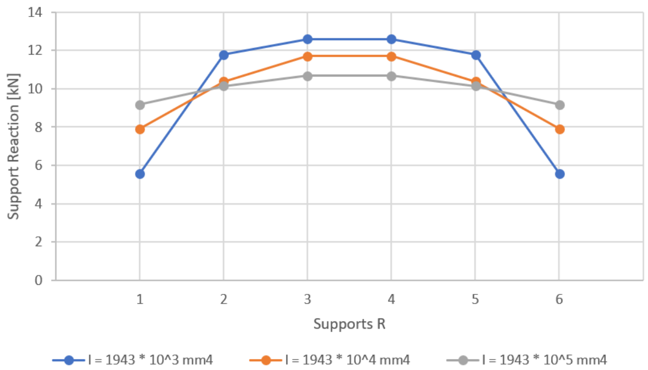
Table 4.5: Initial properties for the example calculation

Property		Values
q	[kN/m]	10
L_1	[m]	1
E	[MPa]	210000
I_{beam}	[mm^2]	1943 * 10^4
k_{soil}	[MN/m]	1.0

In Figure 4.8a the reaction forces are shown for varying stiffness of the soil, and in Figure 4.8b the reaction forces are shown for varying stiffness of the structure. The figures show that both the stiffnesses influence the reaction forces. For varying stiffness of the soil can be concluded that for stiffer soil, the load is the most supported by the middle support (R_3 and R_4) and when the soil is less stiff the load is supported more equally by the supports. For the varying stiffness of the structure, it can be concluded that when the structure is stiff (high I_{beam}), the load will be distributed more equally over the support compared for a less stiff structure (the small I_{beam}). For the less stiff structure, the load is more supported by the middle of the structure (R_3 and R_4) compared to the edge supports (R_1 and R_6).



(a) Stiffness of the soil (stiffness structure is IPE200)



(b) Stiffness of the structure (stiffness soil is 1 MN/m)

Figure 4.8: The results of the example calculation to show the influence of the stiffnesses of the soil and the superstructure on the support reactions

4.3.1. How the Soil Structure Interaction is Modeled

For this research the SSI is modeled in the program SCIA Engineer. The complexity of modeling the SSI is the connection between two different disciplines, the geotechnical engineer and the structural engineer. The structural analysis is done in the program SCIA engineer, and the deformation of the substructure is calculated with the programs D-Settlement and D-Foundations. It is necessary to create interaction between these programs in order the model the reality in the right way. As already explained the deformation of the substructure is the sum of s_1 the deformations due to settlement of the foundation piles and s_2 the deformation of the soil under the foundation piles. In Figure 4.9 the parts s_1 and s_2 are shown and how these parts are modeled in the SCIA model. The behavior of the substructure is modeled as two consecutive springs, for which the stiffness is still to be determinate with the help of the programs D-Settlement and D-Foundation. The sequence of the springs is swapped compared to the positions of s_1 and s_2 in order to simplify the SCIA Model. In Appendix C is proven that this change in sequence does not change the outcome.

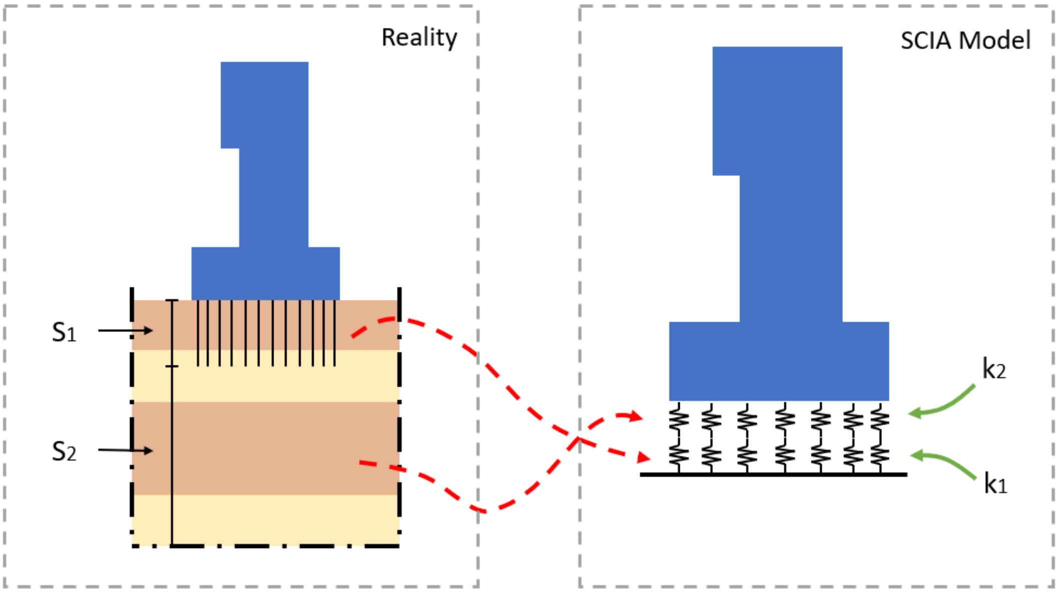


Figure 4.9: Schematic drawing of how the reality is modeled with the consecutive springs in the SCIA model (the springs are swapped compared to the reality in order to simplify the model)

The stiffness of k_1 is determined in Subsection 4.3.2 and the stiffness s_2 is determined in Subsection 4.3.3.

The building in the SCIA model is supported by 50 supports. Figure 4.10 shows the location of the supports with the red dots. The supports are located underneath the columns and the stability core. This figure shows also the considered cross-sections.

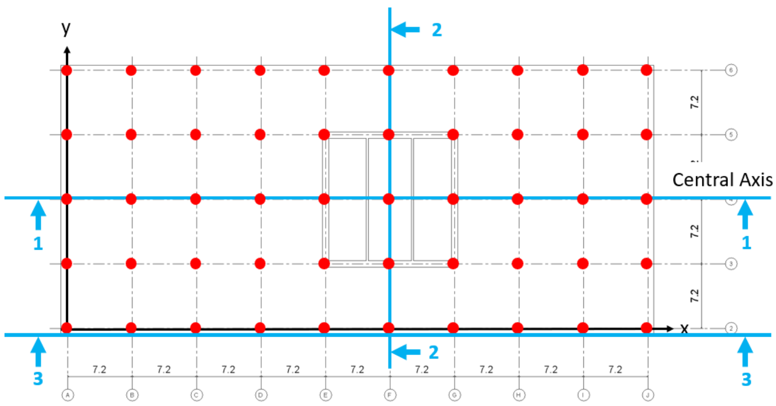


Figure 4.10: Top-view of the foundation floor with the locations of the supports and the considered cross-sections

4.3.2. Determination Spring-Stiffness k_1

The stiffness of the foundation pile is determined with the program D-Foundations. In the program D-Foundations, the properties of the foundation piles are inputted together with the CPT of the locations. The program D-Foundations calculates the settlement of the foundation piles under an applied load. By combining the deformations with the applied load, the load-settlement curves of the foundation piles for the three locations is determined. The results for the settlements of the foundation piles for the three locations are shown in Table 4.6.

Table 4.6: Deformation of the foundation piles under applied loads for the three locations

	Displacement Top of the Pile		
Pile-load [kN]	Amsterdam [m]	Rotterdam [m]	Utrecht [m]
0	0	0	0
1000	0.005	0.005	0.002
2000	0.013	0.018	0.007
2600		0.066	
3000	0.034		0.022
3500	0.065		0.063

The load-settlement curve for the three locations are shown in Figure 4.11.

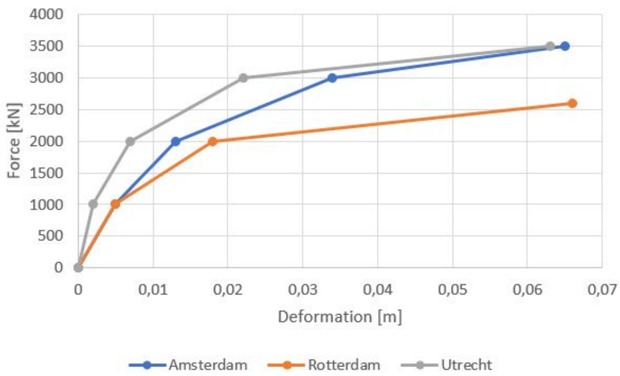


Figure 4.11: Load-settlement curve of the foundation pile per location

In the SCIA model, the springs k_1 are modeled as non-linear spring supports. These supports have the option to be non-linear, so the results of the load-settlement curve are inputted as the properties of the non-linear spring supports in SCIA. In this way, the stiffness of the foundation piles is modeled.

Determination of the number of foundation piles

The number of foundation piles is still te be determined. As already mentioned the reaction forces of the supports depend on the soil-structure interaction. For this reason, the number of piles was not earlier calculated. In the next paragraph, the stiffness of the soil is calculated, this stiffness is used to determine the number of piles needed to meet the ULS requirement.

The support reactions are calculated for the settlement at two years when the building is finished, and for the final settlement at 25 years. For this calculation the assumption is made that the maximal reaction force is reached after two years due to finishing the building and that reaction forces are shifting during the 23 years to a final state at 25 years, assuming that no big settlements will occur anymore after 25 years. The number of foundation piles must have enough capacity that it will not exceed the ULS limit during the lifetime. So, the maximum of the two support reactions is taken into account and is divided by the load-bearing capacity calculated by the program D-Foundation in Subsection 3.4.2. In this subsection the groups of the foundation piles are also mentioned.

$$piles = \frac{max(F_{z,t=2years}; F_{z,t=25years})}{R_{c,d}} \tag{4.1}$$

For the concrete alternative, the number of foundation piles per locations are shown in Table 4.7.

Table 4.7: The number of foundation piles for the concrete alternative

Pile Group	Amsterdam	Rotterdam	Utrecht
P1	3	6	3
P2	7	12	7
P3	14	17	14
P4	12	17	12
P5	11	14	11
P6	20	22	22

For the hybrid alternative, the number of foundation piles per locations are shown in Table 4.8.

Table 4.8: The number of foundation piles for the hybrid alternative

Pile Group	Amsterdam	Rotterdam	Utrecht
P1	3	5	3
P2	6	10	6
P3	11	14	12
P4	10	14	10
P5	9	12	9
P6	16	18	18

For the steel alternative, the number of foundation piles per locations are shown in Table 4.9.

Table 4.9: The number of foundation piles for the steel alternative

Pile Group	Amsterdam	Rotterdam	Utrecht
P1	3	5	3
P2	5	8	5
P3	8	10	9
P4	8	11	8
P5	7	9	7
P6	12	13	13

4.3.3. Determination Spring-stiffness k_2

The stiffness k_2 represents the stiffness of the soil. The settlement of the soil is the result of the SSI and therefore depending on the stiffness of the soil and the stiffness of the superstructure. The complexity is that two different programs calculate these stiffnesses. The stiffness of the superstructure is calculated with the SCIA model and the settlements of the soil are calculated with the program D-Settlement.

The spring stiffness k_2 is determined by an iteration of the following steps (Figure 4.12 shows a schematic drawing of the method used for the determination of k_2):

1. In the first step, an assumption is made for the stiffness for the k_2 is made. The structural calculation of the SCIA model is done with the assumed spring stiffness supporting the building.
2. The necessary results of the structural calculations are the reaction forces of the springs and additional the settlement-line (the red line in Figure 4.12).
3. For the third step, the reaction forces are inputted in the program D-Settlement. The soil structure is built-up with the information of the layers of the soil structure together with the mentioned soil properties. The reaction forces are divided over the corresponding area to result in a q-load, which is applied to the soil. The q-load is applied at the bottom of the foundation piles, assuming that the foundation piles are transferring the applied load directly to the soil, and neglecting the load-bearing part of the shaft friction of the foundation pile.
4. The program D-Settlement calculates the settlement of the soil due to the applied q-loads. The result is the settlement line (the bleu line in Figure 4.12)
5. For the final step the two settlements lines are compared with each other. When the right spring stiffness of k_2 is assumed in the first step, the two settlement lines are comparable, and therefore the iteration is done. When the wrong stiffness of k_2 is assumed in step 1 and used in the structural calculation, this results in two settlements lines that are not comparable. Then the iteration has to be done again with a new assumed stiffness to ensure that the two settlement lines are overlapping each other.

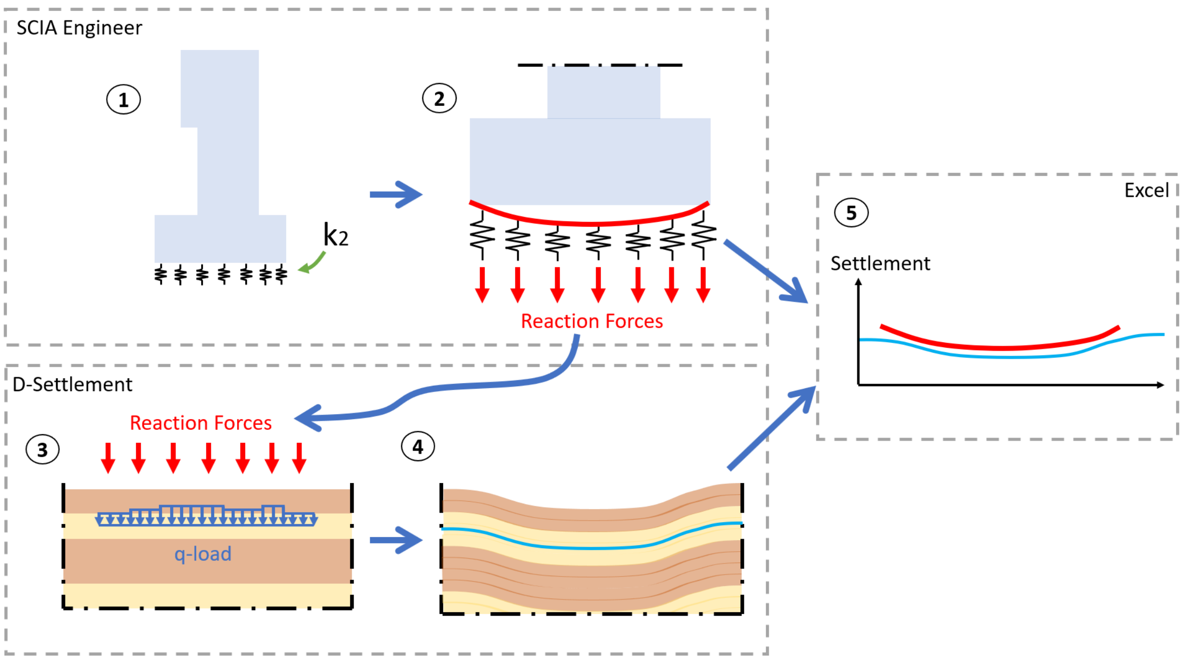


Figure 4.12: Overview of the method used to determine spring-stiffness k_2

The calculation is executed in 3D and this results in the 3D settlement of the soil. Figure 4.13 shows the settlement of the soil over the plot for the concrete building at the location of Rotterdam. The maximum settlement occurs under the stability core of the building.

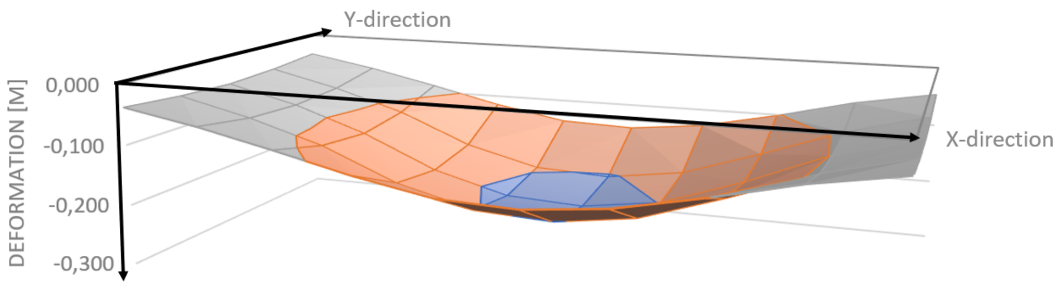
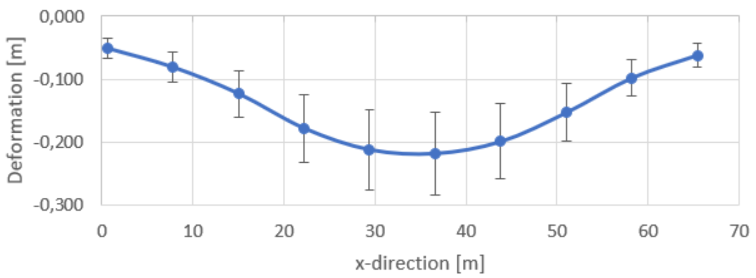
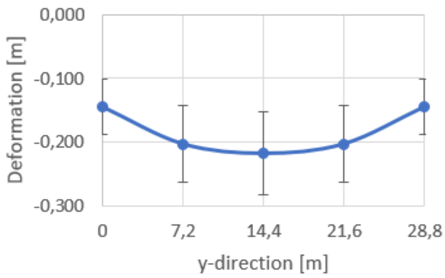


Figure 4.13: 3D view of the calculated deformation of the soil over the plot of the building for the concrete alternative in Rotterdam

Figure 4.14 the result for the settlement of the soil are shown for cross-section 1-1 and 2-2. The results show that the differential deformation for the long cross-section are bigger compared to other cross-section, therefore the following results are shown in cross-section 1-1. The structure is more stiff in y-direction compared to the x-direction.



(a) Deformation of the soil in cross-section 1-1



(b) Deformation of the soil in cross-section 2-2

Figure 4.14: Deformation of the soil in the two cross sections

For the determination of the stiffness, the settlement lines of the two models are compared over the central axis of the building, as shown in Figure 4.10. Figure 4.15 shows the graphs used to determine the springs stiffness k_2 for the concrete building at the three locations. In the figures, the settlement lines of D-Settlement model (the continuous lines) and the settlement lines of the SCIA model (the dashed lines) are shown. The FIT-line is shown in the graphs too, this is the line after deciding which stiffness is used and by shifting the corresponding line of SCIA vertically, which represents the uniform settlement.

The FIT-line is the result of the following two properties:

1. **Differential settlement** is the result of the deformations in the springs in the SCIA model which represent the differential deformation of the soil.
2. **Uniform settlement** is added manually to the deformation line to fit the settlement line of D-Settlement.

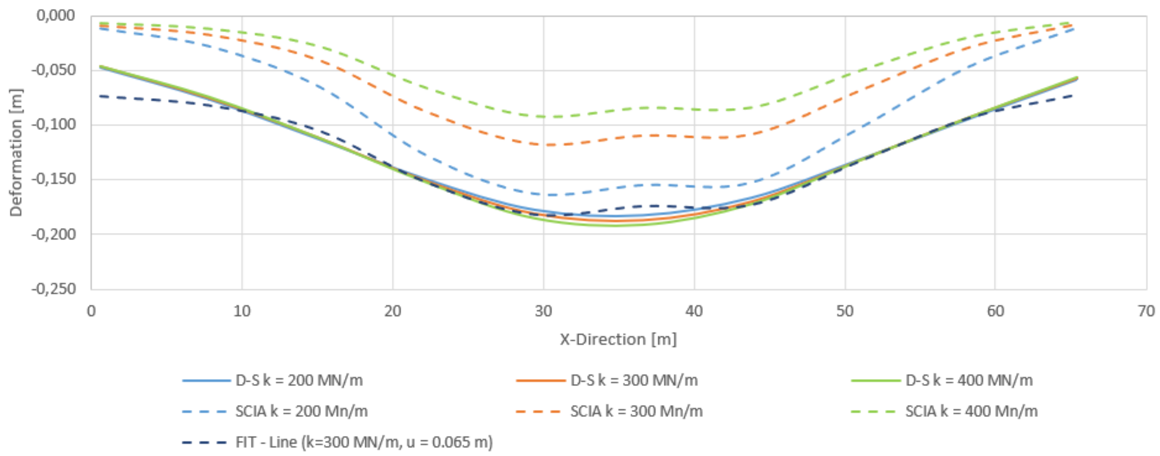
The results of the determined spring-stiffness and the amount of uniform displacement are shown in Table 4.10.

Table 4.10: The determined spring-stiffness and uniform settlement for k_2

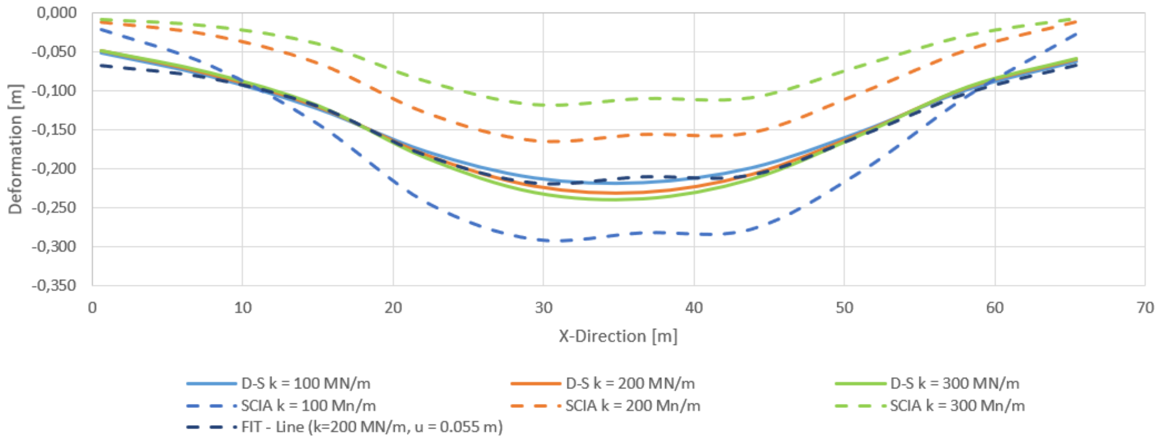
Alternative	location	k [MN/m]	$u_{uniform}$ [m]
	AMS	300	0.065
Concrete	ROT	200	0.055
	UTR	600	0.045
	AMS	300	0.055
Hybrid	ROT	200	0.045
	UTR	600	0.035
	AMS	300	0.045
Steel	ROT	200	0.035
	UTR	600	0.03

The results show that spring stiffness representing the soil is independent of the weight and stiffness of the building. Of the three locations the soil of Rotterdam is the least stiff. Therefore the most significant settlements are happening at the location of Rotterdam. The soil of Utrecht has the highest stiffness and Amsterdam is in between, this conclusion is in line with the expectations after observing the soil structures of the locations as discussed in Section 4.1. Looking to the settlement lines in Figure 4.15 it can be seen that the FIT-line is better comparable for the settlement line of D-Settlement for a higher spring stiffness. For stiffer soil, the loads of the building are less distributed because the deformations are smaller, which results in a more direct loading to the supports. The direct loading causes the less accuracy of comparison of the D-Settlement and SCIA settlement lines for stiffer soil.

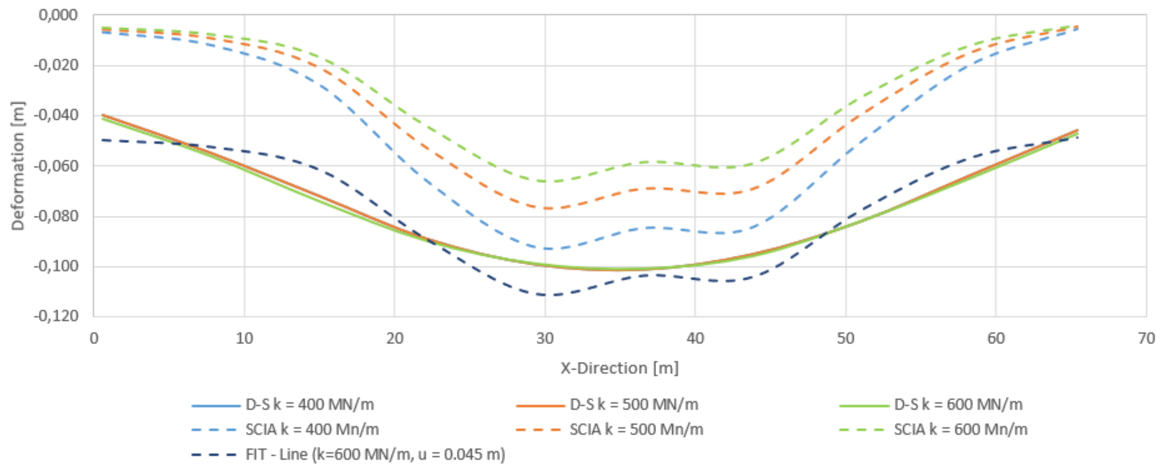
Table 4.10 shows the spring stiffness and the uniform displacement. The table shows that the uniform settlement depends on the alternative of the structure and that this settlement also depends on the location. It is as expected that the concrete building settles more compared to the hybrid alternative and that the uniform settlement of the steel alternative is the smallest.



(a) Amsterdam



(b) Rotterdam



(c) Utrecht

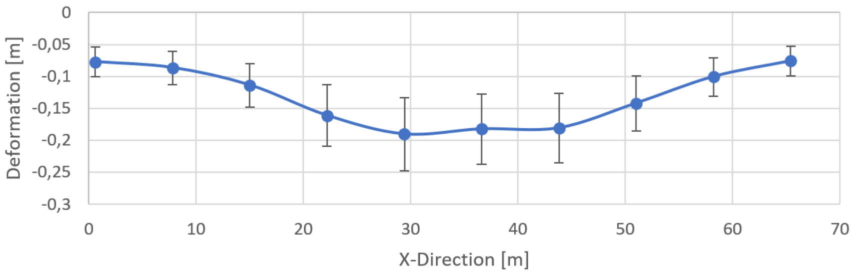
Figure 4.15: Graphs of the results used to determine the value of k_2 for the concrete alternative per location

4.4. Results

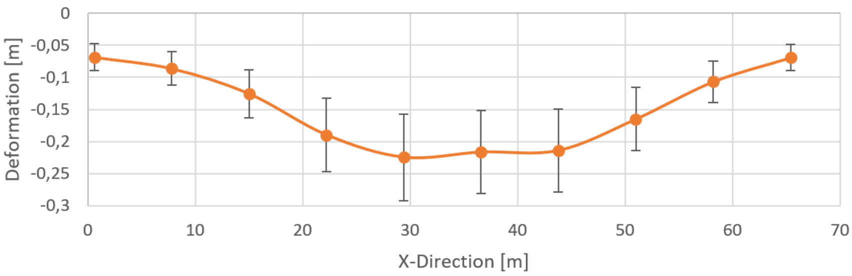
In the following subsections the results of the deformations are shown and explained. The deformations are calculated with the SCIA model. In the SCIA model the superstructure is modeled and is supported by the 2 springs per support with stiffness k_1 and k_2. In Figure 4.10 is the location of cross-section 1-1 shown, which is used to show the deformations in this section.

4.4.1. Results of the Total Deformation

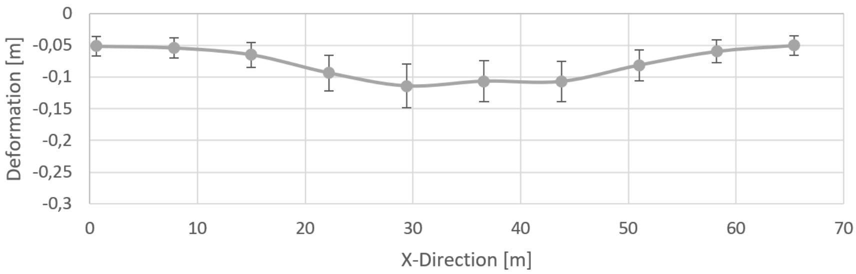
Figure 4.18 shows the total deformations of the substructure for the concrete alternatives at the three locations. The total deformation of the foundation floor is measured. The results are shown in cross-section 1-1 with on the horizontal axis the x-direction and on the vertical axis the deformation of the foundation slab with the error-bars for the uncertainty of the soil behavior.



(a) Amsterdam



(b) Rotterdam



(c) Utrecht

Figure 4.16: Results of the total deformation of the foundation slab for cross-section 1-1 per location

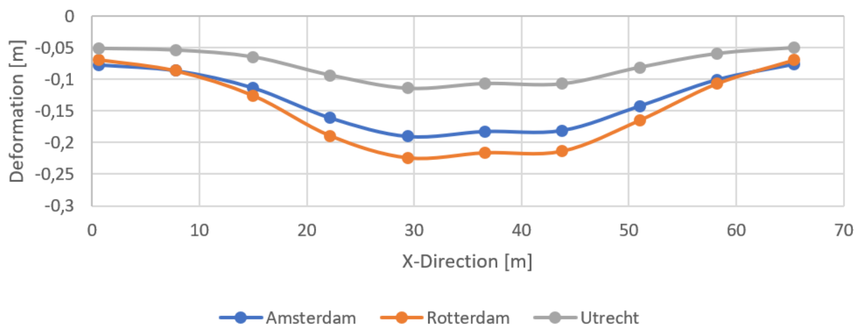
The results of the total deformation show that the most considerable deformation occurs at Rotterdam. The total deformation in Amsterdam is slightly smaller than the total deformation of Rotterdam. The difference is the result of the thickness of the compressible layer. The thickness of the Kedichem-layer is significantly larger than the thickness of the Eemclay layer. The results does not clearly show the difference in thickness. The OCR is the only difference, which can causes the comparable total deformation for these locations independent of the thickness of the compressible layers. The OCR of the layer of Kedichem is higher compared to the OCR of the Eemclay layer, which results in a stiffer behavior of the clay-layer in Rotterdam compared to the stiffness of the clay-layer in Amsterdam. In Utrecht, the total deformation is significantly smaller, and to explicitly emphasize the differential deformation is much smaller compared to Amsterdam and Utrecht.

The total deformation of the concrete alternative in Rotterdam, as shown in Figure 4.16a, has a similar magnitude as the deformation calculated for the building De Rotterdam, as explained in Section 2.5. The geometry and the dimensions are comparable, only the width of De Rotterdam is larger, with two additional towers. Therefore the total deformation is slightly larger in this case, compared to the total deformation shown in Figure 4.16a.

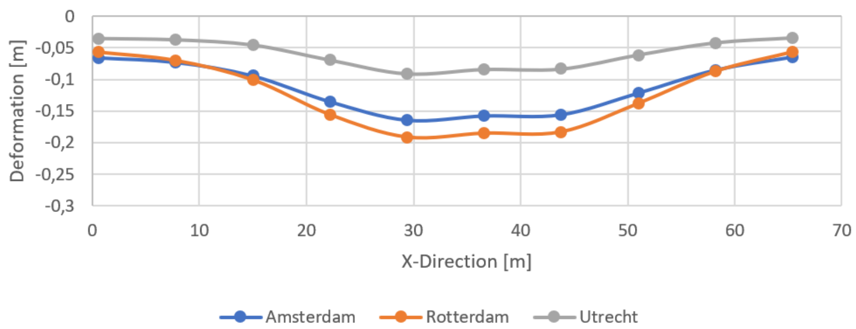
The error bars are shown in the graphs too. In the figures can be seen that the uncertainty for Utrecht are the smallest compared to the other two locations. The uncertainty is 30% of the deformations, hence the uncertainty for Rotterdam and Amsterdam are bigger because the total deformation is larger compared to Utrecht.

Total deformation of the 9 options

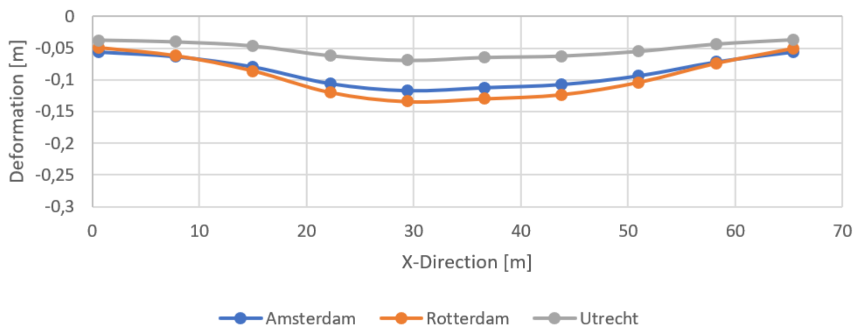
The total deformation for the three alternatives at the three locations is calculated in this research. Figure 4.18 shows the total deformations for the three alternatives at the three locations. The results are shown for the same properties as used in Figure 4.16, but only without the uncertainties bars to present a clear overview of the total deformation.



(a) Concrete alternative



(b) Hybrid alternative



(c) Steel alternative

Figure 4.17: Results of the total deformation of the foundation slab for cross-section 1-1 without the error bars

The total deformations for the nine options show a clear overview, which is in line with the expectations. The total deformation of the concrete alternative, for which the weight is comparable to the weight of the building De Rotterdam, is in line with the calculated deformations of the building De Rotterdam. The deformation lines show that the concrete alternative causes the biggest deformation compared to the other two options.

The three graphs of Figure 4.17 show that the largest deformations are happening in Rotterdam. The deformations of Amsterdam and Rotterdam are comparable, but the deformation in Rotterdam slightly larger at all the locations. As explained, the total deformation consist of uniform and differential deformations. When the deformation lines of Amsterdam and Rotterdam are compared, it shows that the differential deformation in Rotterdam is bigger compared to the differential deformation of Amsterdam. This can be observed in the graphs because the deformation of Rotterdam is larger in the middle of the building and to the edges of the building the deformation of Rotterdam is smaller compared to the deformation in Amsterdam. Therefore the problems in the building itself are bigger when the building is constructed at Rotterdam.

Total deformation of the three alternatives in Rotterdam

For a clear comparison between the total deformation of the three alternatives, the total deformations of the three alternatives in Rotterdam are shown in Figure 4.18 without the error bars for the uncertainty.

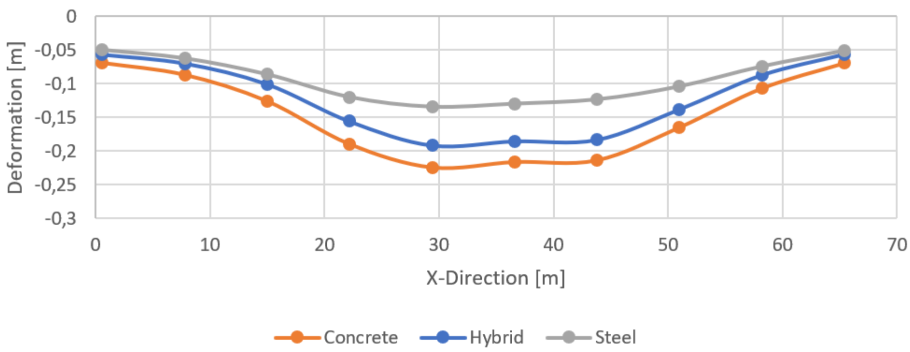


Figure 4.18: Results of the total deformation of the foundation slab in cross-section 1-1 of the three alternatives in Rotterdam without the error bars

The results shows that the concrete alternative caused the largest deformation. With this results the relation between the weight of the structures and the total deformations are proven. The deformation line of the Hybrid structure is in the middle of the graph close to the deformation of the concrete alternative, but on the edges it is comparable to the total deformation of the steel alternative. This shows that the surrounding steel structure (around the core) of the hybrid alternative is not as stiff as the surrounding concrete structure of the concrete alternative.

4.4.2. Components of the Deformation

The total deformation of the substructure is the sum of the deformation of the foundation piles and the deformation of the soil, as explained in Subsection 4.3.1. Figure 4.19 shows the different components of the total deformation. In this figure can be observed that the sum of the three components results in the total deformation. These deformations are measured at the middle support, underneath the stability core.

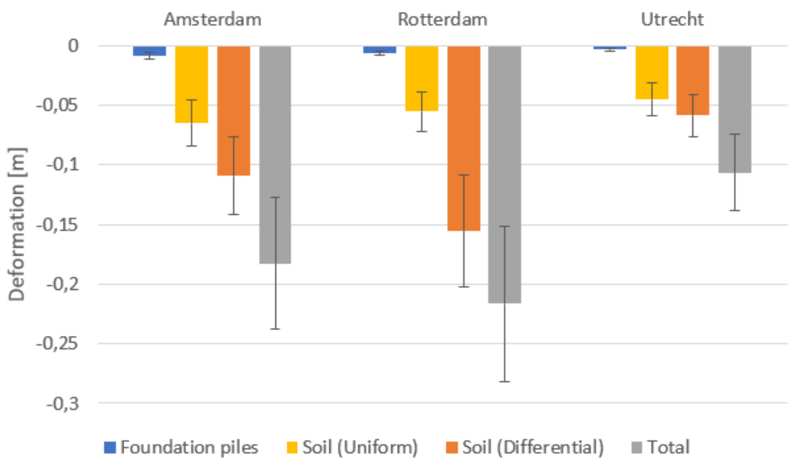


Figure 4.19: Components of the maximal deformation for the concrete alternative in Rotterdam

Figure 4.20 shows the components of the total deformation is once more, but now for cross-section 1-1. In this graph the deformations are shown over cross-section 1-1 for the concrete alternative at Rotterdam. In this graph can be observed that the deformation of the soil causes almost the total deformation.

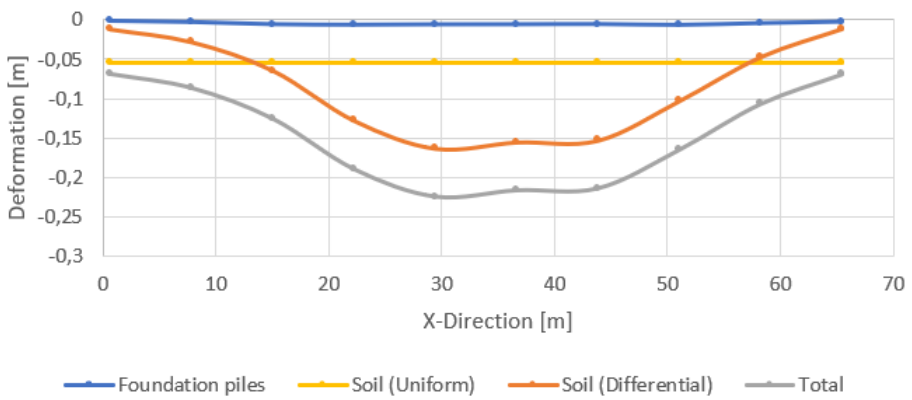


Figure 4.20: Components of the deformation in cross-section 1-1 for the concrete building at Rotterdam

Figure 4.19 and Figure 4.20 shows that the deformations of the foundation piles are small compared to the deformation of the soil. In reality the pile deformations will be larger in reality. The causes for this difference is that for this research the number of piles are determined in a simplified way, this can result in an exaggeration of the number of piles. Therefore the stiffness of the foundation piles is too high, which results in a small deformation as shown in this graph.

As explained in Section 2.1, the differential deformations create the problems regarding damage to the structure itself or damage to the facade elements. Figure 4.19 shows that the differential deformation in Rotterdam is larger with a smaller uniform deformation compared to Amsterdam. Therefore the problems in the structure at Rotterdam will be bigger.

4.4.3. Time-Dependent Behavior of the Deformation

As explained the settlement of the soil has a time-dependent behavior. Figure 4.21 shows the time-dependent settlement for the maximal settlement at the location of Rotterdam. For the calculation the construction stages explained in the introduction of Chapter 5 are taken into account.

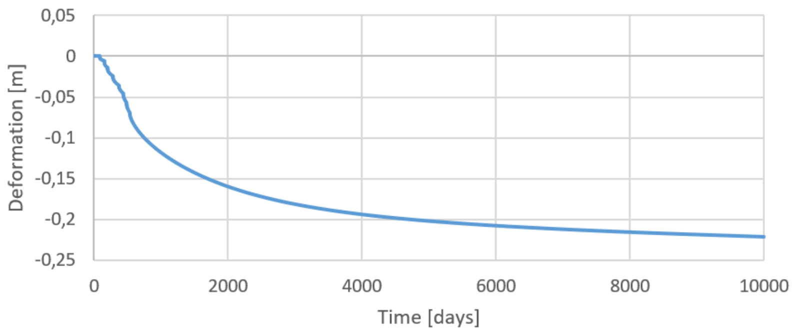


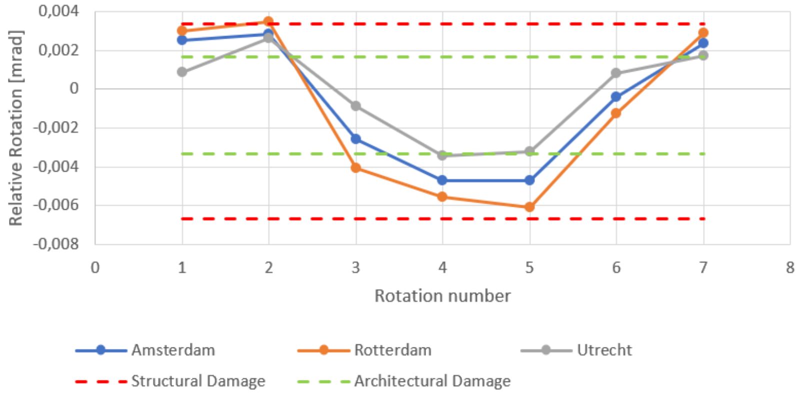
Figure 4.21: Deformation of the soil over time for the maximal deformation of the concrete alternative in Rotterdam

The graph shows that the deformation will increase over time to reach the maximal deformation long after the building is finished. The time-dependent behavior is caused by the deformation of the compressible layer under the foundation piles. The first part of the settlement is occurring during construction, the immediate settlement. After 500 days the settlement curve shows the consolidation phase, which is gradually decreasing over time to reach the final settlement and the end of the graph the settlement rate is approximately zero. This can be indicated as the secondary compression settlement, as shown in Subsection 4.2.2.

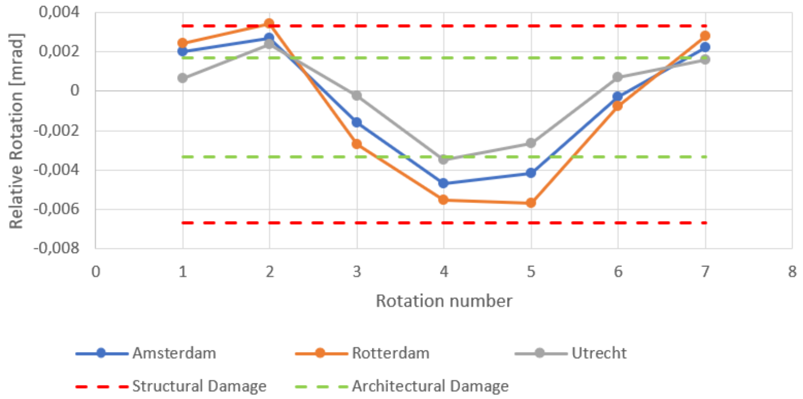
4.4.4. Relative Rotation for the Concrete Alternative

In Figure 4.22 the results of the relative rotation of the foundation are shown for the location of Rotterdam. In Section 2.3 the limits of the NEN9997 are stated. These limits are shown in the figures with the horizontal dashed lines, the limit for the architectural damage with the green color and the limit for the structural damage with the red color.

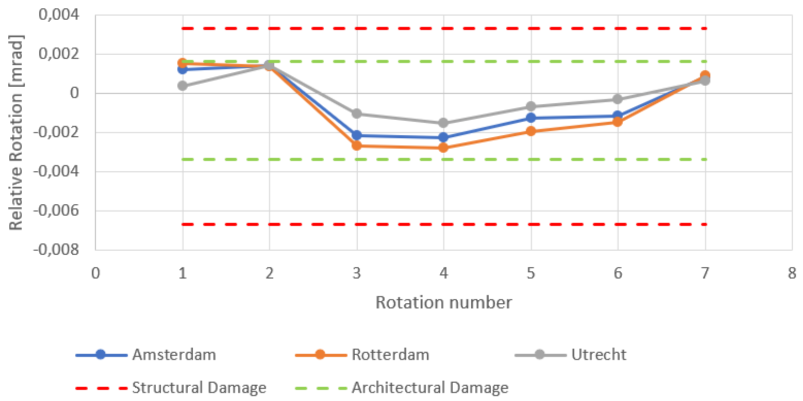
The results of the relative rotation of the foundation slab for the three alternatives at the three locations shows that the limit for structural damage is not significantly exceeded. The steel alternative does not exceed the limit for the architectural damage. The concrete and hybrid alternative at Utrecht only slightly exceed the architectural damage under the core and on the left side of the building. The concrete and hybrid alternative at Amsterdam and Rotterdam are exceeding the architectural limit and are close to the structural limit. It can be concluded that for the concrete and hybrid alternative mitigating measures are needed based on the NEN9997 limit for relative rotation. The building at Utrecht is safe for structural and architectural damage based on this limit.



(a) Concrete alternative



(b) Hybrid alternative



(c) Steel alternative

Figure 4.22: Results of the relative rotation of the foundation in cross section 1-1 in Rotterdam per alternative

5

Mitigating Measures

In this chapter the mitigating measures are explained and compared. In the first section of this chapter the problems of exceeding deformations are explained in order to understand how the mitigating measures influence the issues and in the second section the considered mitigating measures are explained.

5.1. Problems Encountered

In this section the deformation and rotations of the floors and the facades are shown. Before showing and explaining these deformations, the input parameters of the calculation are explained. The deformations in the structure are calculated for the concrete alternative at Rotterdam. To create a clear image of the deformations happening in the structure due to deformation of the substructure, the deformation of the selected levels are calculated. The selected levels are shown in Figure 5.1. These floors are selected in order to align with the assumed construction sequence. In the bottom of this figure are also the floor-numbers shown, these numbers represent a span of the floor. Per floor-span the maximum deformation and rotation are calculated with the SCIA model.

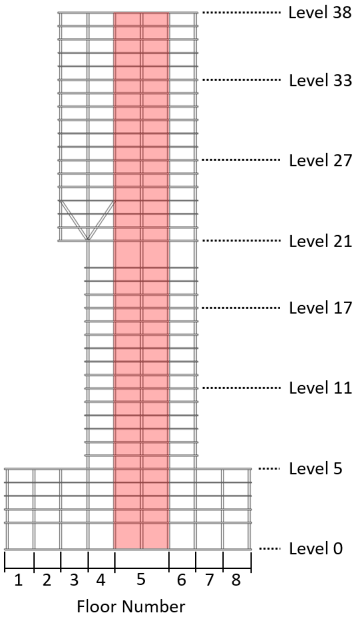


Figure 5.1: Selected floors and the floor numbers to show the results

Figure 5.2 shows the 9 construction stages which are used in this research. In the figures the contours of the structure are presented in gray, the primary structure in red and the facade elements in blue.

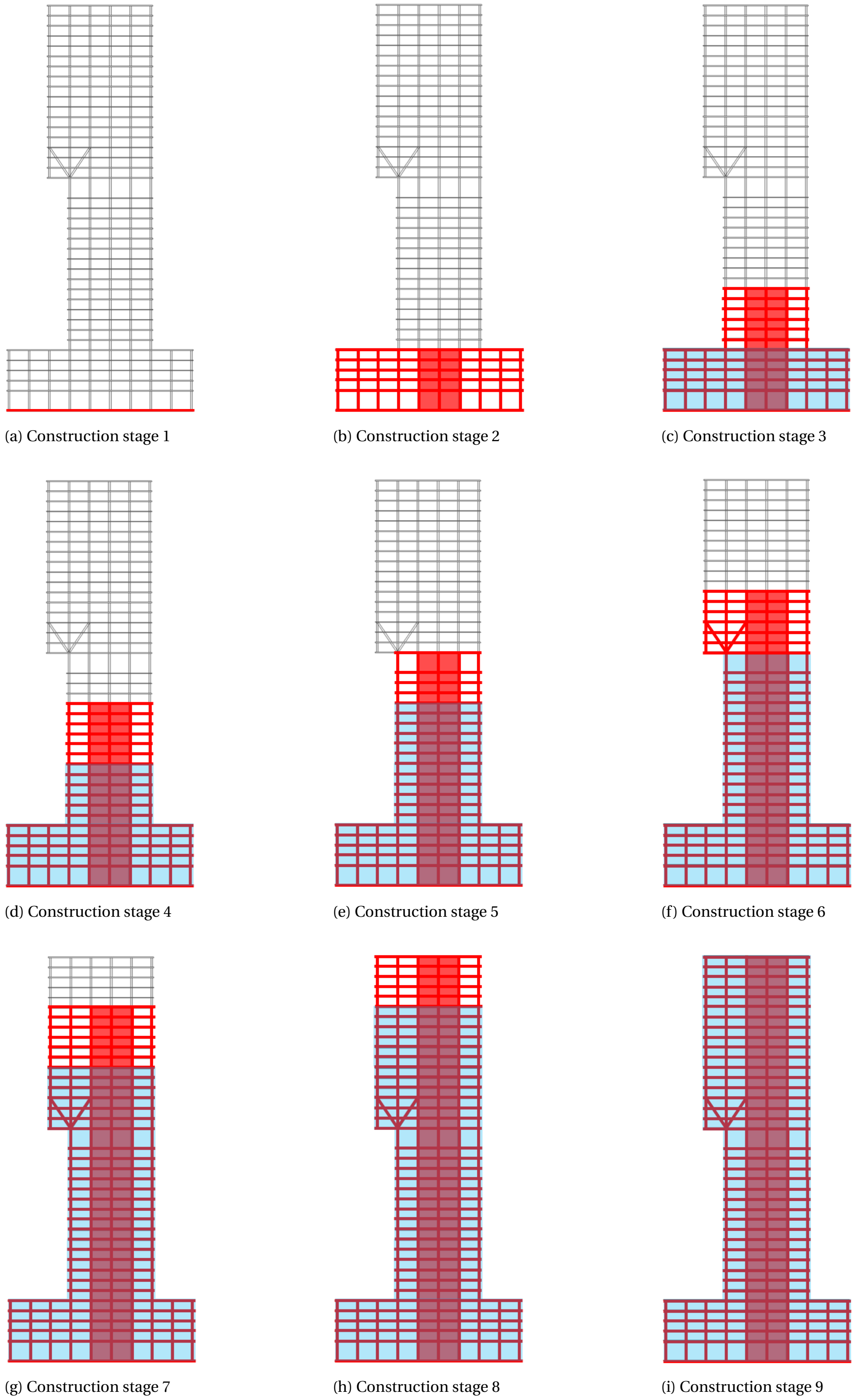


Figure 5.2: Assumed construction sequence used for the structural analysis

This research focuses on the floor and the facade support rotation. In Section 2.3 are the limits for the floor and facade rotation determined. The floor and facade rotation are both analyzed in one cross-section, both of these cross-sections are shown in Figure 4.10. In cross-section 1-1 is the floor rotation measured, and in cross-section 3-3 is the facade rotation measured.

Time dependent settlement

In addition to the construction stages, the deformation of the soil is also time-dependent. Both of these time-dependent properties are of importance to distinguish how significant the deformations of the structural elements are. In Section 4.4.3 the settlement over time of the compressible layer is shown for the location of Rotterdam. This settlement line is calculated with the construction stages of Figure 5.2 taken into account. This curve is calculated with the program D-Settlement. The structural analysis is done with the SCIA model in which also the deformations are calculated. Figure 5.3 shows the results of the deformations over time for both the calculations. In the figure can be seen that the settlement line of D-Settlement is continuous and the deformation of the SCIA model are measured for specific time in order to combine the construction stages and the time-dependent behavior of the soil.

In Figure 5.3 the construction stages are mentioned for which the deformations are shown in the following subsection.

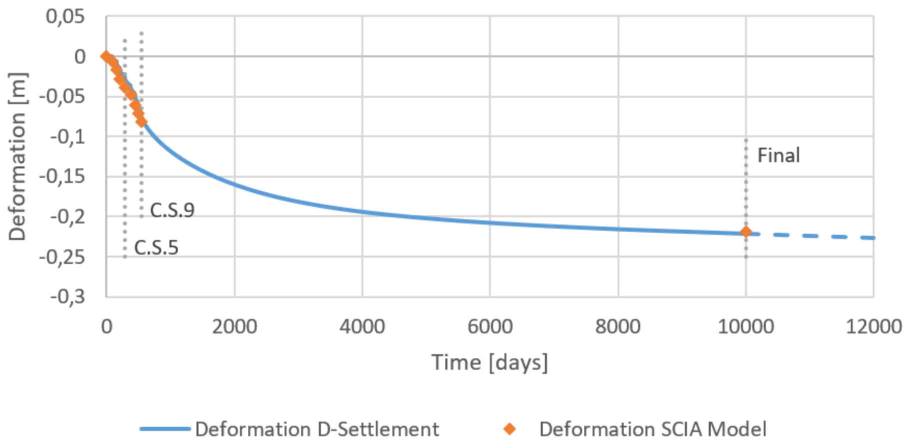


Figure 5.3: Time-dependent deformation of the soil compared to the deformation results in the SCIA model

Figure 5.4 shows a detail of the previous figure. In this figure can be seen than for the deformation of the SCIA model is slightly larger that the deformation line of the D-Settlement model. This graph shows clearly when the different construction stages are applied. The total weight of the part of the structure which is constructed in the construction stage is applied in one step to the models. This results in the downward jumps of the D-Settlement line. In reality the building is constructed gradually, this will result in a smooth continuous line of the settlement of the soil. For this research the calculated line with the jumps is used to compare with the deformations of the SCIA model. It can be concluded that the deformations of the SCIA model are an approximation of the settlement of the D-Settlement program.

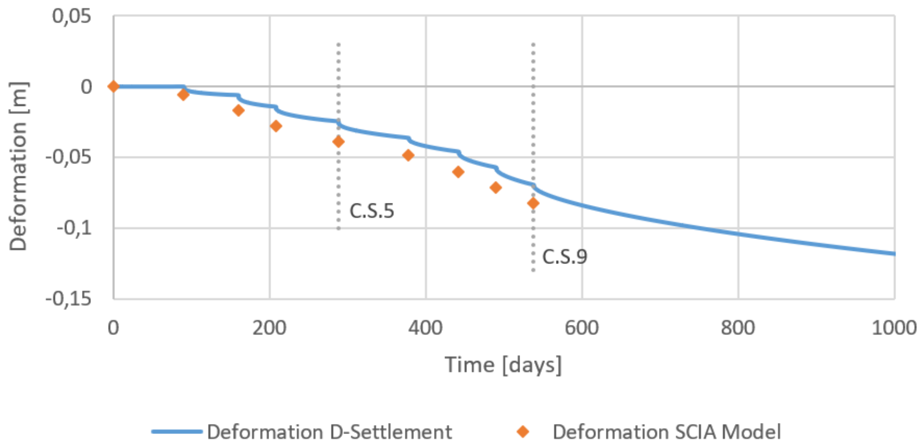


Figure 5.4: Detail of the time-dependent deformation of the soil compared to the deformation results in the SCIA model

5.1.1. Final Deformations

The limit for the floor and facade rotation used for the structural analysis are determined according to the formulas in Section 2.3.

Floor rotation

In Chapter 3 is the span of the floor set on 7.2 m. Substituting this span of the floor in Formula 2.6 results in the limit for vertical deformations of:

$$w_{floor} = \frac{7200mm}{500} = 14.4mm$$

The limit for floor rotation is determined by substituting the limit for vertical deformation in Formula 2.8.

$$\theta_{floor} = \frac{3}{2} * \frac{14.4}{0.5 * 7200} = 6mrad$$

Facade Support rotation

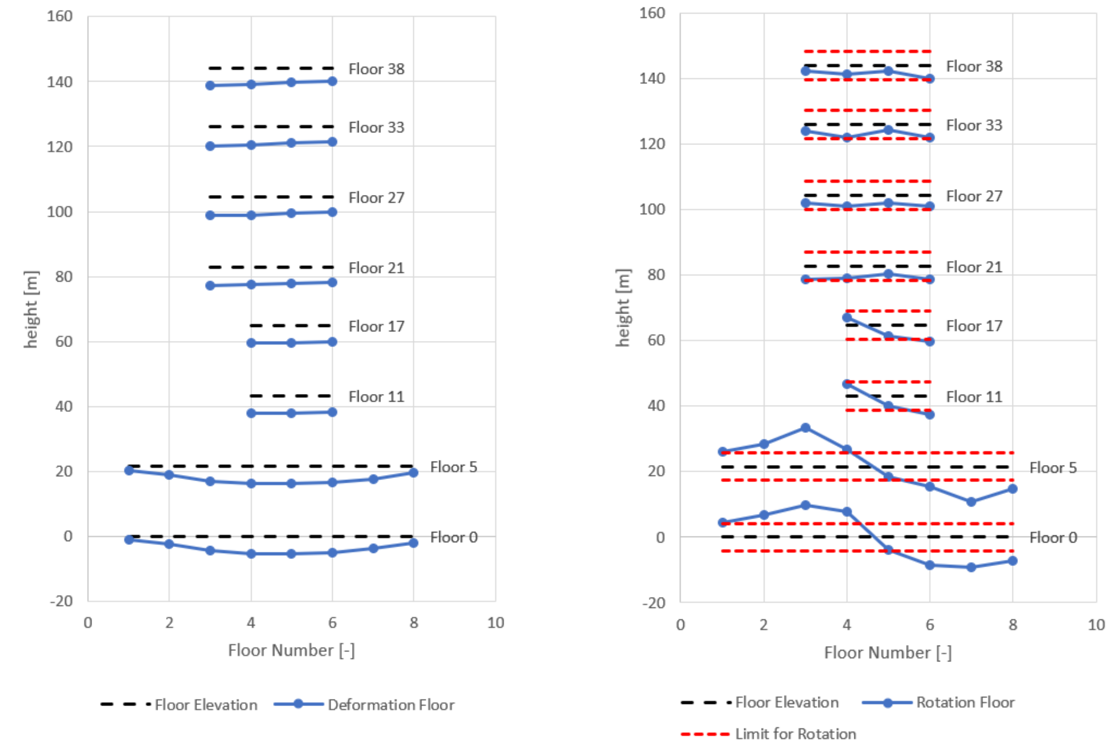
In Section 2.3 the vertical deformation of the facade supporting element is set on 10 mm. The limit for rotation of the facade supporting elements is determined by substituting the vertical deformation limit in Formula 2.8.

$$\theta_{floor} = \frac{3}{2} * \frac{10.0}{0.5 * 7200} = 4.1mrad$$

Final deformation of the floors

Figure 5.5 shows the floor deformation and the floor rotation of the selected floors for the final settlement in cross-section 1-1. The contours of the building are visible in the graphs because the initial level height of the selected floor, pictured with the black-dashed line, is the local axis for the floor deformation and rotation of the associated floor is measured. The continuous blue lines in Figure 5.5a represent the floor deformations for the selected floor. The distance between the black-dashed line and the blue line represents the floor deformation relative to the initial position. The floor deformation is multiplied by a factor to show the shape of the deformation in the graph clearly. In Figure 5.5b the continuous blue line represents the rotation of the floor. The limits for the floor rotation are shown with the dashed red lines above and under the line of the initial floor position (black-dashed line). If the blue line of the floor rotation stays between the red lines, it is not exceeding the limit for floor rotation. If the blue line is exceeding one of the red lines the floor rotation is exceeding the rotation limit.

The floor rotation is the derivative of the vertical floor deformation and for verification of the outcome the vertical floor deformations are analyzed because the vertical deformation is easier to understand than the rotation.



(a) Vertical deformation

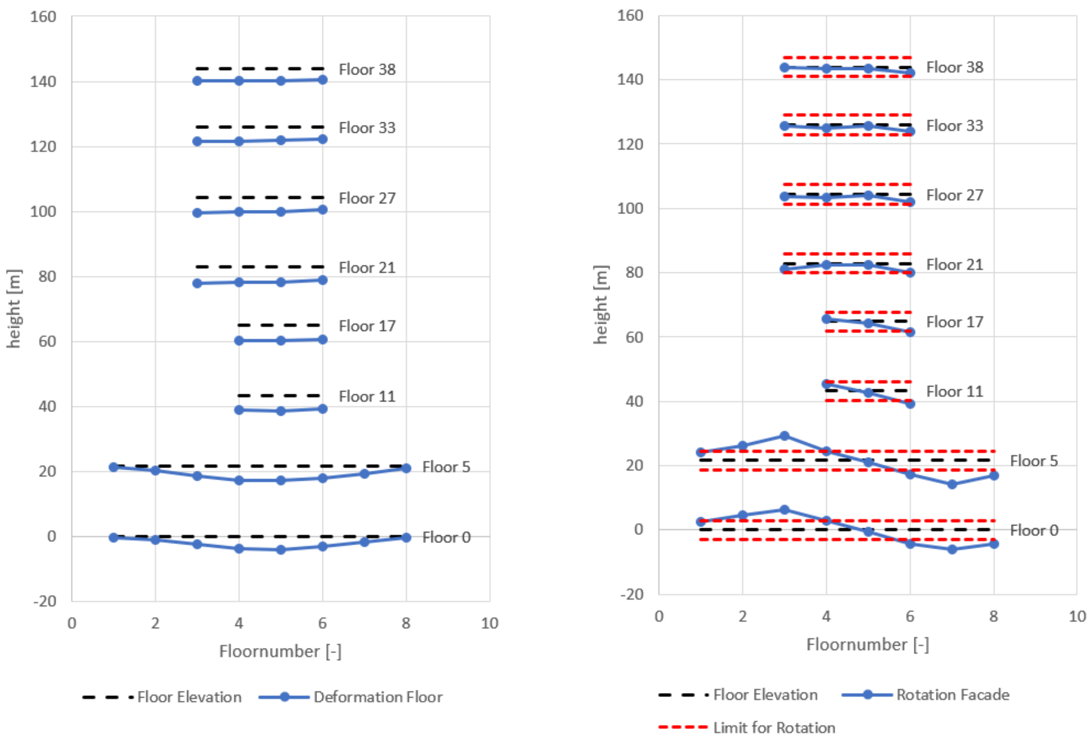
(b) Rotation

Figure 5.5: Final floor deformation in cross-section 1-1 of the concrete alternative in Rotterdam

The graph in Figure 5.5a shows the vertical deformation of the floors. In the graph can be seen that maximal vertical deformation is happening under the low-rise and the high-rise part of the building. The floor numbers representing the part of the building under the low-rise and the high-rise part are 4, 5, and 6 (see Figure 5.1). Observing the deformation of this floor numbers, all the levels shows that the floor element of floor number 4 is deforming slightly more compared to floor number 5 and 6. This is as expected because the load of the cantilevering part of the building is supported by the columns on the left side of floor number 4. The vertical deformation of the cantilevering part of the building is deforming more compared to the left side of the high-rise part due to geometry of the building, the center of gravity of the high-rise part is shifted to the left due to the cantilever, therefore the deformation on this part are bigger. The low-rise part of the building is not as heavy as the high-rise part of the building, therefore the settlements are smaller. The part of the low-rise part connected to the high-rise part is 'dragged-down' by the weight of the high-rise part, and the outer parts of the low-rise are closer to the initial position. Figure 5.5b is now better understandable because of the relation with the deformation. The biggest problems occur at the bottom of the building and especially in the low-rise part. The connection between the low-rise and the high-rise part is where the floor rotation is exceeding the limit the most. In the high-rise part of the building is the limit for floor rotation almost not exceeded.

Final deformation of the facades

Figure 5.6 shows the deformation and rotation of the facade support deformation for the final settlement. Comparing this graph to the deformations and rotation of the floors in Figure 5.5 shows that the deformations and rotation are smaller for the facade supports. The reason for this decrease is the distance between the facade supports and the stability core. The stability core will settle the most compared to a smaller deformation of the perimeter of the building.



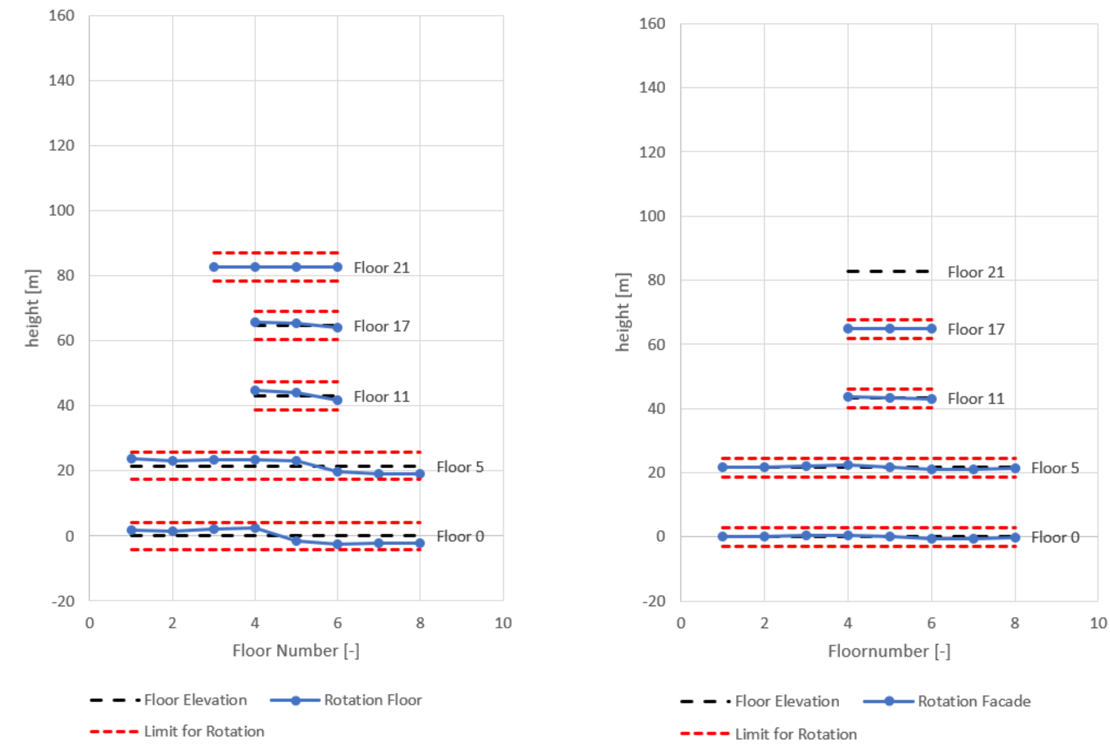
(a) Vertical deformation (b) Rotation of the facade supports

Figure 5.6: Final facade deformation in cross-section 2-2 of the concrete alternative in Rotterdam

The results of the facade rotation also shows that the limit of facade rotation is only exceeded at the lower floors.

5.1.2. Deformations During Construction

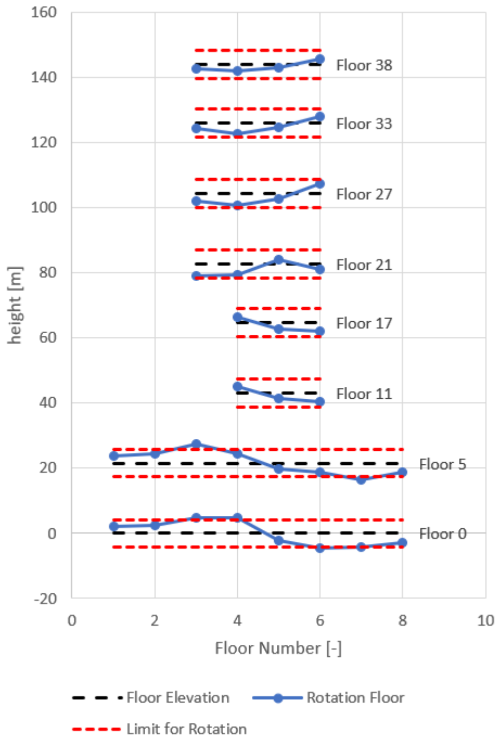
As explained in Section 2.4, the construction sequence needs to be considered to determine the deformations of the floors and the facade supports. In Figure 5.5 and Figure 5.6 the deformations are shown for the final settlement, with the construction sequence taken into account. The construction sequence decreases the problem, therefore the influence of the construction sequence is explained more in-depth. In the following figures the floor rotation and the facade support rotation are shown for construction stages five and construction stage 9. As explained in Section 2.4, the facade elements are attached consecutively to the primary structure but with 6 stories in between. For this calculation the construction stages as shown in Figure 5.2 are used. The floor rotation is measured from the moment the primary structure is constructed. In the graph, the initial floor height (black-dashed line) is shown from the moment the primary structure is constructed to that level. The blue line appears at the same level, which means that the floor deformation is measured. The floors are constructed horizontally, therefore the floor rotation is zero (equal to the black line). For the facade support rotation this is different, the facade elements are constructed six stories below the primary structure. In the graphs for rotation of facade supports this can be seen because the initial story level (black dashed line) is shown six stories above the level where the rotation line of the facade supports are shown. The rotation line starts also horizontally, because the facade elements are attached horizontally.



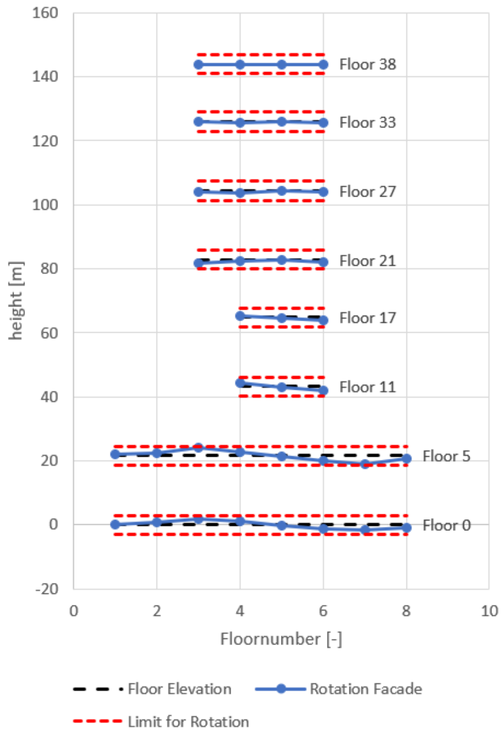
(a) Floors (b) Facade

Figure 5.7: Deformation in construction stage 5 in cross-section 2-2 of the concrete alternative in Rotterdam

The results in Figure 5.7 show that the floor and facade start to deform already during construction. In this figure can be seen that the highest rotation line is horizontal for both the floor and facade rotation, however the lower levels are already deformed in this construction stage. The deformations are not exceeding the corresponding limits so far.



(a) Floor



(b) Facade

Figure 5.8: Deformation in construction stage 9 in cross-section 2-2 of the concrete alternative in Rotterdam

The results in Figure 5.8 show the rotations of construction stage 9. In this construction stage the primary structure and the attachment of the facades are constructed. Therefore, the total weight of the structure is applied on the soil. The results for the rotation shows that the facade rotation is not exceeding the corresponding limit, only the floor rotation is slightly exceeding the limit at the two bottom levels of the structure.

5.1.3. Sensitivity Analysis of Permeability of the Soil

In this subsection the results of a sensitivity analysis of the permeability of the clay-layers are explained. A sensitivity analysis of this soil property provides a better understanding of the influence on the deformations in the building and whether it is worth the money and effort in determining the specific value for these properties.

The permeability of the soil is linked to the time-dependent settlement behavior of the soil. The settlement can be divided into the amount of the settlement which is happening during the construction sequence and after the construction sequence.

To show the influence of the permeability of the soil, the initial parameter of the consolidation coefficient are multiplied by 100, by 10 and divided by 10. The resulting values are not scientifically determined but are only used to show the influence. The initial soil properties are shown in Table 4.4 and only the vertical consolidation coefficient C_v parameter is changed in this sensitivity analysis. Table 5.1 shows the permeability coefficient values used for the sensitivity analysis.

Table 5.1: Variable values of for the consolidation coefficient for the sensitivity analysis

Type of Soil	Vertical Consolidation Coefficient C_v Option 1 [m^2/s]	Option 2 [m^2/s]	Option 3 (Initial) [m^2/s]	Option 4 [m^2/s]
Gravel	drained	drained	drained	drained
Sand	drained	drained	drained	drained
Sandy Clay	10^{-4}	10^{-5}	10^{-6}	10^{-7}
Clay	10^{-6}	10^{-7}	10^{-8}	10^{-9}
Peat	10^{-2}	10^{-3}	10^{-4}	10^{-5}

With the program D-Settlement the deformations of the soil are calculated for the location of Rotterdam. Figure 5.9 shows the four settlement curves for the four permeability options from Table 5.1. The curves show a difference in time-dependent behavior. In the figure can be seen that the four curves have a different settlement at t = 10000 days. For the settlement line of the consolidation coefficient of initial * 100 it can be observed that the line is decreasing a lot in the beginning and after at t = 5000 days, the settlement line becomes constant. For the other options the settlement will go on long after t = 10000 days.

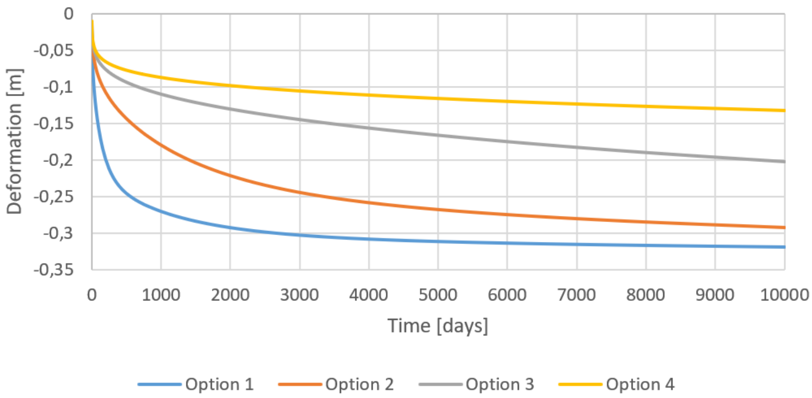


Figure 5.9: Deformation of the soil over time for the 4 options in Rotterdam

Figure 5.9 shows the settlement values for t = 1000 days is varying due to the difference in permeability. When the total consolidation is occurred the 4 lines will end with the same settlement. The total consolidation time can be calculated according to the theory of Terzaghi with the following formula:

$$t_{final} = \frac{H_{dr}^2}{C_v} \tag{5.1}$$

In Formula 5.1 is t_{final} the total consolidation time, H_{dr} is the drainage length and C_v is the consolidation coefficient. The thickest layer of clay can be found below Rotterdam, a layer with a thickness of 31 m, because the clay layer is in between drained layers (sand layers) the drainage length in half of the thickness of the clay layer, so H_{dr} = 15.5 m, which significantly decreases the consolidation time.

This analysis proves that it is essential to determine the consolidation coefficient for the specific building site because it has a significant influence on the settlement behavior.

5.2. Possible Mitigating Measures

In this section the possible mitigating measures are described to reduce the exceeding deformations. The following mitigating measures are considered for this research:

1. Change the construction sequence
2. Increase the length of the foundation piles
3. Application of a camber in the structure
4. Increase stiffness of the foundation slab
5. Increase the stiffness of the superstructure
6. Application of the jack-down system

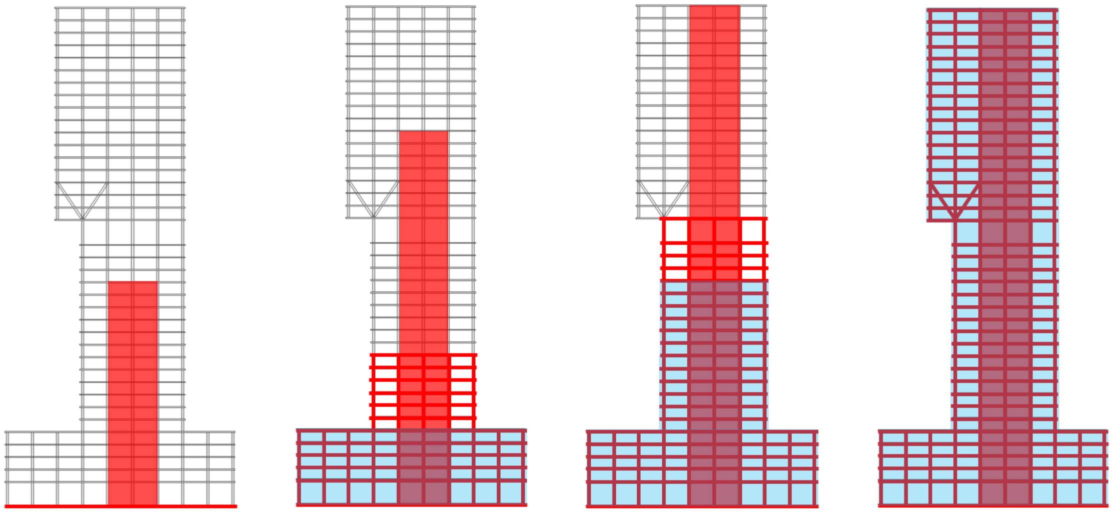
In the following subsections these mitigating measures are explained. In this research, the mitigating measures are considered to be used solely per mitigating measure in the structural design. In reality the problem can be mitigated with combinations of these measures.

5.2.1. Change the Construction Sequence

If during the design of the building problems due to compressible soil are discovered, changing the construction sequence can be considered as a mitigating measure. The construction sequence can be adjusted in multiple ways. For changing the construction sequence, it is crucial that the structural engineer knows when and where the problems occur in the construction sequence. For this research, this mitigating measure is explained by adjusting the construction sequence to construct the stability core in advance.

In the initial construction sequence is assumed that floors of the primary structure are constructed to the same level as the stability core is constructed. The weight of the stability core is high compared to the support reaction of the columns. Therefore the deformation of the soil is induced by the weight of the structural walls of the core. Due to the weight of the stability core, the soil immediately begin to deform during construction. When observing the deformation of the soil over time in Figure 4.21, it can be seen that after 600 days the core is settled around 0.10 m, compared to the total deformation this is already a big part of the total settlement. Adjusting the construction sequence to create a delay between construction of the stability core and the rest of the primary structure can reduce the deformations of the floors.

Figure 5.10 shows the adjusted construction sequence. When this construction sequence is compared to the initial construction sequence (Figure 5.2), the difference between the construction sequence is clear. The initial construction sequence construct the floors and the core simultaneously, which results in that the floors will follow the total settlement of the core. In the adjusted construction sequence the floors are attached later. Therefore a part of the deformation has already occurred before the floors are attached, which results in a smaller deformation experienced by the floors.



(a) Adjusted construction stage 1

(b) Adjusted construction stage 2

(c) Adjusted construction stage 3

(d) Adjusted construction stage 4

Figure 5.10: Adjusted construction sequence where the stability core is constructed in advance

This mitigating measure has impact on the construction time of the building. To create an gap between constructing the stability core and the primary structure of the floors, the construction of the floors will be delayed resulting in an increase of construction time. The longer the construction of the floors is postponed, the smaller the rotation of these floors is in the final state. It depends on the properties of the soil what the best delay time is. If the building is build on soil with high permeability, the settlements are done relative quickly. Therefore the delay does not have to be too long to decrease the rotation of the floors. Then changing the construction time is a considerable mitigating measure. When the permeability of the soil is low the settlement takes years, than the construction of the floors has to be delayed with years in order to be an efficient mitigating measure. So, it depends on the permeability of the soil if this mitigating measure can be considered as an option.

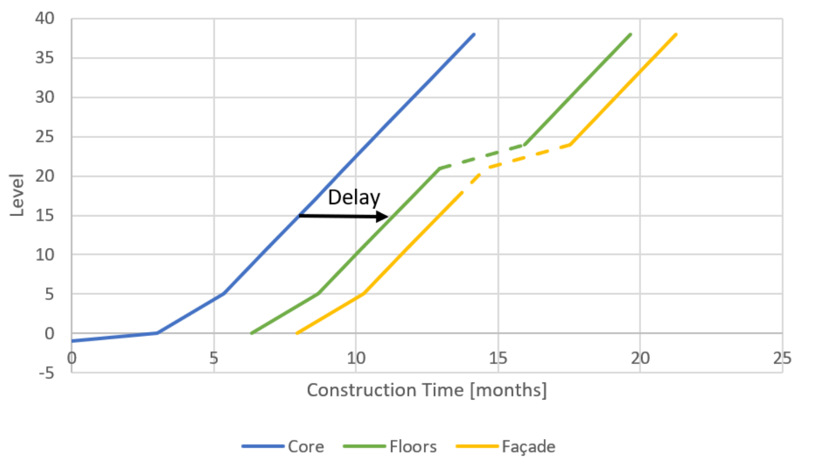


Figure 5.11: Height of the structure over time when the stability core is constructed in advance

5.2.2. Increase Length of the Foundation Piles

Increasing the length of the foundation piles can also be applied as mitigating measures. This mitigating measure is the first solution that comes to mind by the structural engineer when problems occur due to settlement. Figure 5.12 shows the mitigating measure of increasing the length of the piles. In the left figure the foundation piles are founded in the first sand layer. In this situation, the building is founded on top of the compressible clay-layer. Increasing the foundation piles is shown in the right figure. In this figure can be seen that the foundation piles are founded on the second sand-layer. The foundation of the piles in the second sand-layer will reduce the deformation of the compressible layer to zero because this layer is not loaded anymore. To reach the second sand layer, the piles have to be vibrated/screwed through the first sand-layer and through the compressible clay-layer. This mitigating measure is only effective when the piles are founded under the compressible clay-layer.

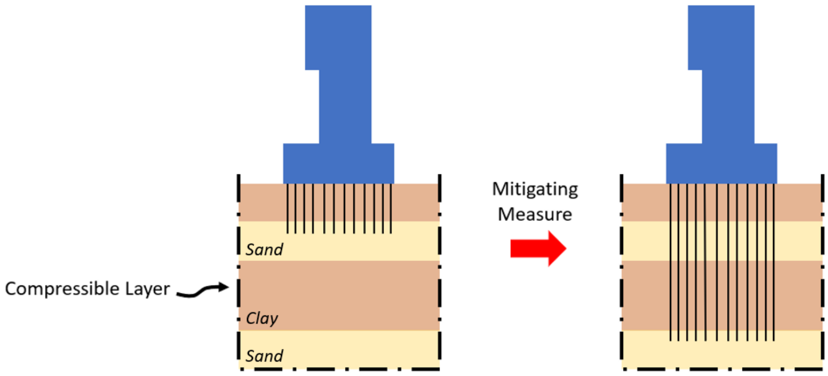


Figure 5.12: Schematic drawing of increasing the length of the foundation piles to the second sand-layer

The location of Amsterdam and Rotterdam have both to deal with a compressible clay layer underneath the foundation layer, as explained in Section 4.1. For Amsterdam, the compressible layer is situated at a depth of -30 m to -64 m under ground-level. In the initial structural design of the building, the foundation piles reach a depth of -23 m to reach the first sand-layer. When the mitigating measure is applied at the location of Amsterdam, the piles must reach a minimum depth of -64 m as shown in Figure 5.13. This requires a new length of almost three times the initial length of the foundation piles. For Rotterdam, the compressible layer is much thicker than the layer in Amsterdam. The clay layer reaches from -38 m depth to a depth of -97 m with small sand-layers in between. Increasing the length of the foundation piles is only useful when the foundation piles are founded in the sand-layer situated at a depth of -97 m under ground-level as shown in Figure 5.13. This depth is an immense depth for the foundation piles and is therefore not possible with the conventional pile types and equipment.

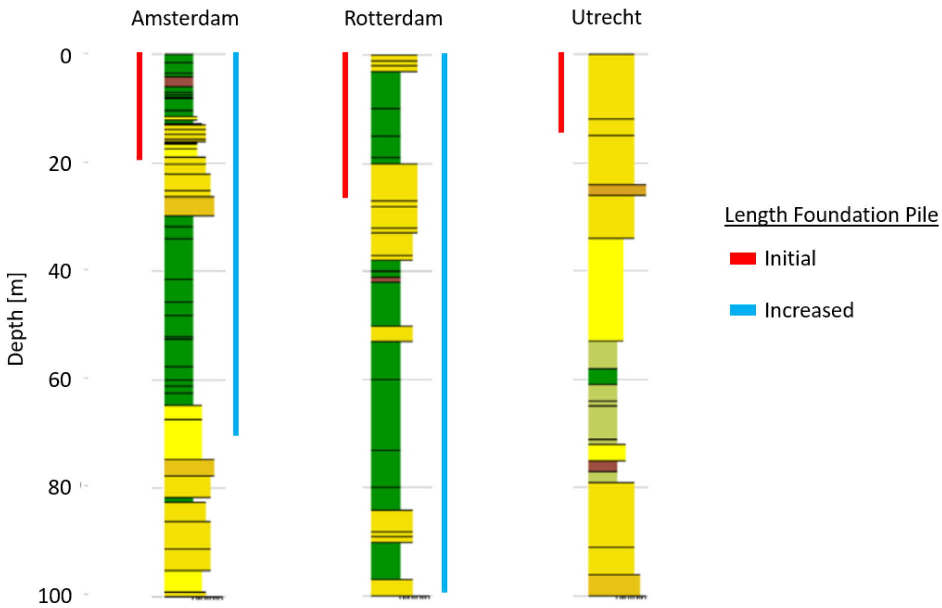


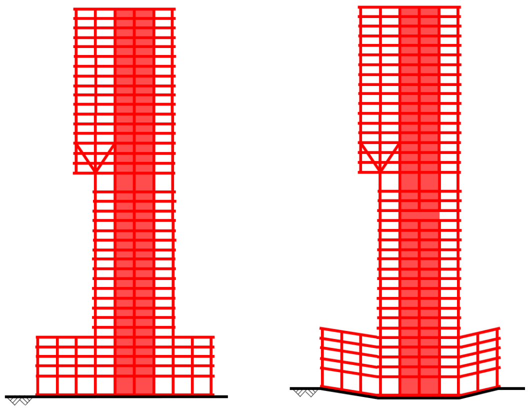
Figure 5.13: Schematic drawing to show the additional length of the foundation piles

When the length of the foundation piles is increased the deformations of the soil under the foundation piles will significantly decrease. However, due to the length of the foundation piles, the piles itself are now more susceptible to deformations.

To emphasize, the applicability of this mitigating measure depends on the structure of the soil. When the sand layer under the compressible layer is reachable, this mitigating measure can be applied to overcome the problem of deformation of the compressible soil and therefore, it will reduce the deformation in the building effectively. Nevertheless, this is an expansive measures.

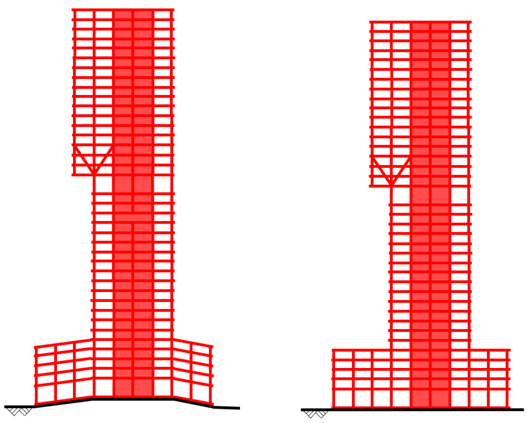
5.2.3. Application of Camber in the Structure

The next mitigating measure is the application of a camber in the structure. A camber in the structure can be used to counteract the deformation in the structure. In this way the final deformation can be reduced. When the building is designed with initial upward bending, this can reduce the final inclinations of the floors. Figure 5.14 shows the final deformation without a camber. The mitigating measure of applying an initial deformation in a building is shown in Figure 5.15. This figure shows that the final inclinations of the floors can be reduced by designing an initial deformed position. For the application of this mitigating measure, the structural behavior must be understood correctly. This mitigating measure is only effective when the initial position is precisely the opposite of the deformation happening in the structure.



(a) Initial position (b) Deformed position

Figure 5.14: The deformation of the structure when the building is constructed without camber



(a) Initial position (b) Deformed position

Figure 5.15: The deformation of the structure when the building is constructed with camber

Limitation

An important note to this mitigating measure is that this mitigating measure is not reducing the total deformation experienced by the structure and facade elements. The total deformation is equal to the design without mitigating measures. However, the final deflections and inclinations of the floors can be limited. In Section 2.3 the limits for relative rotations to prevent architectural and structural damage are explained. A camber in the structure does not change the amount of relative rotation and therefore it does not reduce the architectural damage and the structural damage of the structure. It can be concluded that these limitations for relative rotations are the limit for when a camber is applicable.

In Section 4.4, the results of the relative rotation of the foundation are shown for the structures designed in this research. When the results of the relative rotation of the concrete alternative are observed, it is clear that the building at the location of Rotterdam will experience architectural damage based on this limitation. Only the final floor rotations can be kept between the limits. Therefore this mitigating measure is applicable for the alternatives in Utrecht to reduce the final deformations, because the relative rotation is not exceeding the limits, as shown in Figure 4.22.

5.2.4. Increase Stiffness of the Foundation Slab

Increasing the stiffness of the foundation floor can be appkied in the structural design to reduce the differential deformations in the structure. When the stiffness of the foundation floor increases, loads of the structure distributes more evenly over the soil.

Increasing the stiffness of the foundation plate can be achieved with the following ways:

1. Increase the thickness of the foundation slab
2. Increase the Strength/E-modulus of the concrete

The stiffness of the foundation can be increased in multiple ways. In this research only the influence of the thickness is considered. By increasing the thickness of the foundation slab, the bending moment and the shear resistance of the foundation slab will increase. Due to the geometry of the building, the soil has the tendency to settle unequally with the stability core as the maximum. This settlement subjects the foundation slab to enormous internal forces. By increasing the thickness of the foundation plate, the moment capacity and the shear resistance will increase, and therefore the foundation slab will be stiffer.

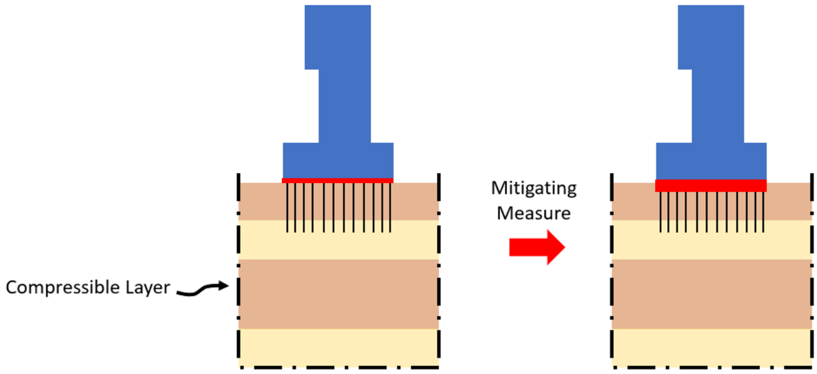


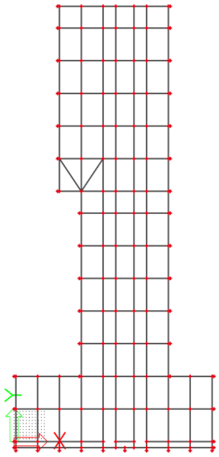
Figure 5.16: Schematic drawing of increasing the thickness of the foundation slab

To provide an indication of the efficiency of this measure, the deformation of the foundation slab calculated for different thicknesses of the foundation slab. This calculation is done with in a 2D model, which is explained in Appendix D. The results of the settlement calculation are shown in Figure 5.17. This graph shows that increasing the height of the foundation slab results in a decrease of differential deformation. It is the question if the amount of material needed is worth the amount of less deformation.

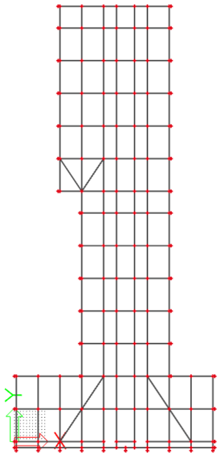
Figure 5.17: Graph of the deformation of the foundation slab for different thicknesses of the foundation slab

5.2.5. Increase Stiffness of the Superstructure

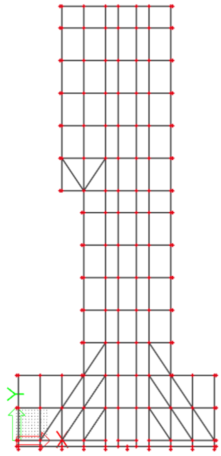
The stiffness of the superstructure can be increased as a mitigating measure. When the stiffness of the superstructure is increased, the weight of the core is distributed over a bigger area, just like increasing the thickness of the foundation slab. In Chapter 3 is the structure designed with a stability core and a skeleton structure around it. This skeleton structure has not much stiffness in vertical direction, therefore it is susceptible for differential deformations. A tube-in-tube structure is a type of stability system which is already more stiff compared to the system used in this research. To increase the stiffness of the superstructure diagonals are add in the structural system. There are more possibilities to increase the superstructure, but for this research, the diagonals are explained. In Figure 5.18 the considered options for the locations of the diagonals are shown.



(a) Initial design



(b) Option 1: with one diagonal on both sides



(c) Option 2: with 3 diagonals on both sides

Figure 5.18: Schematic drawing of adding the diagonals to the structural design

Due to the time-dependent behavior of the settlement, the internal forces in the diagonals can increase significantly over time. After constructing the primary structure, the settlements of the soil will continue. The stability core tend to settle more compared to the outside parts of the building. The diagonals will distribute the weight of the core to the outside of the building, therefore the internal forces can increase a lot.

To get an indication of the efficiency of this measures, the deformation of the foundation slab is calculated in a 2D model for the options shown in Figure 5.18. The 2D model is explained in Appendix D. Figure 5.19 shows the deformation of the foundation slab for the three options. The graph shows a decrease in settlement when more diagonals are placed in the structural design. The graph shows clearly that the load of the concrete core is distribute over a bigger area due to the diagonals. The more diagonals, the better the load is spread and result in a lower differential deformation.

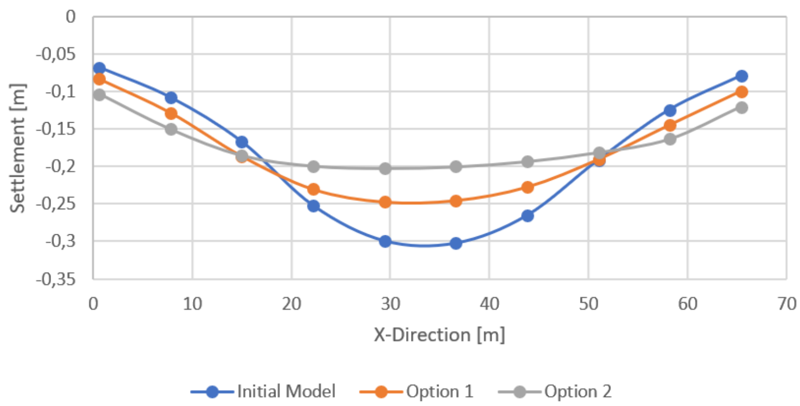


Figure 5.19: Graph of the deformation of the foundation slab for the different options of adding the diagonals

Regarding the settlement line, this mitigating measures seems like a possible option, but the internal forces in the diagonals will become huge. For the calculation of the internal forces in the diagonals it is important to take the SSI interaction into account. To find the internal force in the diagonal it is important that the settlement line calculated with the SCIA model is equal to the settlement line of the SCIA model as explained in Subsection 4.3.3.

To provide an indication of the internal forces in the diagonals, the support reaction of the core are shown in Figure 5.20. The graph shows that the support reaction are decreasing for the options with the diagonals. The diagonals distribute the weight of the core, so the difference between the weight of the core in the initial design and the options is related to the internal forces in the diagonals.

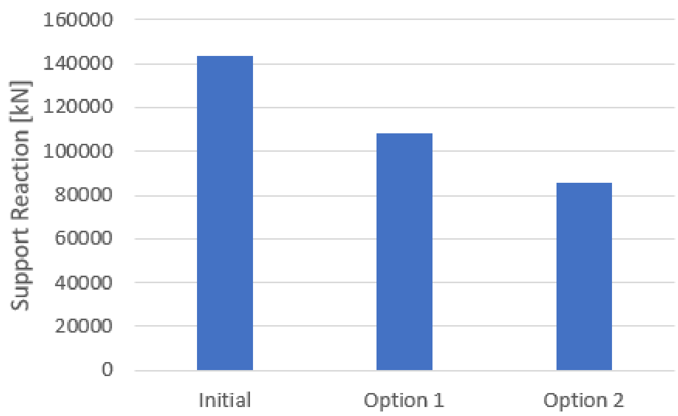
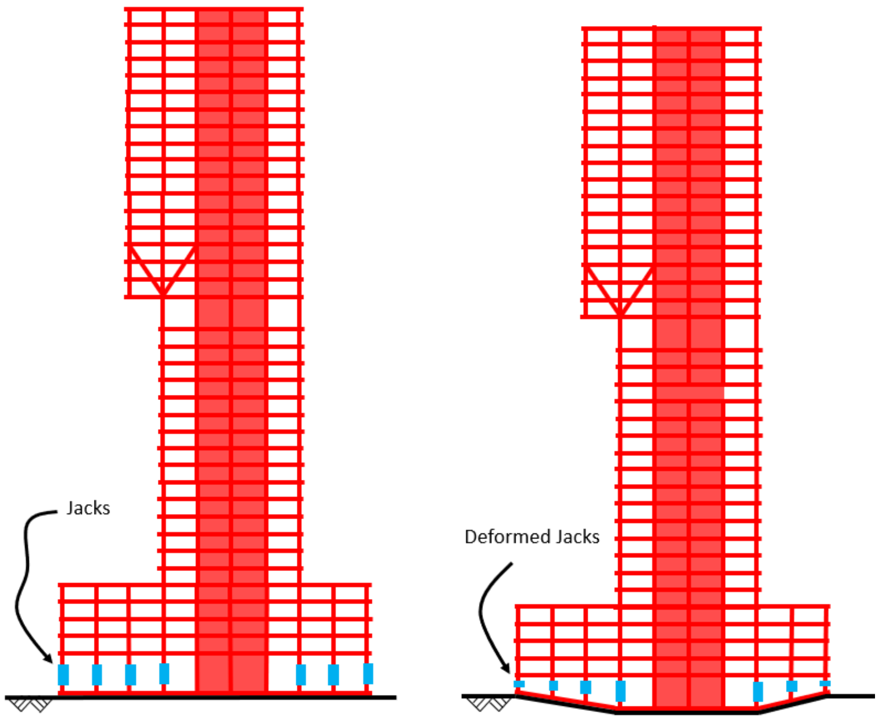


Figure 5.20: Comparison of the reaction force under

5.2.6. Apply Jack-Down System

The last considered mitigating measures of this research is the jack-down system. The jack-down system can be applied in the columns, as shown in Figure 5.21. This figure shows the initial and deformed position shown. As can be observed in the figure, the upper floors (the floors above the jack-down system) remain straight even in the deformed position. The jack-down system will gradually shorten its length in order to follow the settlement of the core. The core starts to settle during construction, therefore the jack-down system will already be operative during the construction of the building. The advantage of the jack-down system is that the upper floors will not experience the deformation of the soil. The total upper-structure will move a little bit downward. Only the foundation floor experiences differential deformation.

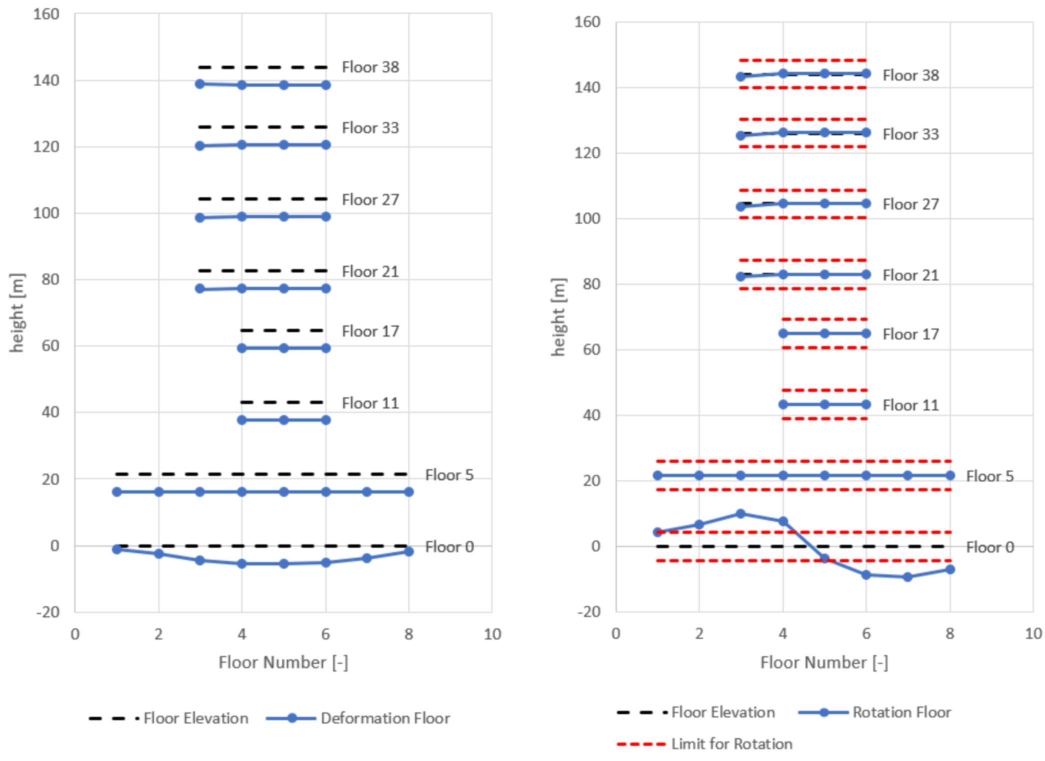


(a) Initial position (b) deformed position

Figure 5.21: Schematic drawing of the jack-down system in the structure

Monitoring of the movement of the structure is necessary for this mitigating measure. During construction and the lifetime of the building the soil will deform, and therefore, the foundation slab will deform accordingly. With measuring the structural movement the amount of jacking can be determined. Due to the application of the jack-down system, the redistribution of the internal forces due to the settlement of the soil can be minimized.

Figure shows 5.22 the vertical deformation and the floor rotation.



(a) Vertical deformation (b) Rotation

Figure 5.22: Final floor deformation for the structural system with jack-down system in cross-section 1-1 of the concrete alternative in Rotterdam

The results in Figure 5.22 show that only the lower floor will experience the deformation. The floors above the jack-down system will deform uniform, accordingly to the deformation of the core.

6

Multi-Criteria Analysis

In this chapter, the mitigating measures are assessed by a multi-criteria analysis. In the first section, the aspects of the multi-criteria analysis are explained, in the second section the multi-criteria analysis is explained.

6.1. Aspects of the Multi-Criteria Analysis

The multi-criteria analysis of the mitigating measures provides an overview of the advantages and the disadvantages of the different measures. Table 6.1 shows the multi-criteria analysis aspects and the corresponding design considerations. The multi-criteria analysis aspects represent the primary considerations of the structural engineer when designing a construction.

Table 6.1: List of the multi-criteria aspects and the corresponding design considerations

Design Considerations	Multi-Criteria Aspects
Cost	Material Use
Quality	Functionality of the Building Effect Surroundings
Time	Construction Time
Risk	Conventionality Adaptability

As can be seen in Table 6.1, the aspects of the multi-criteria analysis have corresponding design considerations as cost, quality, time, and risk. These main design considerations are present in every project. An ideal project is constructed within budget, with the highest quality and lowest risk, in the quickest construction time possible. In reality, the quality, the construction time and the risk will influences the total cost of the project. Concerning to the multi-criteria analysis aspects, it is good to see the relation between the more measurable aspects of the multi-criteria analysis and the global design considerations.

Material use

For every project is the amount of materials used an essential aspect. The amount of material used has a direct relation with the total cost of the project and the sustainability of the entire building. The increasing technology and measurements can predict the structural behavior more precisely together with the understanding of the structural behavior, it can result in an economical design. A decrease in material use has a positive effect on the total cost of the project and the sustainability of the building.

For this aspect, it is favorable when the amount of material used for the mitigating measure is as small as possible.

Functionality of the building

This aspect is considering the functionality of the building. The building is designed by an architect to full-fill a function in the best way possible. Unfortunately, the architectural design cannot be simplified by a formula or measured according to a value [22]. The building used in this research is designed to accommodate predefined functions. The public function of the building is situated at the bottom two levels of the building. The public space is intended to function as atria and connective space. For these levels, the structure present must be as small as possible. The higher floors are occupied by office spaces, for the office floors the areas of the floors are divided into smaller spaces. It is favorable for office floors that there is a possibility to rearrange the floors according to the needs of the users, so for these floors, it is negative when the structure is restricting this freedom of rearranging the space.

So, for this aspect, it is favorable that the least structure is present. The initial design of the structure consists only of the vertical and horizontal structural elements, so there is enough freedom to occupy the space as wanted. Any other obstruction of this freedom is regarded as negatively for the aspect of the functionality of the building.

Effect surroundings

Assumed is that the building of this research will be constructed in a city center, which means that the structure will be surrounded by already constructed buildings. The surrounding buildings, especially when the building are historic and constructed with masonry, can be damaged by the imposed deformation due the weight of the new building. Previously in this research, only the settlement of the area of the building is considered, but the settlements also occur outside of the perimeter of the building due to the continuity of the soil. In the high-rise vision of Amsterdam, this aspect is explicitly written down [9].

For this aspect it is favorable to keep the surrounding settlement as small as possible. It is the responsibility of the new building to reduce the settlements around the building in order to exclude damage to the surrounding buildings.

Construction time

The construction time of a building is essential for every project. The construction time relates to the price of the total project and whether the building starts making revenues for the investor. Usually, the investor determines the available construction period based on an estimation, and then the contractor prepares its quotation based on the available construction period [12]. So, the price for the construction time is usually based on the interaction between the investor and the contractor. For this research, the aspect construction time regards the constructability of the mitigating measure and eventually delay in construction of the building.

For this aspect it is favorable that the construction time is as short as possible. Any delay or extra intended construction time will cost money.

Conventionality

This aspect of the multi-criteria analysis is considering if the mitigating measure is a conventional system applied in buildings. When the mitigating measure is not conventional, specialism is needed to construct the mitigating measures correctly. In this research, specialism is considered when for construction of the building, specialized knowledge or machines are needed compared to a regular constructed building.

For this aspect it is favorable that the mitigating is conventional so that not much specialism is needed during construction.

Adaptability

Every project has to deal with uncertainties. In this research, the uncertainties of the soil are of interest. The structural behavior and the settlement of the soil are estimated based on the known properties of the soil, but the results of these calculations are susceptible to uncertainties. Due to these uncertainties, the structural behavior and soil can deviate from the calculations. The structural engineer has to take these uncertainties into account. As explained earlier, the uncertainties of the settlement of the soil are equal to 30%. The adaptability of the mitigating measures is needed when the uncertainties in the soil behavior are high.

For this aspect it is favorable that the mitigating measure is capable of being modified to respond to uncertainties.

6.2. Ranking of the Mitigating Measures

For the multi-criteria analysis, the mitigating measures are ranked per aspect based on the gained knowledge on the soil-structure interaction, the structural behavior, and the investigation of how the mitigating measures are working. An overview of the ranking is shown in Table 6.2 where the aspects of the multi-criteria analysis are mentioned in the rows, and the mitigating measures are mentions in the columns. The results of the ranking are simplified illustrated, where + is a favorable ranking on the concerning aspect, ≈ is ranked in-between for the concerning aspect, and − is ranked negatively on the concerning aspect.

Table 6.2: Overview of the ranking of the mitigating measures fo the multi-criteria aspects

Design Considerations	Multi-Criteria Aspects	Change Building Sequence	Increase Pile Length	Apply Camber	Increase Stiffness Foundation Slab	Increase Stiffness Superstructure	Apply Jack-Down System
Cost	Material Use	+	-	+	-	≈	+
Quality	Functionality	+	+	+	≈	-	≈
	Effect Surroundings	≈	+	≈	-	-	≈
Time	Construction Time	-	≈	+	≈	+	+
Risk	Conventionality	+	-	+	≈	≈	-
	Adaptability	≈	-	≈	-	≈	+

In the overview can be seen that for example, the mitigating measure of applying a camber in the structure is favorable regarding the aspects of material use, the functionality of the building, the construction time, and is a conventional mitigating measure. The aspects of affecting the adjacent buildings and adaptability are neutral. The other mitigating measures can be read in this way as well.

The overview can be read in the other way around as well, for example, the effect on the surroundings is considered. The overview shows that the mitigating measure of increasing the pile length is favorable for this aspect. The mitigating measures with negative ranking are increasing the stiffness of the foundation slab or the superstructure, and the mitigating measures with a neutral ranking are changing the construction sequence, applying the camber and the jack-down system.

So, in this way the overview provides a global insight into the strength and weaknesses of the mitigating measures. In the following paragraphs, the ranking of the mitigating measures per aspect is explained.

6.2.1. Material Use

The mitigating measures are using different amounts of material to mitigate the risk of exceeding deformation limits. In Section 6.1 is explained that less material use is favorable for the costs and the sustainability of the project. Figure 6.1 shows the ranking of the mitigating measures for material use. In the ranking can be observed that the change of the building sequence, application of a camber, and application of the jack-down system are favorable, because these measures use less material compared to the other mitigating measures. Increasing the stiffness of the foundation slab and increasing the length of the foundation piles are negative because these measures require much more material. The mitigating measure of increasing the stiffness of the superstructure is in-between.

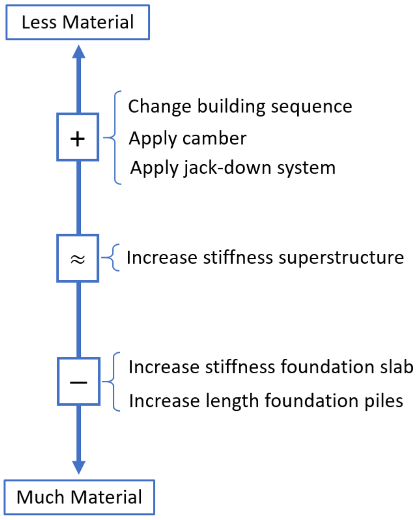


Figure 6.1: Ranking of the material use of the mitigating measures

In Section 5.2 is explained how the measures are reducing the deformations. For the material use the mitigating measures can be divided into two groups. The first group contains the mitigating measures which allows the deformation to happen, and the second group uses added material to resist the deformations from happening. The first group contains the mitigating measures of changing the building sequence, applying a camber or application of the jack-down system. In the second group of resisting the deformation, a distinction can be made between efficient use of the materials or inefficient use of materials. Increasing the stiffness of the superstructure with diagonals is an efficient use of materials, due to understanding of the structural behavior the diagonals can be placed at the locations where they are needed to distribute the load efficiently. When the thickness of the foundation slab is increased, a lot of material is needed to be effective. The thickness of the foundation slab needs to be increased significantly to be able to distribute the vertical load of the concrete core to the outside of the foundation plate. Additionally, to the inefficient shape of the foundation plate, the additional weight of the increased height of the foundation slab will result in extra settlements of the soil. As last, increasing the length of the foundation piles can require much material too, due to the depth of the second sand-layer.

6.2.2. Functionality of the Building

When looking at the aspect of the functionality of the building, it is favorable that the mitigating measure does not create any obstruction in the design. Figure 6.2 shows the ranking of the mitigating measures for this aspect. The ranking shows that the mitigating measures of increasing the length of the foundation pile, application of the camber, and changing the building sequence are favorable for the functionality of the building. The increase of the stiffness of the superstructure has the most influence on the design of the building. The jack-down system and increase of the stiffness of the foundation slab are in the middle for this aspect.

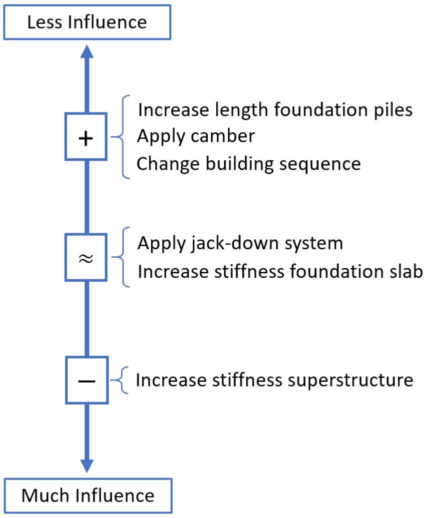


Figure 6.2: Ranking of influence on the functionality of the mitigating measures

The mitigating measures which are favorable for this aspect, are measures that do not need any significant adjustments in the structural system. Increasing of the foundation piles does not change the superstructure at all, the piles only need to be placed at a greater depth. Application of the camber and changes in the building sequence will not influence the functionality as well, but the structural design needs to be designed to allow the floors to be cambered or details need to be adjusted when a different construction sequence is used. So, no notable changes for the functionality of the building.

The jack-down down system, and increasing the thickness of the foundation slab have a minor influence on the structural design. The detail for the jack-down system needs to be applied in the structural system and must be accessible to adjust the length of the jack-down system when needed. The foundation slab can influence the functionality of the building. Increasing the thickness of the foundation slab can decrease the functionality but does not have to. In the preferable solution, the additional thickness of the foundation slab can be solved by a deeper excavation. However, when this excavation is not possible or too expensive, the additional thickness will replace valuable space like a basement level. Concluded, these mitigating measures can influence the functionality of the building.

The last mitigating measure of increasing the stiffness of the foundation floor will undoubtedly obstruct the functionality of the building. The diagonals influence the layout of the floor-plans. In Subsection 5.2.5, the diagonals are considered in the lower levels because at this location the diagonals are the most effective. For most of the buildings, the lower levels are desired as open as possible to function as an atrium or connective space. So for this mitigating measure, the architect has to make some sacrifices in the design of the building.

6.2.3. Effect Surroundings

The aspect of affecting the adjacent buildings is explained shortly in Section 6.1. Figure 6.3 shows the ranking of the mitigating measures on this aspect. The ranking shows that increasing the foundation piles is the favorable measure, increasing the stiffness of the superstructure and the foundation slab are the least favorable measures. The other three measures, changing the building sequence, application of the camber, and the jack-down system are in the middle on the ranking.

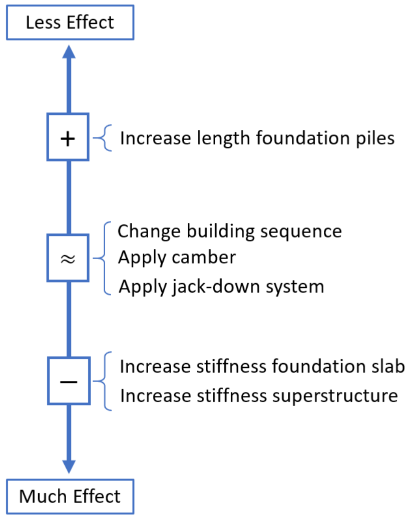
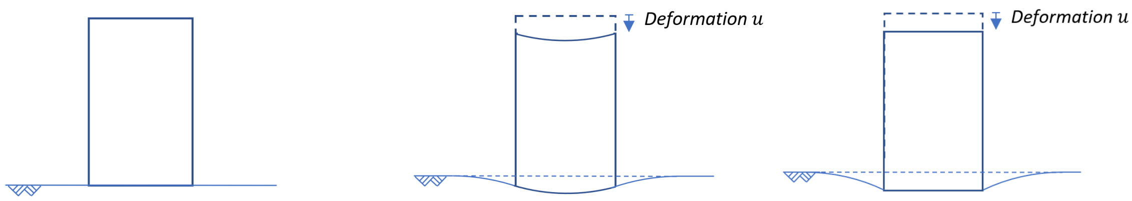


Figure 6.3: Ranking of the effect on the surroundings of the mitigating measures

The mitigating measures are divided in three groups. This distinction can be made based on how the mitigating measures decrease the deformations. Figure 6.4 shows the division, the first option is that the soil will not deform at all, the second option is when the soil deforms due to the weight and that the structure deforms accordingly and the third option is that the soil deforms uniform and that the superstructure will not deform. In these schematic drawings can be seen that the settlements due to the building are smaller in the option with deformed structure compared to the uniform settlement of the building.



(a) No deformation (b) Differential deformation (c) Uniform deformation

Figure 6.4: Schematic drawing to explain the options of deformation of the soil

For verification of the options, as mentioned above, a calculation is done with the program D-Settlement. In this program, the shape factor is the parameter that specifies the shape of the contact pressure [8]. Figure 6.4 shows that the building can deform, which means that the soil under the building deforms as a parabolic. The other variant shown in this figure is the uniform settlement, for this option the settlement under the building is uniform. In Figure 6.5, the results of the settlement calculation are shown for a distributed load with the shape factor assumed as 1 and as 0. The uniform distributed load is equal to the weight of the concrete alternative divided over the total area of the building. For the settlement lines of the shape factor 1 and 0 the surrounding settlements are the comparable. In figure the settlement line of the concrete alternative is also shown, as calculated in Subsection 4.3.3. For this settlement line, the distributed load under the stability core is much higher compared to the low-rise part of the building, resulting in a bigger settlement under the core of the building. Figure 6.5 shows the surrounding settlements are much smaller compared to the uniform distributed load with the shape-factors.

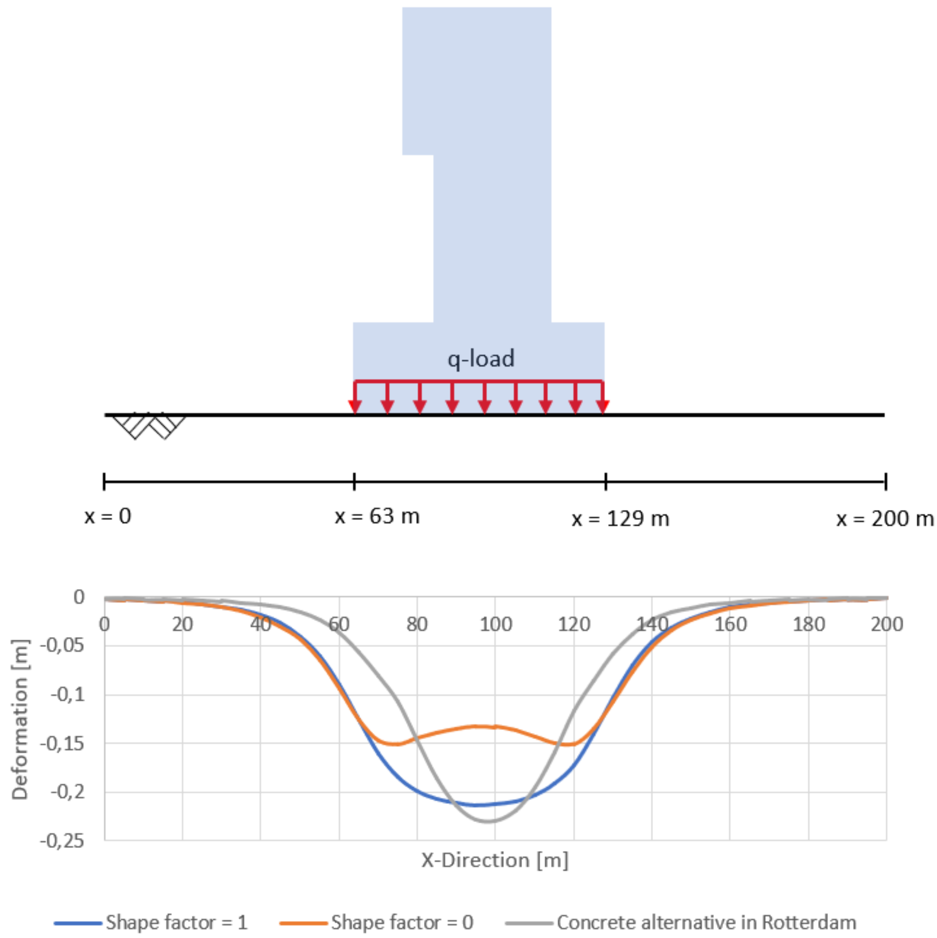


Figure 6.5: Results of the deformation of the soil for different load distribution to demonstrate the settlements of the building to the surroundings

In the CUR166 limits are provided for settlements under adjacent buildings [15]. These limits regard architectural and structural damage due to deformation of the soil, as explained in Section 2.3. In Appendix E the calculations of the relative rotation are shown for this building, which shows that only a (historic) masonry building in a range of 5-10 m around the new building can be damaged according to the limits of CUR166 for the considered building and soil structure in this research.

It depends on the mitigating measure of how big the settlements of the adjacent buildings are. The mitigating measures of increasing the stiffness of the superstructure and the foundation slab are designed that the building will settle uniformly according to Figure 6.4c. The mitigating measures of the camber, the jack-down system, and changing the construction sequence are designed to let the structure deform and will, therefore, settle according to Figure 6.4b. The advantage of the mitigating measure of increasing the foundation piles is that the settlement of the building will not happen since the weight of the building is not resting on the compressible soil, this also prevent settlement around the building, as shown in Figure 6.4a.

6.2.4. Construction Time

The ranking of the construction time is shown in Figure 6.6. In this figure can be observed that increasing the stiffness of the superstructure, application of the camber, and application of the jack-down system are ranked as favorable for the construction time. Changing the building sequence will increase the construction time and has, therefore, a negative effect on this aspect. In the middle of the ranking are the measures of increasing the stiffness of the foundation slab and the length of the foundation piles.

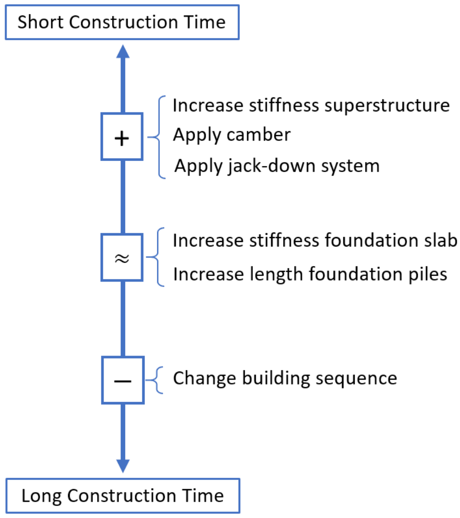


Figure 6.6: Ranking of the construction time of the mitigating measures

The mitigating measures with a favorable ranking will not shorten the construction time, and will not increase the construction time significantly. For increasing the stiffness of the superstructure a number of extra structural elements needs to be constructed, but compared to the initial number of structural elements this number is small. For the mitigating measures of applying the camber, the horizontal structural elements need to be constructed under an inclination, so the form-work needs to be constructed under inclination when the structure is constructed with in-situ concrete. For prefabricated elements this is done more easily. The jack-down system needs to be constructed in the structure. For these mitigating measures no significant changes in the construction time are needed.

For the mitigating measures of increasing the stiffness of the foundation slab or increasing the length of the foundation piles, the construction time can significantly increase. The foundation piles become much longer when they have to be drilled/vibrated through the first sand-layer and the compressible layer to reach the second sand-layer, as explained in Subsection 5.2.2. Increasing the thickness of the foundation slab can also increase the construction time. Increasing the thickness is only sufficient when the slab is much thicker. When the architectural design of the building can not change too much, this will result in a deeper excavation with all the corresponding complications and an increased the construction time.

The last mitigating measure is the change of the construction sequence. For this mitigating measure the construction time is increased on purpose. Due to the time-dependent behavior of the soil, it rewards when the construction sequence is delayed, as explained in Subsection 5.2.1.

6.2.5. Conventionality

The ranking of the conventionality of the mitigating measures is shown in Figure 6.7. In the ranking can be observed that the change in building sequence and the application of the camber are favorable for this aspect. Compared to a regular construction, these measures are commonly used. In the middle of the ranking are both the mitigating measures of increasing the stiffness of the structure. The mitigating measures of application of the jack-down system and increasing the length of the foundation piles are at the bottom because these measures are not commonly used and need special equipment or knowledge.

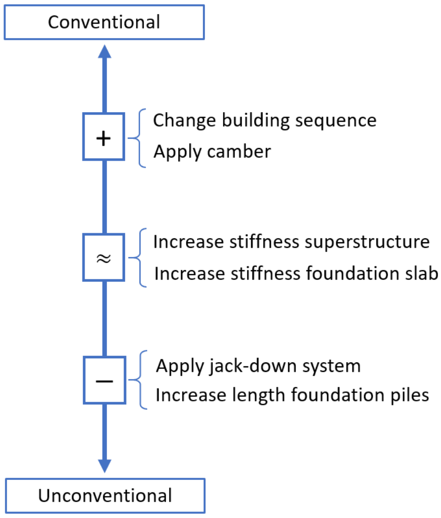


Figure 6.7: Ranking of the conventionality of the mitigating measures

The mitigating measures of changing the building sequence and application of the camber is equal to a regular construction. For the change in construction sequence the stability of the structure must be guaranteed.

For increasing the stiffness of the superstructure and the foundation slab, more specialism is needed during construction. When these measures are used for decreasing the deformations of the building, the measures are vital for the building. As explained in Section 5.2, the internal forces in these elements increase significantly due to the settlement. Therefore the detailing and the construction needs to be done carefully and the dimensions of these structural elements can become remarkably large.

The mitigating measures of application of the jack-down system and increasing the length of the foundation piles are not commonly used when looking at the already constructed buildings. The jack-down system, as applied in De Rotterdam and First Rotterdam is not common practice. The principle of the jack-down system is transparent and is efficient in reducing the deformations in the superstructure, but the measure is unconventional, so not many contractors have experience in applying them. For increasing the length of the foundation piles the length of these piles will increase significantly. Therefore many pile type systems are not applicable. As already explained, the piles need to be bored/vibrated through the first sand-layer and the compressible layer. The first sand-layer will resist this vibrating/screwing, so the machines have to provide enormous forces to put the foundation pile in the second sand-layer.

6.2.6. Adaptability

The aspect of adaptability is essential concerning the uncertainties in the behavior of the soil. The ranking for this aspect is shown in Figure 6.8. In the ranking, is the jack-down system marked as the favorable measure. In the middle of the ranking the mitigating measures of increasing the stiffness of the superstructure, changing the building sequence and application of the camber. The mitigating measures of increasing the stiffness of the foundation slab and increase the length of the foundation piles are placed below on the ranking.

The jack-down system is a favorable measure considering the aspect of the adaptability. With the jack-down system, the deformation can be accommodated during the construction and the lifetime of the building. During the construction and the life-span of the building, the movement of the foundation has to be monitored, and can be calculated with structural calculations if adjusting the jack-down system is necessary. When the settlement of the soil deviates from the calculated settlement, it will not result in problems because the jack-down system can anticipate on this difference.

The mitigating measures which are not favorable related to the uncertainties of the soil are increasing the stiffness of the foundation slab and increasing the length of the foundation piles. These two measures are situated below ground level. When the superstructure of the building is being constructed, nothing can be changed anymore to these measures.

The mitigating measures of increasing the stiffness of the superstructure, changing the building sequence, and application of the camber can be changed during construction of the building and are above ground level. For these mitigating measures, the uncertainties must be taken into account. For the diagonals, the internal forces can increase and the structural system loses its ductility due to the possibility of buckling. For the application of the camber, it is also essential to incorporate the uncertainties.

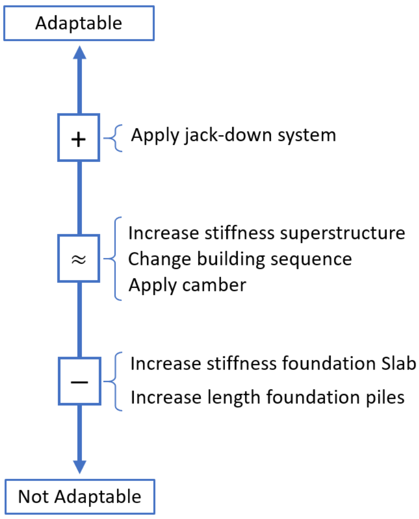


Figure 6.8: Ranking of the adaptability of the mitigating measures

7

Discussion, Conclusions and Recommendations

High-rise buildings in the Netherlands are reaching higher and higher. Some of the cities in the Netherlands are built where a compressible clay-layer is situated under the foundation layer. Most of the buildings in these cities are founded on the first sand-layer without problems of deformation of the compressible layer. Problems start to occur when the weight of the building increases, which is the case for high-rise buildings. Due to this weight the compressible layer starts to deform. The amount of the deformation is related to the weight of the building, this deformation can induce inclinations of the floors or damage the structural and non-structural elements, when no mitigating measures are applied in the structural design. The structural engineer has to design the structure in a way that these problems do not occur during construction and further lifetime of the building.

This research aims to get a better understanding of the structural behavior of high-rise buildings on compressible soil and to become acquainted with the possible mitigating measures which can be applied in the structural design to exclude unacceptable deformations during the lifespan of the building.

7.1. Discussion

In this research, a number of simplifications and assumptions are made, which can influence the result of this research. In this paragraph is elaborated on these simplifications and assumptions and is discussed what the consequences are for this research:

- In this research limitations for the vertical deflection of floors and the facade supporting elements are assumed. The size of the problems is related to these limits. In the Eurocode is stated: 'the limits for the serviceability criteria should be specified for each project and agreed with the client.' The problem of high-rise buildings on compressible soil can be solved by increasing the limits for serviceability are increased. In this research, this option is not taken into account. In reality, the discussion of increasing the limits is not that easy either.

  The interface between the structural design and the facade elements is no strict limit too. The manufacturer of the facade elements must guarantee that these elements are wind and waterproof during the total lifetime of the building. The manufacturer of these elements requires that the movement of the support is minimal. Therefore the limit for the facade deformation is set on 10 mm in the vertical direction for this research. However, this limit is also discussable and defines the size of the problem.

  Additional to the assumed limit, the limit for vertical deflection is rewritten to a rotation limit used to analyze where the problems occur. The rotation limit is determined according to the beam formula of a beam clamped on both sides, which is assumed. This is a simplification of the column-floor connection. The deformation of the floor can decrease or increase the rotation limit.

To conclude, the size of the problem depends on these assumed limits.

- In this research, the redistribution of the internal forces due to the deformations in the structure is neglected. In this research, the structure is designed for a building on rigid supports, the next step was to calculate the deformations of the soil. The deformation of the soil creates a different load distribution. In practice, the structure must be designed to be able to resist all the occurring internal forces. An iteration to include the redistribution of the forces is left out. By designing the building on rigid supports, it is possible that the structure is not strong enough, which can result in a lower stiffness of the structure. This lower stiffness has an influence on the deformations in the structure, and therefore the results can differ from reality.
- For the determination of the number of piles, a simplified method is used. This has resulted in an exaggeration of the number of piles, which results in a high stiffness and therefore a low deformation.
- For this research, a construction sequence of the building is assumed. As explained in the research, it is essential to consider the moment of attachment of the non-structural elements. Before the attachment of these elements, the structure will start to deform. The non-structural elements will not experience this part of the deformation. The building sequence, in combination with the time-dependent behavior of the soil, can have a significant influence on the magnitude of the problem and, therefore, on the results of this research.
- The soil structure and the properties of the soil are simplified. In reality, the soil structure can vary over the plot of the building, resulting in an unequal stiffness of the soil. This can increase the complexity of the problem. The properties of the soil layers are determined according to the characteristic values for soil properties stated in the NEN9997. In reality, the soil properties are location specific and can vary per location and over the depth. In this research, these simplifications are made to investigate the problem. Therefore, the results of this research are simplified and give only an indication of the total deformations of the substructure for the three locations.

7.2. Conclusions

The objective of this research was as stated in section 1.2:

To get a better understanding of the problem of a high-rise building on compressible soil, how the SSI is working and to investigate what the possible mitigating measures are to keep the deformation between the limits.

This research concludes that this objective is partly satisfied. However, the subject is too broad and is dependent on multiple factors like the geometry of the building, structural system, and the soil conditions. Therefore it is difficult to make conclusions which are applicable in general. Some conclusions can be made based on this research, these conclusions are explained in the following paragraphs.

7.2.1. Sub-Questions 1

The first sub-question is answered in Chapter 2.

What type of deformation is affecting the building due to deformation of the soil and are there additional factors influencing this deformation?

The forces of a building are supported by the soil, and the foundation is used to transfer the loads of the building to the soil. Deformations of the soil can result in the following types of deformations of the structure:

- **Differential deformation (sagging)**: Is unequal deformation over the plot of the building. The deformations are usually the largest where the core is located. This type of deformation induces inclination of the floors and causes damage to the structural and non-structural elements.
- **Uniform deformation**: Is equal deformation over the entire plot of the building. This type of deformation does not cause problems to the building itself, but it can result in damage to the adjacent building because of hogging of the soil.
- **Tilting**: Is rotating of the building resulting in a inclined building. This type of deformation can create significant second-order effects are therefore damage the structure and/or non-structural elements. As explained in the demarcation, this type of deformation is left out of scope.

Additional to deformation of the soil, column shortening causes vertical deformation in the building too. The difference in the shortening of the columns compared to the core can induce inclination of the floors and cause damage to structural and non-structural elements like the differential deformation of the soil. However, column shortening is likely to incline the floors in the opposite direction compared to inclinations due to differential deformation of the soil. Due to dimensions of the vertical structural elements, it can be expected that the columns will shorten more compared to the core. Therefore it can be said that the column shortening only decreases the problem in the building due to the deformations of the soil.

The construction sequence of the structure is affected by these deformations. Deformations of the soil and column shortening start to occur during construction, and this gives the possibility to level-out these deformations already during construction. The higher the building becomes, the bigger the deformations become at the bottom of the building, and the new to construct levels are constructed horizontally. This results in a decrease of deformation for the higher levels of the building. Nevertheless, the construction sequence is not decreasing the deformation at the lower levels.

7.2.2. Sub-Questions 2

The second sub-question is answered in Chapter 4 according to the structure designed in Chapter 3.

How big is the total deformation of the substructure for a high-rise building when the building is constructed in Amsterdam, Rotterdam, and Utrecht?

The total deformation of the substructure consist of the following components:

- Deformation of the foundation piles
- Deformation of the soil under the pile tip:
  - Differential Settlement
  - Uniform Settlement

Figure 7.1 shows the components of the deformation of the concrete building for the three locations. The building used for the calculations is 144 m high and is constructed with a concrete primary structure.

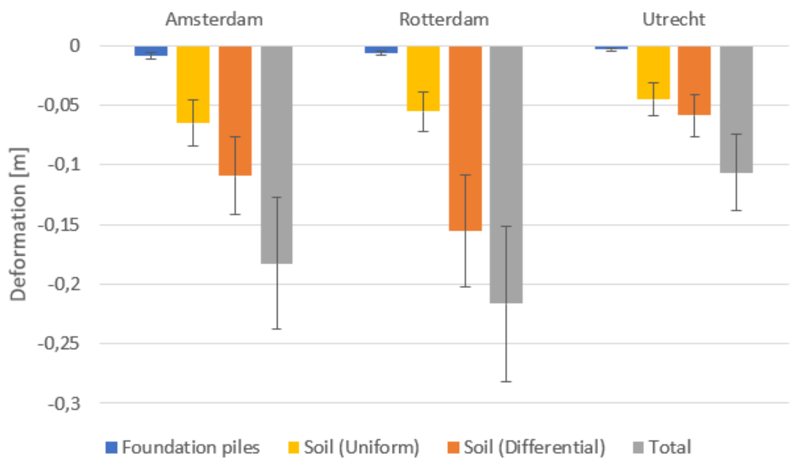


Figure 7.1: Components of the deformation for the concrete alternative

The biggest deformation happens at Rotterdam, where Amsterdam is it slightly smaller. The deformations in Utrecht are the smallest. This is as expected, when observing the soil structures. As concluded in Sub-Question 1, the differential deformation induces problems in the building itself. Therefore the problems due to compressible soil are the biggest in Rotterdam.

The results in Figure 7.1 shows that the deformation of the foundation pile is small compared to the deformation of the subsoil. This result is affected by a simplified method in determining the stiffness of the foundation piles, as explained in the discussion.

The structure of the building is designed in three alternatives, one in concrete, one in steel and one in a combination of concrete and steel (hybrid). The comparison between the weight of the building and the amount of settlement proves that the total weight is an important parameter. An optimal mitigating measure is to build as light as possible. However, this can be an expensive mitigating measure.

In this research, the uncertainty in the deformation of the substructure is assumed as 30 % of the deformation. Therefore, the bigger the deformation, the bigger the uncertainty of the result.

7.2.3. Main-Question

The research question of this research is:

What are possible mitigating measures in the structural design for a high-rise building with discontinuities in the geometry to reduce the risk of unacceptable deformations induced by settlements of compressible soil and what are the advantages and disadvantages of these mitigating measures?

Research proves that the deformation of the compressible soil imposes problems for the concrete alternative at Rotterdam. The lower levels of the high-rise will experience unacceptable deformations, where the deformations at the higher levels are not exceeding the limits. The deformations are calculated considering the construction sequence and the time-dependent behavior of the soil. In this initial design, no explicit mitigating measures are used in the structural design. Therefore it is concluded that this structure needs mitigating measures to decrease the deformations.

The conclusion to the main question can be divided into three parts. In the first part are the considered mitigating measures explained, the second part elaborates on the possibility of applying the mitigating measures, and the third part point-out the advantages and disadvantages of the measures according to a multi-criteria analysis.

Considered Mitigating Measures

The problems due to compressible soil can be mitigated in multiple ways depending on the structural system, the geometry of the building, the structural material, etc. In this research, the following mitigating measures are considered:

- **Changing the building sequence**: This measure adjust the construction sequence in order to attach structural elements which are susceptible for deformation in a later construction stage. In this way a part of the settlement can occur before the structural elements are attached.
- **Increase length of the foundation piles**: For this mitigating measure the foundation piles are founded on the sand-layer which is situated under the compressible layer.
- **Application of a camber in the structure**: For this measure the structure is constructed with a camber in the opposite direction of how the construction will deform.
- **Increase stiffness foundation slab**: This measure distributes the load of the building more evenly over the soil.
- **Increase stiffness of the superstructure**: This measure distributes the load of the buildng more evenly over the soil too.
- **Application of the jack-down system**: With this measures the load-bearing columns can be shortened in order to follow the settlement of the core.

In this research, the mitigating measures are considered to be used solely per mitigating measure in the structural design. In practice, the problem can be mitigated with combinations of mitigating measures.

Possibility of application

If the mitigating measures can be applied depends on the following primary factors:

- **Amount of differential deformation**: The expected amount of differential deformation is an important factor in the decision of which mitigating measure to use. The differential deformation causes the problems in the structure itself.

  The mitigating measures of using a camber is only applicable when the relative rotation limits are not exceeded in the initial design. This mitigating measure does not decrease the total deformation, which is experienced by the structural and non-structural elements.

  The mitigating measures of resisting the deformation (increasing the stiffness of the superstructure or the foundation slab) will reduce the differential deformation. This research shows that the internal forces in the structural elements become significantly large in order to resist the deformations, which results in uneconomical dimensions of these structural elements. The internal forces in these elements are related to the amount of settlement. The bigger the settlement, the higher the internal forces become in the structural elements. To conclude, increasing the stiffness is not an option when the differential settlement of the soil is too big. The other mitigating measures can be applied without significantly increasing internal forces in the structure.

- **Soil structure**: The mitigating measure of increasing the length of the foundation piles to the second sand-layer depends on the soil structure. Increasing the length of the foundation pile is an expensive mitigating measure because special equipment is needed, and a suitable type of foundation pile is needed to be vibrated/screwed through the first sand-layer. For the soil structures used in this research, this measure means that the foundation piles must have a length of 63 m in Amsterdam and 95 m in Rotterdam. The deformation of the soil happening under the foundation piles will significantly reduce because the building does not support on the compressible layer. However, the deformation of the foundation piles increases due to the increased length.

- **Time-dependent behavior of the deformation**: The deformation of the compressible layer can affect the building after construction. This research shows that the deformations in the concrete building in Rotterdam are exceeding the limits after construction. The consolidation time is related to the permeability of the soil and the thickness of the compressible layer.

  When the deformation of the soil is happening quickly due to high permeability, adjusting the building sequence can be considered as mitigating measures. If the deformation of the soil is happening slowly due to high permeability or a thick compressible layer, the jack-down system is a possible mitigating measure. The jack-down system can also be applied when the permeability is high to shorten the construction time.

These primary factors determine the possibility of the mitigating measures. The intermediate conclusion of this part is that the jack-down systems can be applied in all the cases, independent of the amount of settlements, the depth of second sand-layer deep, and the time-dependent behavior. The other mitigating measures do have particular boundary conditions or increase the total cost of the project.

Multi-criteria analysis

An overview of the advantages and disadvantages of the mitigating measures is shown in Table 7.1. In the overview the mitigating measures with positive influences on the aspect compared to the other are mentioned with a '+', the mitigating measures with a negative influence on the aspect have a '-', and the mitigating measures in between are noted with a '≈'. An important footnote must be made; the mitigating measures are ranked for the aspects based on the gained knowledge during the investigation of the problem and into the possible mitigating measures.

Table 7.1: Overview of the ranking of the mitigating measures fo the multi-criteria aspects

Design Considerations	Multi-Criteria Aspects	Change Construction Sequence	Increase Pile Length	Apply Camber	Increase Stiffness Foundation Slab	Increase Stiffness Superstructure	Apply Jack-Down System
Cost	Material Use	+	-	+	-	≈	+
Quality	Functionality	+	+	+	≈	-	≈
	Effect Surroundings	≈	+	≈	-	-	≈
Time	Construction Time	-	≈	+	≈	+	+
Risk	Conventionality	+	-	+	≈	≈	-
	Adaptability	≈	-	≈	-	≈	+

Research on the effect on the surroundings proves that differences between the mitigating measures are small on this aspects, the difference is the way the structure will deform. Only when a (historic) masonry building is in a range of 5-10 m around the new building, the deformation of the soil can exceeds the limits as stated in the CUR166 for the considered building and soil structures in this research.

The overview of the multi-criteria analysis shows that changing the construction sequence, application of the camber, and the jack-down system are the most favorable based on the six aspects of the analysis. Only for adjusting the construction sequence is the aspect of construction time negative and for the jack-down system, the conventionality of the measure is negatively ranked in this analysis. The other three mitigating measures have more negative aspects.

Final conclusions

To conclude this research, the possible mitigating measure for a building depends on the geometry of the building, the amount of settlement, the soil structure, and the time-dependent behavior of the soil. When considering these primary factors and the advantages and disadvantages of the mitigating measures based on the multi-criteria analysis, the jack-down systems can be considered as the favorable option. With this mitigating measure, the deformations in the superstructure can be controlled. This mitigating measure can be applied when the differential deformation is significant, the depth of the foundation layer under the compressible layer does not matter, and the jack-down system can be adjusted during the total life span of the building. The only down-side of this mitigating measure is that this measure is not conventional.

The application of the jack-down system is definitely worth considering when a high-rise building is built on compressible soil. The other mitigating measures can be applied too, however, only with very specific conditions. This makes the jack-down system the most favorable mitigating measure.

7.3. Recommendation

In this section, the recommendations are explained. The recommendations are divided into recommendations for the practice of structural engineer and for further research.

For the structural engineer in practice, the following recommendation is given:

- As already explained in the discussion, the limits are assumed in this research. In reality, these limits need to be determined with the design team of the building. The problem of the compressible soil will be smaller when the limit for the vertical deformation is increased. It is essential to establish the limit for the deformation in an early phase of the design.

For further research the following subjects are proposed:

- The SSI interaction is considered in this research. However, the method used to determine the settlement is simplified and has inaccuracies. For this research, this method was used to get an idea of how big the problems are in the superstructure. When the structure is designed in practice, this method will not satisfy all the requirements. The interface between the structure and the deformation of the soil is a complex subject. More research can be done in order to simplify this problem.
- The redistribution of the internal forces as a consequence of the deformation of the soil can be investigated. In this research, the redistribution is neglected. Further research is needed to see how significant this redistribution is and if it can damage the structure.
- In this research, limits for the deformation of the floor and facade supports are assumed. Further research can be done to investigate the optimal limit for deformation, where the functional reasons of the non-structural elements can be guaranteed and where the most structural material can be saved.

Bibliography

[1] Bepalen draagkracht van de paalfundering. URL https://handboek-prefab-beton.betonhuis.nl/algemeen-prefab/online-handboek-prefab-beton/heipalen/funderingsontwerp/bepalen-draagkracht-van-de-paalfundering--2-/bepalen-draagkracht-van-de-paalfundering--3-.

[2] Prefab heipalen (beton). URL https://www.vroom.nl/nl/products/15-prefab-heipalen-beton.

[3] Betonschroefpalen. URL https://www.vroom.nl/nl/products/7-betonschroefpalen.

[4] Vibropalen type hbf. URL https://www.vroom.nl/nl/products/8-vibropalen-type-hbf.

[5] De rotterdam te rotterdam, 2019. URL https://www.roodenburg.nl/service-projecten/de-rotterdam-middentoren/.

[6] J Banga. Bouwbestluit constructieve veiligheid, 2012.

[7] J. De Vree. Vaste zandlaag. URL http://www.joostdevree.nl/bouwkunde2/vaste_zandlaag.htm.

[8] *DSettlement Manual*. Deltares, February 2016.

[9] Dienst Ruimtelijke Ordening & Bureau Monumenten en Archelogie. *Hoogbouw in Amsterdam*. Gemeente Amsterdam, June 2011. URL https://www.amsterdamsebinnenstad.nl/archief/hoogbouw/hoogbouwvisie.pdf.

[10] Mark Fintel, S. K. Ghosh, and Hal Iyengar. *Column Shortening in Tall Structures - Prediction and Compensation*. Portland Cement Association, 1987.

[11] J Fitoury and J Font Freide. Bijzondere oplossingen zettingen. *Cement*, 2013.

[12] C Hofstadler. Calculation of construction time for building projects. https://www.irbnet.de/daten/iconda/CIB19076.pdf.

[13] Stichting Hoogbouw. *Hoogbouwvisie 2019*. Gemeente Rotterdam, 2019. URL https://www.stichtinghoogbouw.nl/wp-content/uploads/2019/10/Hoogbouwvisie-2019.pdf.

[14] ing. H. J. Everts. Funderen van hoogbouw in nederland. *Cement*, 2001.

[15] HL Janssen. Cur 166 sheet pile walls 6th edition. *SBRCURnet. Gouda, The Netherlands ISBN*, pages 90–3760, 2012.

[16] Cheng Liu, Jack B Evett, et al. *Soils and foundations*. Prentice Hall International, 1992.

[17] Sami S Matar and William J Faschan. A structural engineer's approach to differential vertical shortening in tall buildings. *International Journal of High-Rise Buildings*, 2017.

[18] *NEN-EN 1990+A1+A1/C2 (Eurocode: Grondslagen van het Constructief Ontwerp)*. The Netherlands Standardization Institute, December 2011.

[19] *NEN-EN 1991-1-4 (Eurocode 1: Belatingen op Constructies - Deel 1-4: Algemene Belastingen - Windbelasting)*. The Netherlands Standardization Institute, December 2011.

[20] *NEN 9997-1+C1 (Geotechnisch Ontwerp van Constructies - Deel 1: Algemene Regels)*. The Netherlands Standardization Institute, April 2012.

[21] Ruwan Abey Rajapakse. *Geotechnical engineering calculations and rules of thumb*. Butterworth-Heinemann, 2015.

[22] Wolfgang Schueller. *The Vertical Building Structure*. Van Nostrand Reinhold, 1990.

[23] Gemeente Utrecht Dienst Stadontwikkeling. *Hoogbouwvisie Gemeente Utrecht*. Gemeente Utrecht, January 2005. URL `https://omgevingsvisie.utrecht.nl/fileadmin/uploads/documenten/zz-omgevingsvisie/thematisch-beleid/hoogbouw/2005-01-hoogbouwvisie.pdf`.

[24] Arnold Verruijt. *Soil Mechanics*. Delft University of Technology The Netherlands, 2001.

[25] Nederlandse Organisatie voor Toegepast-natuurwetenschappelijk Onderzoek TNO. URL `https://www.dinoloket.nl/`.

A

Structural Design

In this Appendix the structural calculations are explained.

Windload

The windload is calculated according to the Eurocode and to the National Annex written down in the Bouwbesluit Constructieve Veiligheid [6].

The wind load F_{w,z_e} in kN/m^2 at the reference height z_e is calculated with the following formula:

$$F_{w,z_e} = c_s cd * q_p(z_e) * c_{pe} * A_{ref} \tag{A.1}$$

With $c_s cd$ is the structural factor, $q_p(z_e)$ is the peak velocity pressure in kN/m^2 at the reference height z_e, c_{pe} is the pressure coefficient for the external pressure, A_{ref} is the reference area of the structure in m^2.

In Appendix D of the Eurocode 1 a the graph shown in Figure A.1 is given to determine the $c_s cd$ factor. This graph goes only to maximal 100 meter high. The $c_s cd$ factor is depending on the dynamics of the structure. To make an assumption in the early design a value for, the value will be set on 1.0.

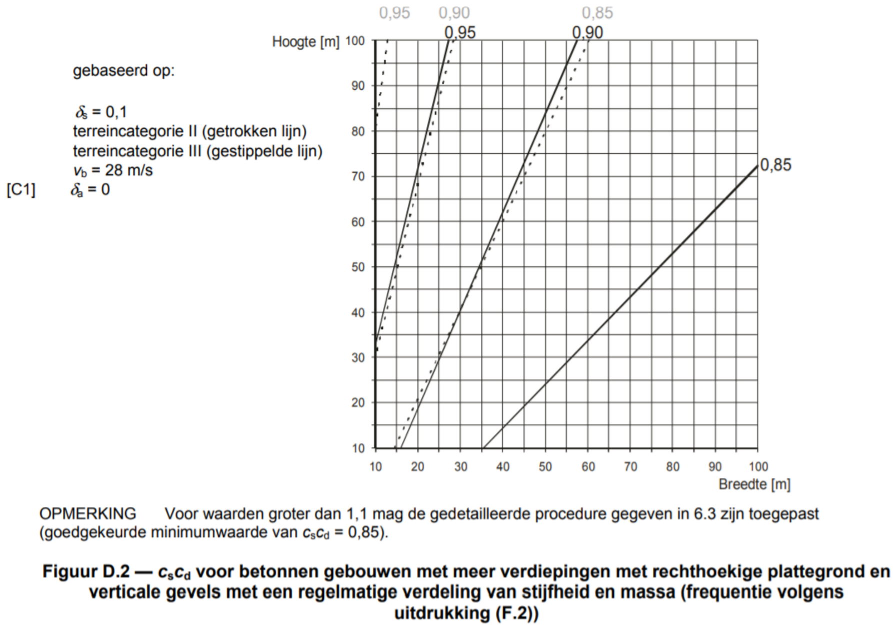


Figure A.1: Structural Factor

The location is also important for the wind loads. The Bouwbesluit Constructieve Veiligheid divides The Netherlands into 3 areas according to Figure A.2. The map shows that the cities of Amsterdam and Rotterdam are in area II and Utrecht is in wind area III. For this research the wind load will be calculated according to wind area II.

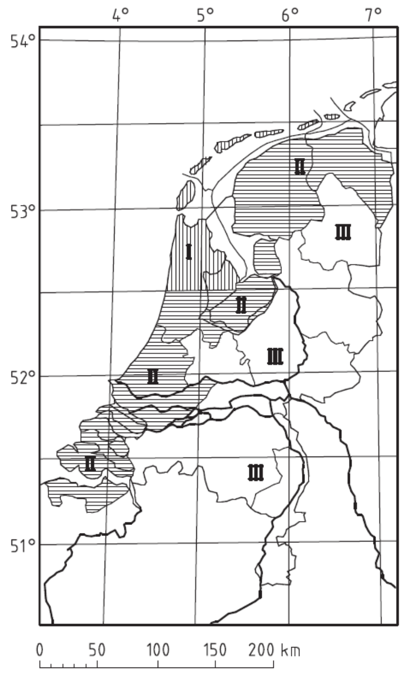


Figure A.2: Wind areas divisions in the Netherlands

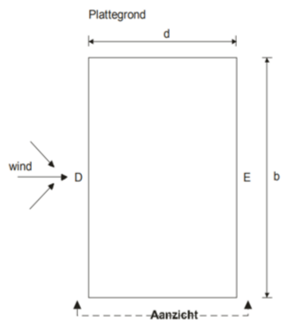
In Figure A.3 the values are shown for the velocity pressure per wind area and for different heights.

Hoogte	Gebied I			Gebied II			Gebied III	
m	kust	onbe-bouwd	be-bouwd	kust	onbe-bouwd	be-bouwd	onbe-bouwd	be-bouwd
1	0,93	0,71	0,69	0,78	0,60	0,58	0,49	0,48
2	1,11	0,71	0,69	0,93	0,60	0,58	0,49	0,48
3	1,22	0,71	0,69	1,02	0,60	0,58	0,49	0,48
4	1,30	0,71	0,69	1,09	0,60	0,58	0,49	0,48
5	1,37	0,78	0,69	1,14	0,66	0,58	0,54	0,48
6	1,42	0,84	0,69	1,19	0,71	0,58	0,58	0,48
7	1,47	0,89	0,69	1,23	0,75	0,58	0,62	0,48
8	1,51	0,94	0,73	1,26	0,79	0,62	0,65	0,51
9	1,55	0,98	0,77	1,29	0,82	0,65	0,68	0,53
10	1,58	1,02	0,81	1,32	0,85	0,68	0,70	0,56
15	1,71	1,16	0,96	1,43	0,98	0,80	0,80	0,66
20	1,80	1,27	1,07	1,51	1,07	0,90	0,88	0,74
25	1,88	1,36	1,16	1,57	1,14	0,97	0,94	0,80
30	1,94	1,43	1,23	1,63	1,20	1,03	0,99	0,85
35	2,00	1,50	1,30	1,67	1,25	1,09	1,03	0,89
40	2,04	1,55	1,35	1,71	1,30	1,13	1,07	0,93
45	2,09	1,60	1,40	1,75	1,34	1,17	1,11	0,97
50	2,12	1,65	1,45	1,78	1,38	1,21	1,14	1,00
55	2,16	1,69	1,49	1,81	1,42	1,25	1,17	1,03
60	2,19	1,73	1,53	1,83	1,45	1,28	1,19	1,05
65	2,22	1,76	1,57	1,86	1,48	1,31	1,22	1,08
70	2,25	1,80	1,60	1,88	1,50	1,34	1,24	1,10
75	2,27	1,83	1,63	1,90	1,53	1,37	1,26	1,13
80	2,30	1,86	1,66	1,92	1,55	1,39	1,28	1,15
85	2,32	1,88	1,69	1,94	1,58	1,42	1,30	1,17
90	2,34	1,91	1,72	1,96	1,60	1,44	1,32	1,18
95	2,36	1,93	1,74	1,98	1,62	1,46	1,33	1,20
100	2,38	1,96	1,77	1,99	1,64	1,48	1,35	1,22
110	2,42	2,00	1,81	2,03	1,68	1,52	1,38	1,25
120	2,45	2,04	1,85	2,05	1,71	1,55	1,41	1,28
130	2,48	2,08	1,89	2,08	1,74	1,59	1,44	1,31
140	2,51	2,12	1,93	2,10	1,77	1,62	1,46	1,33
150	2,54	2,15	1,96	2,13	1,80	1,65	1,48	1,35

Figure A.3: Peak velocity pressure of the wind per wind area

With Figure A.4 the pressure coefficients can be retrieved. The following conclusion can be made:

$$c_{pe} = 0.8 - (-0.7) = 1.5 \tag{A.2}$$



Zone	A		B		C		D		E	
h/d	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,7	
1	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,5	
$\leq 0{,}25$	-1,2	-1,4	-0,8	-1,1	-0,5		+0,7	+1,0	-0,3	

Figure A.4: Topview of the building to determine to zones (top) and Table 7.1 of NEN 1991 to determine the pressure coefficient c_{pe} [19]

According to Figure A.5 the simplification is made to Figure A.6.

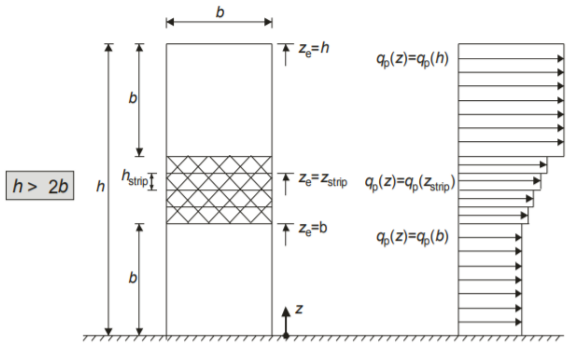


Figure A.5: Reference wind pressure over the height (Figure 7.4 in NEN1991-4) [19]

Windpressure in x-direction

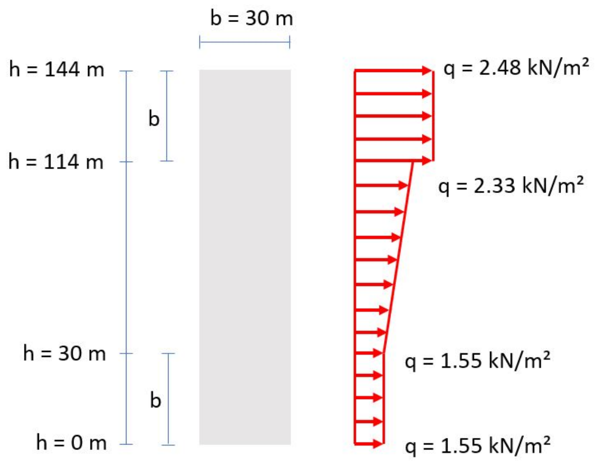


Figure A.6: Simplification of the wind pressure distribution over the height in x-direction

With the above mentioned information the following windpressures are calculated shown in Table A.1

Table A.1: Windpressure on the building according to Figure A.6

Wind pressure [kN/m^2]	Height [m]	$q_p(z_e)$ [kN/m^2]	Wind pressure [kN/m^2]
q_1	144	1.65	2.48
q_2	114	1.55	2.33
q_3	30	1.03	1.55

Windpressure in y-direction

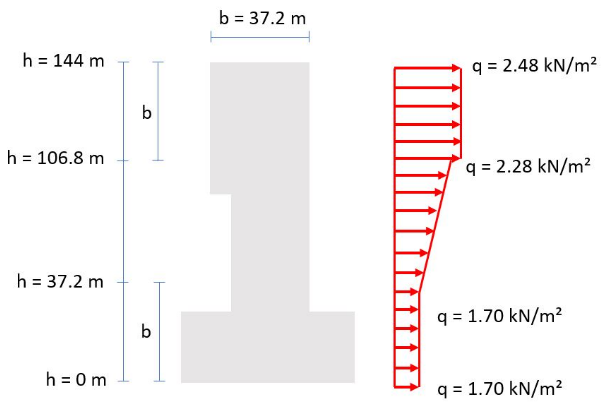


Figure A.7: Simplification of the wind pressure distribution over the height in y-direction

With the above mentioned information the following windpressures are calculated shown in Table A.2

Table A.2: Windpressure on the building according to Figure A.7

Wind pressure [kN/m^2]	Height [m]	$q_p(z_e)$ [kN/m^2]	Wind pressure [kN/m^2]
q_1	144	1.65	2.48
q_2	106.8	1.52	2.28
q_3	37.2	1.13	1.70

For this building the friction force does not influence this calculation. In the Eurocode is stated the friction force needs to be calculated on the area of the outside of the building parallel to the wind direction. Due to the wind behavior the first part of the building, does not contribute to the friction part of the wind loads. In the next Figure a building is shown and only on the dark gray part of the building is friction due to the wind happening. The gray part starts with a distance from the windward side of the building of the minimum distance of 2*b or 4*h as shown in Figure A.8.

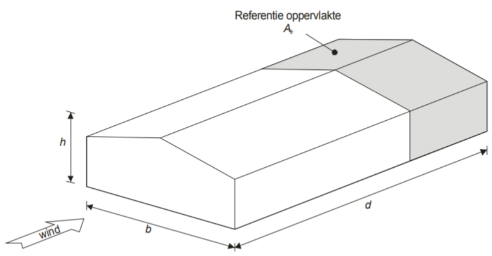


Figure A.8: Friction are of the wind on the building (Figure 7.22 in NEN1991-4) [19]

Stability Core

The stability cores of the structural designs are dimensioned based on the beam formula of a cantilever loaded by a distributed load:

$$u_{top} = \frac{1}{8} * \frac{q * h^4}{E * I} \tag{A.3}$$

The above-mentioned equation can be re-written to calculated the required moment of inertia. The moment of inertia required needs to be increased due to the fact that the core will be constructed with openings in order to enter the core, this will decrease the stiffness. The moment of inertia will be increase by 30%.

$$I_{req} = 1.30 * I = \frac{1}{8} * \frac{q * h^4}{E * u_{top}} \tag{A.4}$$

With the following parameters:

- q = width of the building * windpressure = 30 m * 2.48 kN/m^2 = 74.4 kN/m
- h = 144 m
- w_{top} = 0.144 m

Concrete	Steel
E_c = 0.7 * 38000 N/mm^2	E_s = 210000 N/mm^2
I, req = 1357.2 m^4	I, req = 171.9 m^4

Concrete Core

Figure A.9 shows the geometry of the core and in Table A.3 the dimensions are stated.

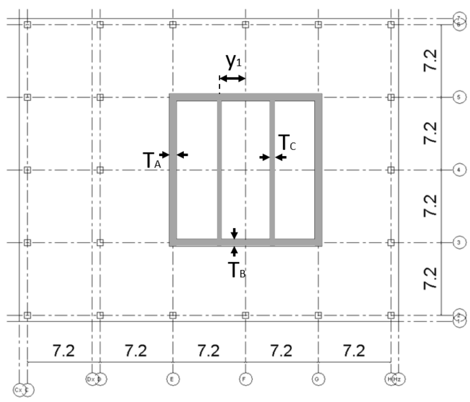


Figure A.9: Horzontal cross-section to show the concrete core

The dimensions of the core are mentioned in the following table:

The moment of inertia in y-direction of wall i is calculated according to the Steiner Rule:

$$I_{y,i} = \frac{1}{12} * x_t * y_t^3 + x_t * y_t * y_1^2 \tag{A.5}$$

The moment of inertia in x-direction can be calculated in the same way, only the x and y's have to be swapped.

Table A.3: Dimensions of the core

Wall	y_t [m]	x_t [m]	y_1 [m]	x_1 [m]
1	14.4	0.75	0	-7.2
2	14.4	0.75	0	7.2
3	0.8	14.4	-7.2	0
4	0.8	14.4	7.2	0
5	14.4	0.5	0	-3
6	14.4	0.5	0	-3

The moment of inertia in the y-direction:

Table A.4: Moment of inertia in y-direction

Wall	I_y (Shape) [m^4]	I_y (Steiner) [m^4]
1	186.6	0
2	186.6	0
3	0.61	597.1
4	0.61	597.1
5	124.4	0
6	124.4	0
Total [m^4]	623.3	1194.4
	Total (Summation) [m^4]	1817.7
	Unity Check	0.75

The moment of inertia in x-direction:

Table A.5: Moment of inertia in x-direction

Wall	I_x (Shape) [m^4]	I_x (Steiner) [m^4]
1	0.51	559.9
2	0.51	559.9
3	199.1	0
4	199.1	0
5	0.15	64.8
6	0.15	64.8
Total [m^4]	399.4	1249.3
	Total (Summation) [m^4]	1648.8
	Unity Check	0.82

The unity checks are both ok, so assumed is that the dimensions of the core are sufficient.

Steel Truss Core

The core for the steel-light weight option is constructed out of HD 400 x 818 columns. The locations are shown in Figure A.10

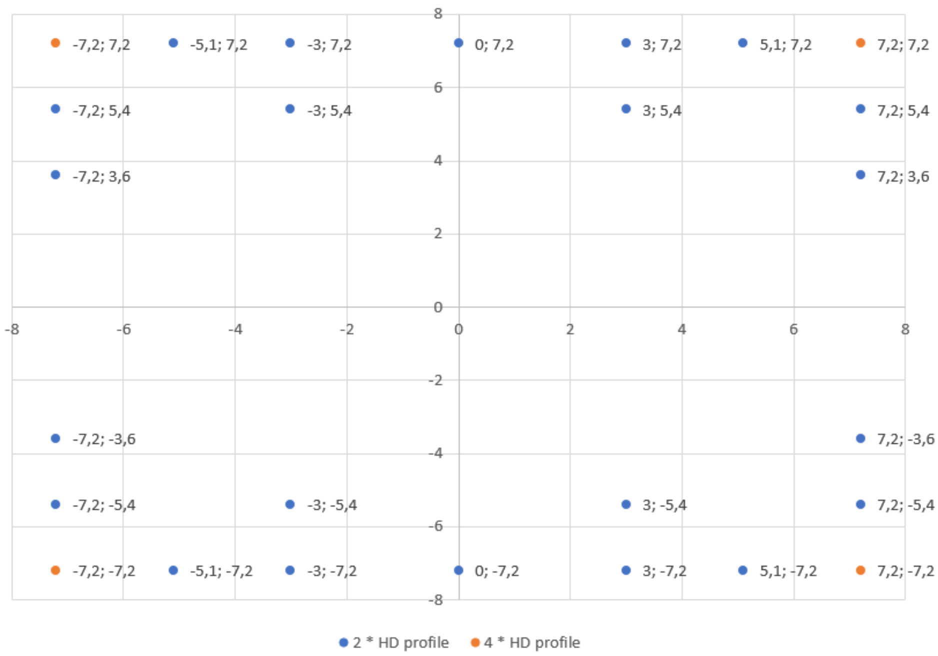


Figure A.10: The locations of the columns for the steel stability truss

The moment of inertia is calculated in the same way as the concrete core, with equation A.5. With the area of column is two times a HD 400 x 818 profile ($A = 2 * 0.10434\ m^2 = 0.20868\ m^2$). For the moment of inertia for the steel stability core only the Steiner part of the equation is considered, because the moment of inertia of the shape of the steel profile is very small compared to the part of the Steiner rule. For deformation of the steel stability core will only stay within the limit when the columns of the stability core are horizontally connected in a way that the total cross section works as one core.

Table A.6: Results of the total moment of inertia in both directions

	y-direction		x-direction	
Total	254.2	m^4	209.8	m^4
U.C.	0.67		0.82	

Floorsystems

The floors are dimensioned to withstand the floor loads and the self-weight of the floors as given in Paragraph 3.2.

Concrete Alternative

The thickness of the concrete floor is 250 mm. The normative floor is the floor loaded by the weight of the installations at the 20th level. So, this floor thickness is checked for the strength and the serviceability limit states.

Strength

Figure A.11 shows that the field moment is 72.15 kNm/m. The negative moment occurs at the supports of the floor and assumed is that these moments are high due singularities and therefore neglected in this validation of the floor thickness.

To validate the floor thickness the reinforcement needed to withstand the field moment is calculated.

$$h = 250mm \tag{A.6}$$

$$d_{assumed} = 200mm \tag{A.7}$$

$$f_{yd} = 435\frac{N}{mm^2} \tag{A.8}$$

$$z = 0.9 * d = 0.9 * 200 = 180mm \tag{A.9}$$

$$A_{s,req} = \frac{M_{Ed}}{z * f_{yd}} = \frac{72.15 * 10^6}{200 * 435} = 830\frac{mm^2}{m} \tag{A.10}$$

So, a reinforcement configuration of 5 ∅ 16 (= 1005 $\frac{mm^2}{m}$) is sufficient, thus the floor thickness is ok.

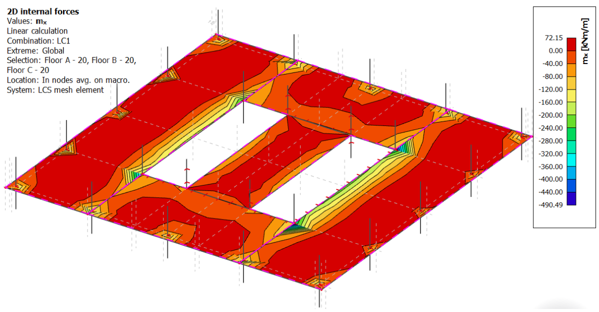


Figure A.11: Moment in x-direction in the concrete floor loaded in ULS

Serviceability

For the serviceability limit the following limit can not be exceeded.

$$w_{max} = \frac{3 * span}{1000} = \frac{3 * 7200}{1000} = 21.6mm \tag{A.11}$$

Figure A.12 shows the floor deflection in SLS. The Figure shows that the maximal floor deflection is 7.4 mm and this value is smaller than the limit of 21.6 mm. So, therefore the floor thickness is also ok for the serviceability limit.

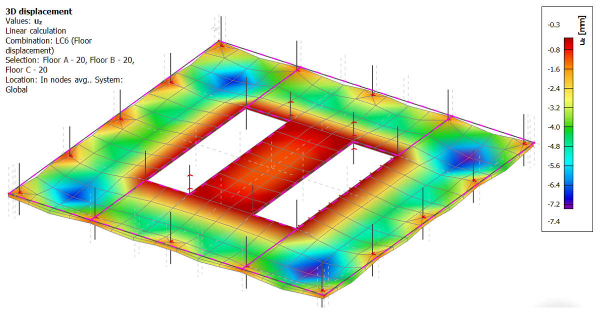


Figure A.12: Deformation of the concrete floor in SLS

Steel Alternative

For the steel Alternative the floor is constructed with an Comflor spanning 7.2 m. The floor configuration is shown in Figure A.13.

The floor system used for this option is Comflor 210 mm with a height of 300 mm. Extra fire resistance is needed because the floor system can only provide fire resistance of 90 minutes where 120 minutes is required.

The floorload is:

- $Q_{G,Comflor210}$ = 327 * 9.81 = 3.21 kN/m^2
- $Q_{Q,office}$ = 2.5 kN/m^2
- Ψ_0 = 0.5
- $Q_{permanent}$ = 2.25 kN/m^2

Span = 7.2 m
ULS: q = 7.2 * (1.32 * (3.21 + 2.25) + 1.65 * 2.5) = 81.59 kN/m
SLS: q = 7.2 * (1.0 * (3.21 + 2.25) + 1.0 * 2.5) = 57.31 kN/m

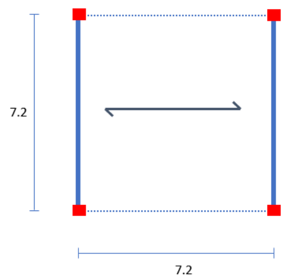


Figure A.13: Top view of the floor plan for the steel alternative

Serviceability

the maximum deflection for the floor is:

$$w_{max} = \frac{span}{500} = \frac{3*7200}{1000} = 14.4mm \tag{A.12}$$

The required moment of inertia assumed that S355 steel is used:

$$I_{req} = \frac{5}{384} * \frac{q*l^4}{E*w_{max}} = \frac{5}{384} * \frac{57.31kN/m*7200^4}{210000*14.4} = 66320*10^4 mm^4 \tag{A.13}$$

The possible beam is:

HEB450 I,y = 79888 * 10^4 mm^4 q_g = 174 kg/m

Strength

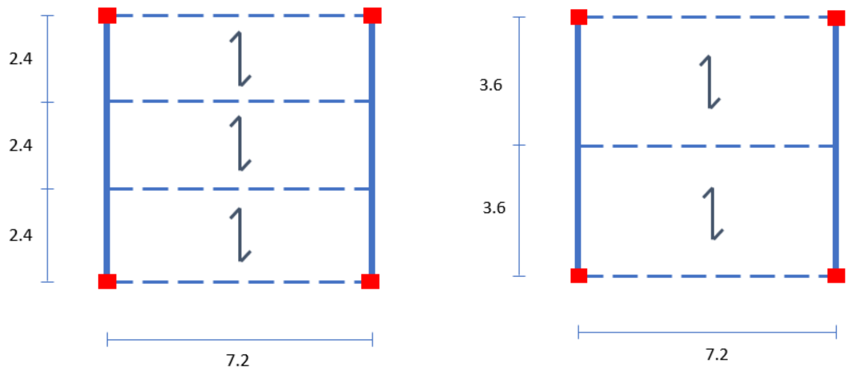
Normally for the floor supporting beams the deflection is normative over the strength of the beam. But, for certainty the strength is checked:

$$\sigma_s = \frac{M_{Ed}}{W_{el}} = \frac{\frac{1}{8}*81.59N/mm*7200^2 mm}{3551*10^3 mm^3} = 148.89MPa \quad <= \quad \frac{f_y}{\gamma_s} = \frac{355}{1.15} = 309MPa \tag{A.14}$$

So the beam is sufficient as supporting beam.

Steel Light Weight Alternative

For the light weight alternative two possible floors configurations are considered. The two configurations are shown in Figure A.14.



(a) Primary Beam of the option with a floorspan of 2.4 m

(b) Primary Beam of the option with a floorspan of 3.6 m

Figure A.14: Topview of the two options considered for the lightweight floor for the steel alternative

For the determination of the floor type to use the following floors are considered:

Span = 2,4 m

Comflor 46	h = 120 mm	=>	q_g = 251 kg/m^2
Comflor 51+	h = 115 mm	=>	q_g = 279 kg/m^2
Comflor E60	h = 150 mm	=>	q_g = 285 kg/m^2

Span = 3,6 m

Comflor 51+	h = 115 mm	=>	q_g = 279 kg/m^2
Comflor E60	h = 150 mm	=>	q_g = 285 kg/m^2
Comflor 75	h = 140 mm	=>	q_g = 267 kg/m^2
Comflor 95	h = 160 mm	=>	q_g = 292 kg/m^2

The spans of 2.4 and 3.6 m are considered because this are the possible spans when using 4 or 3 secondary supporting beams. The floor thickness of the span 2.4 m could be less, but the thickness of the floor to guarantee the fire resistance of 90 minutes is a requirement. So additional fire resistance material is needed to guarantee the 120 minutes fire resistance.

This lead to the following floor loads:

- $Q_{G,Comflor46}$ = 251 * 9.81 = 2.47 kN/m
- $Q_{G,Comflor75}$ = 267 * 9.81 = 2.62 kN/m
- Q_{Office} = 2.5 kN/m
- Ψ_0 = 0.5
- $Q_{permanent}$ = 2.25 kN/m

Load on the secondary beams:

Span = 2.4 m
ULS: q = 2.4 * (1.32 * (2.47 + 2.25) + 1.65 * 2.5) = 24.85 kN/m
SLS: q = 2.4 * (1.0 * (2.47 + 2.25) + 1.0 * 2.5) = 17.33 kN/m

Span = 3.6 m
ULS: q = 3.6 * (1.32 * (2.62 + 2.25) + 1.65 * 2.5) = 37.99 kN/m
SLS: q = 3.6 * (1.0 * (2.62 + 2.25) + 1.0 * 2.5) = 26.53 kN/m

Serviceability

Secondary Beam

The maximum deflection of the floor is:

$$w_{max} = \frac{span}{500} = \frac{7200}{500} = 14.4mm \tag{A.15}$$

The required moment of inertia assumed that S355 steel is used for the floor span of 2.4 m:

$$I_{req} = \frac{5}{384} * \frac{q * l^4}{E * w_{max}} = \frac{5}{384} * \frac{17.33kN/m * 7.2^4}{210000 * 14.4} = 20050 * 10^4 mm^4 \tag{A.16}$$

The possible beams are:

IPE400 I,y = 23128 * 10^4 mm^4 q_g = 67.6 kg/m

The lightest beam possible is the IPE400 for the option with a floor span of 2.4 m.

The required moment of inertia assumed that S355 steel is used for the floor span of 3.6 m:

$$I_{req} = \frac{5}{384} * \frac{q * l^4}{E * w_{max}} = \frac{5}{384} * \frac{26.53kN/m * 7200^4}{210000 * 14.4} = 30701 * 10^4 mm^4 \tag{A.17}$$

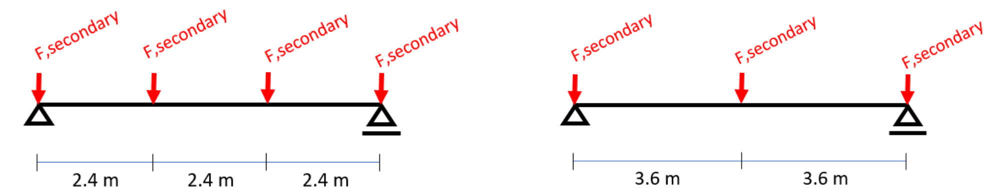
The possible beams are:

IPE400 I,y = 33743 * 10^4 mm^4 q_g = 79.1 kg/m

The lightest beam possible is the IPE450 for the option with a floor span of 3.6 m.

Primary Beam

The primary beam is supporting the secondary beam as shown in Figure A.15



(a) Primary Beam of the option with a floorspan of 2.4 m (b) Primary Beam of the option with a floorspan of 3.6 m

Figure A.15: Schematic drawing of the two options of the primary beam

The moment of Inertia is estimated according to the following beam formula, this is for one load in the middle of the span. So, sufficient for the option with the floor span of 3.6 m. For the option with the floor span of 2.4 m this beam formula is also used, but then F = 2 * F,secondary. Where $F = 7.2 * q_{secondarybeam} + 7.2 * q_{self-weigth,beam}$

The maximum deflection of the floor is:

$$w_{max} = \frac{span}{500} = \frac{7200}{500} = 14.4mm \tag{A.18}$$

The required moment of inertia assumed that S355 steel is used for the floor span of 2.4 m:

$$I,req = \frac{1}{48} * \frac{2 * F * l^3}{E * w_{max}} = \frac{1}{48} * \frac{2 * 128.87kN * 7200^3}{210000 * 14.4} = 66277.3 * 10^4 mm^4 \tag{A.19}$$

The possible beams are:

IPE550 I,y = 67117 * 10^4 mm^4 q_g = 108 kg/m

The lightest beam possible is the IPE500 beam for the option with a floor span of 2.4 m.

The required moment of inertia assumed that S355 steel is used for the floor span of 3.6 m:

$$I,req = \frac{1}{48} * \frac{F * l^3}{E * w_{max}} = \frac{1}{48} * \frac{195.78kN * 7200^3}{210000 * 14.4} = 50342 * 10^4 mm^4 \tag{A.20}$$

The possible beams are:

IPE550 I,y = 67117 * 10^4 mm^4 q_g = 108 kg/m

The lightest beam possible is the IPE550 beam for the option with a floor span of 3.6 m.

Because these beams are determined with a simplified beam formula, the deflection of the beams is checked with SCIA engineer.

Foor the floorspan of 2.4 m:

$$u_{max,occuring} = 8.4mm \quad < \quad u_{max} = 21.6mm \tag{A.21}$$

For the floorspan of 3.6 m:

$$u_{max,occuring} = 9.2mm \quad < \quad u_{max} = 21.6mm \tag{A.22}$$

Strength

Normally for the floor supporting beams the deflection is normative over the strength of the beam, but the strength must be checked:

Floor span = 2.4 m:

$$\sigma_s = \frac{M_{Ed}}{W_{el}} = \frac{\frac{1}{8} * 24.85N/mm * 7200^2 mm}{1156 * 10^3 mm^3} = 139.31MPa \quad <= \quad \frac{f_y}{\gamma_s} = \frac{355}{1.15} = 309MPa \tag{A.23}$$

Floor span = 3.6 m:

$$\sigma_s = \frac{M_{Ed}}{W_{el}} = \frac{\frac{1}{8} * 37.99N/mm * 7200^2 mm}{1500 * 10^3 mm^3} = 164.13MPa \quad <= \quad \frac{f_y}{\gamma_s} = \frac{355}{1.15} = 309MPa \tag{A.24}$$

So the chosen beams are sufficient as secondary beams.

Now, the primary beams. The moment occuring in the beams:

Floor span = 2.4 m:

$$M_{Ed} = F_{sec} * \frac{7.2m}{2} - F_{sec} * \frac{7.2m}{6} = 240.31kN * \frac{7.2}{2} - 240.31kN * \frac{7.2}{6} = 578.35kNm \tag{A.25}$$

The strength of the beam:

$$\sigma_s = \frac{M_{Ed}}{W_{el}} = \frac{578.35kNm}{2441 * 10^3 mm^3} = 299.97MPa \quad <= \quad \frac{f_y}{\gamma_s} = \frac{355}{1.15} = 309MPa \tag{A.26}$$

Floor span = 3.6 m:

$$M_{Ed} = \frac{1}{4} * F_{sec} * 7.2m = \frac{1}{4} * 365.82kN * 7.2m = 660.00kNm \tag{A.27}$$

The strength of the beam:

$$\sigma_s = \frac{M_{Ed}}{W_{el}} = \frac{658.48kNm}{2241*10^3mm^3} = 294.51MPa \quad <= \quad \frac{f_y}{\gamma_s} = \frac{355}{1.15} = 309MPa \tag{A.28}$$

So, the chosen beams are sufficient as primary beams.

Lightest Possible Option?

For the light steel alternative the lightest option of the two configurations needs to be chosen. For this decision the weight of a floor area of 7.2 * 7.2 m is calculated.

Floor span = 2.4 m

$$W = 7.2*q_{G,pri} + 3*7.2*q_{G,sec} + 7.2*7.2*q_{floor} = 396.24kN \tag{A.29}$$

Floor span = 3.6 m

$$W = 7.2*q_{G,pri} + 2*7.2*q_{G,sec} + 7.2*7.2*q_{floor} = 400.86kN \tag{A.30}$$

So the option with the floor span of 2.4 m is the lightest floor configurations.

Columns

The dimensions of the columns are estimated according to a weight calculation. In this weight calculation the geometry, the self-weight of the structure and the imposed loads are taken into account. The columns are divided in Column Groups as shown in Figure 3.12.

The dimensions of the columns are determined based on the axial resistance needed, according to Formula A.31, and the critical buckling load according to Formula A.32.

$$A_{req} = \frac{N_{max}}{f_{compressive}} \tag{A.31}$$

With A_{req} is the needed cross sectional area of the column in mm^2, N_{max} is the maximal axial force in the columns, in kN, and $f_{compressive}$ is the resistance to compression, in N/mm^2.

$$F_{cr} = \frac{\pi^2 * EI}{(K * L)^2} \tag{A.32}$$

With F_{cr} is the critical buckling load, EI is the bending stiffness of the column in the weak axis in Nmm^2, K is the column effective length factor assumed to be 1.0, this is a conservative assumption because this means that the column is hinged at the supports and in fact it will be partial clamped and L is the buckling length in m.

Concrete Alternative

The results of the weight calculation and the Unity Checks of the concrete alternative are shown in Table A.7.

Table A.7: Normal forces in the column of the weight calculation for the concrete alternative

Column Group	Dimensions Column		Normal Force	Axial Force		Buckling	
	b (mm)	d(mm)	N,Ed (kN)	N,Rd (kN)	U.C.	F,Cr	U.C.
1	450	450	13.1 * 10^3	13.2 * 10^3	0.99	160.3 * 10^2	0.08
2	850	850	42.9 * 10^3	47.2 * 10^3	0.91	510.1 * 10^2	0.08
3	600	600	22.2 * 10^3	23.5 * 10^3	0.91	126.7 * 10^2	0.17
4	400	400	4.7 * 10^3	10.5 * 10^3	0.38	25.0 * 10^2	0.16

Hybrid Alternative

The results of the normal forces in the column of the weight calculations and the Unity Checks of are shown in Table A.8

Table A.8: Normal forces in the column of the weight calculation for the hybrid alternative

Column Group	Steel profile	Normal Force	Axial Force		Buckling	
		N,Ed (kN)	N,Rd (kN)	U.C.	F,Cr	U.C.
1	HD 400 x 237	9.6 * 10^3	10.7 * 10^3	0.90	10.6 * 10^3	0.91
2	HD 400 x 818	31.6 * 10^3	37.0 * 10^3	0.85	33.0 * 10^3	0.96
3	HD 400 x 421	16.1 * 10^3	19.1 * 10^3	0.84	16.6 * 10^3	0.97
4	HE 300 M	2.8 * 10^3	10.8 * 10^3	0.26	9.1 * 10^3	0.31

Steel Light Weight Alternative

The results of the normal forces in the column of the weight calculations and the Unity Checks of are shown in Table A.9

Table A.9: Normal forces in the column of the weight calculation for the steel alternative

Column Group	Steel profile	Normal Force	Axial Force		Buckling	
		N,Ed (kN)	N,Rd (kN)	U.C.	F,Cr	U.C.
1	HD 400 x 216	$8.8 * 10^3$	9798	0.90	$45.1 * 10^3$	0.20
2	HD 400 x 818	$29.4 * 10^3$	$37.0 * 10^3$	0.79	$217.5 * 10^3$	0.14
3	HD 400 x 421	$15.1 * 10^3$	$19.0 * 10^3$	0.79	$96.1 * 10^3$	0.16
4	HE 300 M	$2.61 * 10^3$	$10.8 * 10^3$	0.24	$31.0 * 10^3$	0.08

Diagonals

The diagonals are dimensioned based on the results of the weight calculation and a simple calculation to redirect the forces.

The forces are calculated with the following formulas according the Figure A.16:

$$F_1 = \frac{N_{Ed}}{cos\alpha} \tag{A.33}$$

$$F_2 = \frac{N_{Ed}}{tan\alpha} \tag{A.34}$$

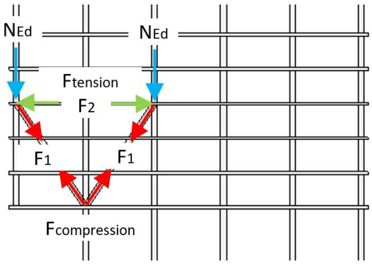


Figure A.16: Diagonals and tension beam

Concrete Alternative

The internal forces of the diagnoals and the tension beam and also the Unity Checks are shown in Table A.10

Table A.10: Normal forces in the diagonal and tension of the weight calculation for the concrete alternative

Structural element	Dimensions (mm)	Internal Force (kN)	Axial Force N,Rd	U.C.	Buckling F,Cr	U.C.
Diagonal	550 x 550	$10.9 * 10^3$	$19.8 * 10^3$	0.55	$247.6 * 10^3$	0.04
Horizontal beam	HEM300	$6.0 * 10^3$	$10.7 * 10^3$	0.56	-	-

Hybrid Alternative

The internal forces of the diagnoals and the tension beam and also the unity checks are shown in Table A.11

Table A.11: Normal forces in the diagonal and tension of the weight calculation for the hybrid alternative

Structural element	Dimensions	Internal Force (kN)	Axial Force N,Rd	U.C.	Buckling F,Cr	U.C.
Diagonal	HD 400 x 509	$8.2 * 10^3$	$23.0 * 10^3$	0.36	$19.1 * 10^3$	0.43
Horizontal beam	HEM300	$4.6 * 10^3$	$10.7 * 10^3$	0.42	-	-

Steel Alternative

The internal forces of the diagnoals and the tension beam and also the Unity Checks are shown in Table A.12

Table A.12: Normal forces in the diagonal and tension of the weight calculation for the steel alternative

Structural element	Dimensions	Internal Force (kN)	Axial Force N,Rd	U.C.	Buckling N,Rd	U.C.
Diagonal	HD 400 x 463	$7.7 * 10^3$	$20.9 * 10^3$	0.37	$20.2 * 10^3$	0.37
Horizontal beam	HEM300	$4.3 * 10^3$	$10.7 * 10^3$	0.40	-	

Bearing Capacity of the Foundation Piles

For this research three different foundation piles are considered. In this section the types of foundation piles is chosen based on the load bearing capacity.

The following piles are considered:

- Prefab Concrete Foundation Piles, A = 500 mm * 500 mm.
- Vibro piles Type HBF, d = 610 mm
- Concrete screw pile, d = 600 mm

The above-mentioned foundation piles are chosen because these piles have an high load bearing capacity, which is needed for an high-rise building. The prefab piles are commonly used in the Netherlands, an advantage is that the properties of the foundation pile itself are predictable. The advantage of the Vibro pile is that the length is variable, as the piles are cast in-situ. The advantage of the screw pile is that the placing can be done without vibration for the surroundings.

For the calculations an equivalente diameter for the square prefab pile is needed, this is calculated with the following formula:

$$d = b * \sqrt{\frac{4}{\pi}} \tag{A.35}$$

The equivalente thickness is of the 500 * 500 mm^2 is 564 mm.

With the selected Cone Penetration Tests (CPT) this results in the following load bearing capacities of the different type of foundation piles in the three locations. In Appendix B the determination of the pile tip resistance and the shaft friction resistance are shown per location. In Table A.13 the load bearing capacities are shown.

Table A.13: Load bearing capacity of the three types of foundation piles at the three different locations

Location	Type of pile	R,b,cal (kN)	R,s,cal (kN)	R,total,cal (kN)
	Prefab	2886	1040	3926
Amsterdam	Vibro	3375	1395	**4771**
	Screw	2613	588	3201
	Prefab	1876	1100	2976
Rotterdam	Vibropile	2194	1467	**3670**
	Schroef	1698	622	2320
	Prefab	2186	2121	4307
Utrecht	Vibropile	2557	2845	**5402**
	Schroef	1979	1199	3179

Based on the load bearing capacities in Table A.13 of the different foundation piles can be seen that the Vibropile type HBF is the pile with the most load bearing capacity. The increase in load bearing capacity is depending on the properties α_p and α_s, these properties the highest for the Vibropile and therefore the load bearing capacity is the highest.

Construction Sequence

The construction sequence used for this research is shown Table A.14

Table A.14: Construction sequence of the concrete alternative

		Constructed to		
Construction Stage	Construction Time [days]	Primary Structure	Facade	Finishes
1	90	level 0	-	-
2	160	level 5	-	-
3	208	level 11	level 5	-
4	228	level 17	level 11	level 5
5	378	level 21	level 17	level 11
6	402	level 27	level 21	level 17
7	442	level 33	level 27	level 21
8	490	level 38	level 33	level 27
9	538	-	level 38	level 33
10	728	-	-	level 38

B
Soil Structures

This appendix shows the used information of the soil structures are shown.

- Figure B.1 shows the pile tip resistance for Amsterdam
- Figure B.2 shows the shaft resistance for Amsterdam
- Figure B.3 shows the pile tip resistance for Rotterdam
- Figure B.4 shows the shaft resistance for Rotterdam
- Figure B.5 shows the pile tip resistance for Utrecht
- Figure B.6 showsthe shaft resistance for Utrecht
- Figure B.7 shows the total DSP for the three locations

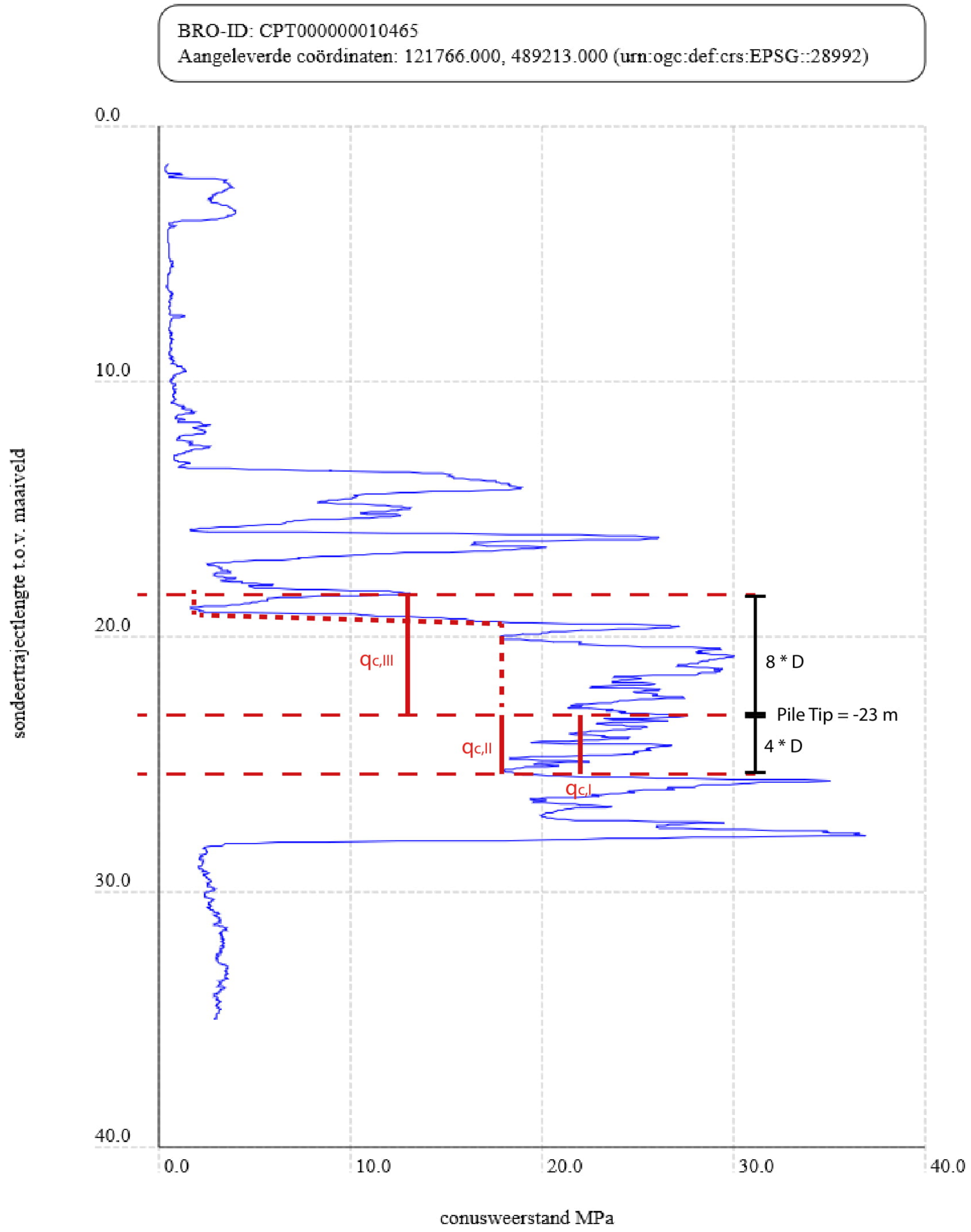


Figure B.1: Amsterdam - pile tip resistance

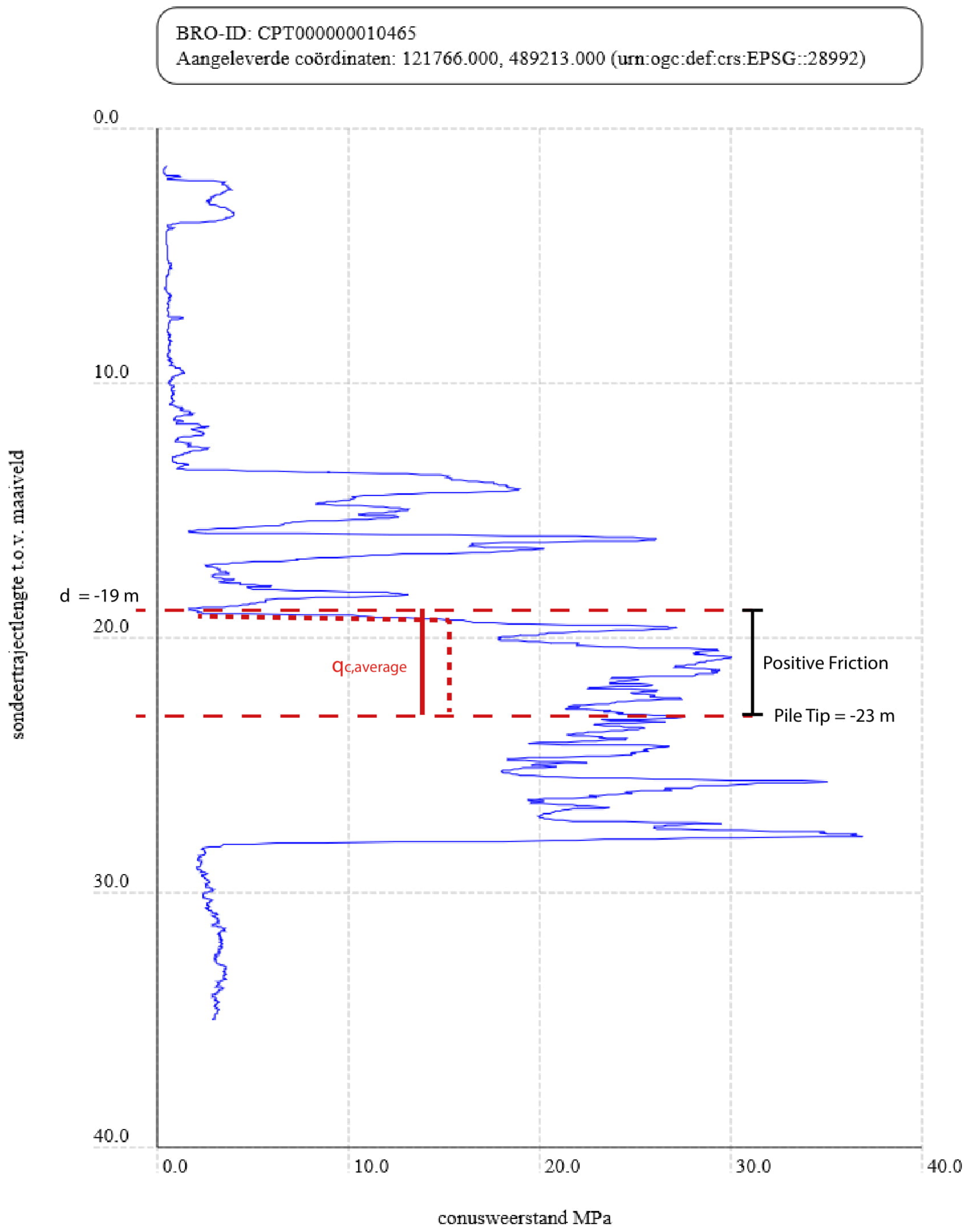


Figure B.2: Amsterdam - shaft friction resistance

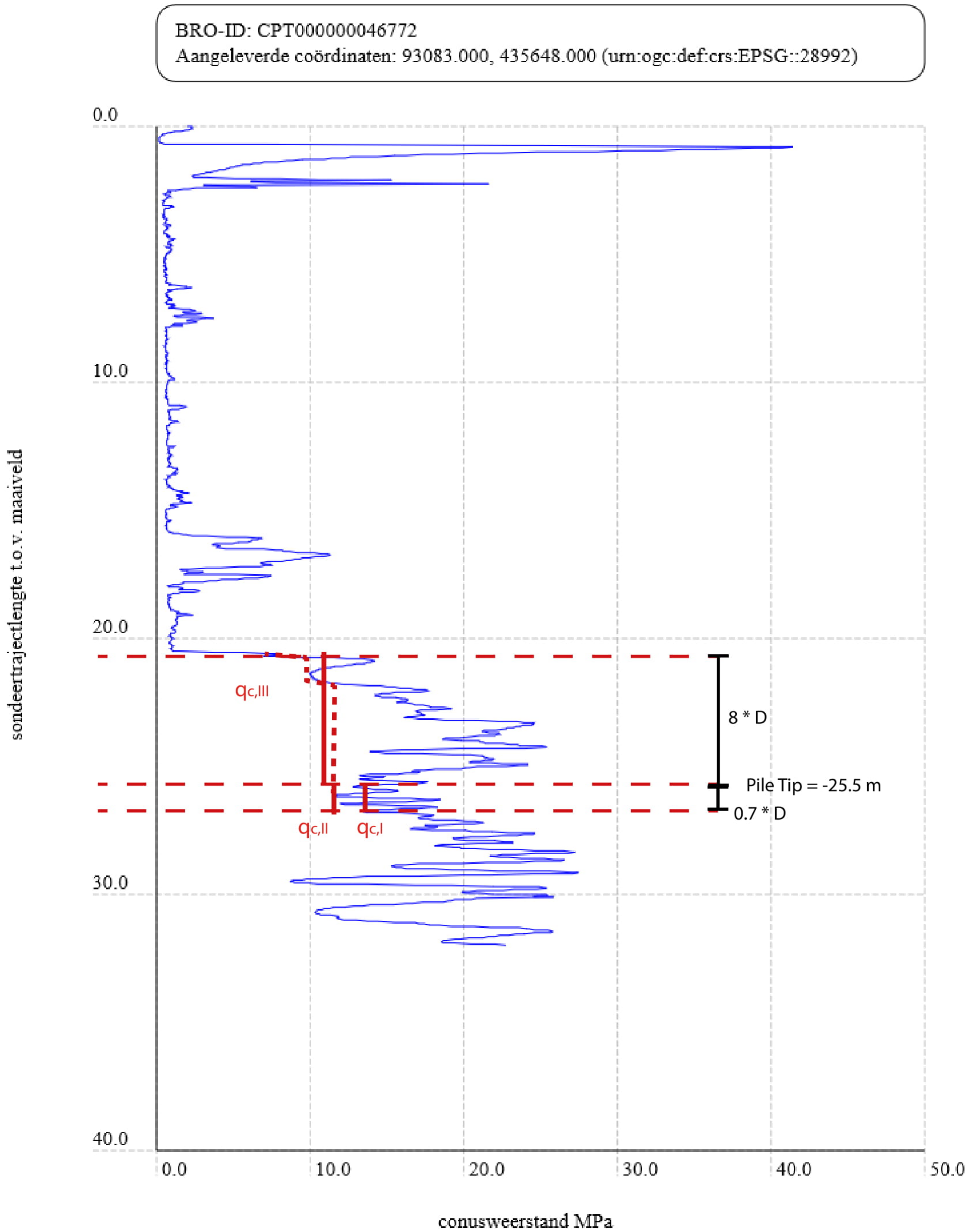


Figure B.3: Rotterdam - pile tip resistance

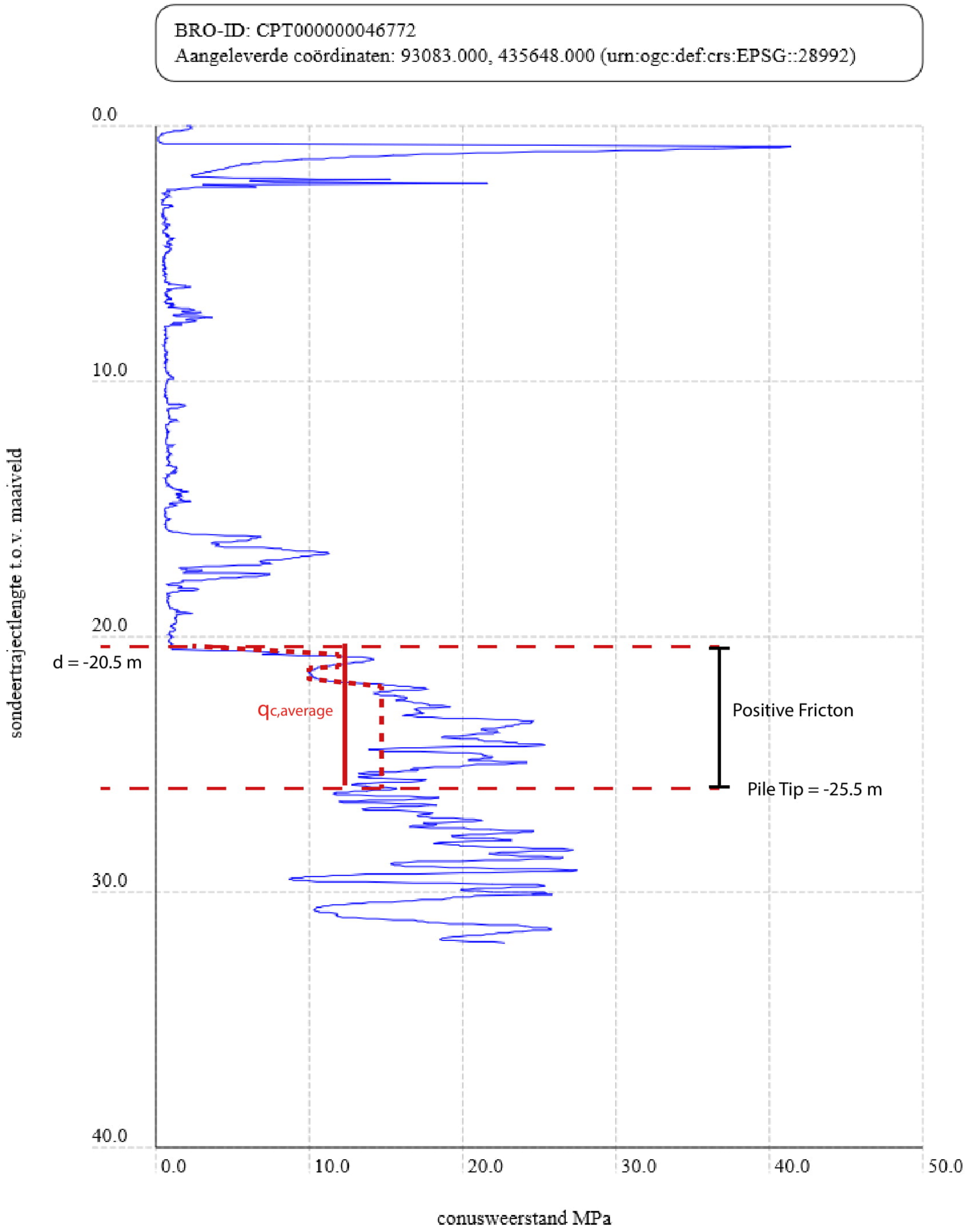


Figure B.4: Rotterdam - shaft friction resistance

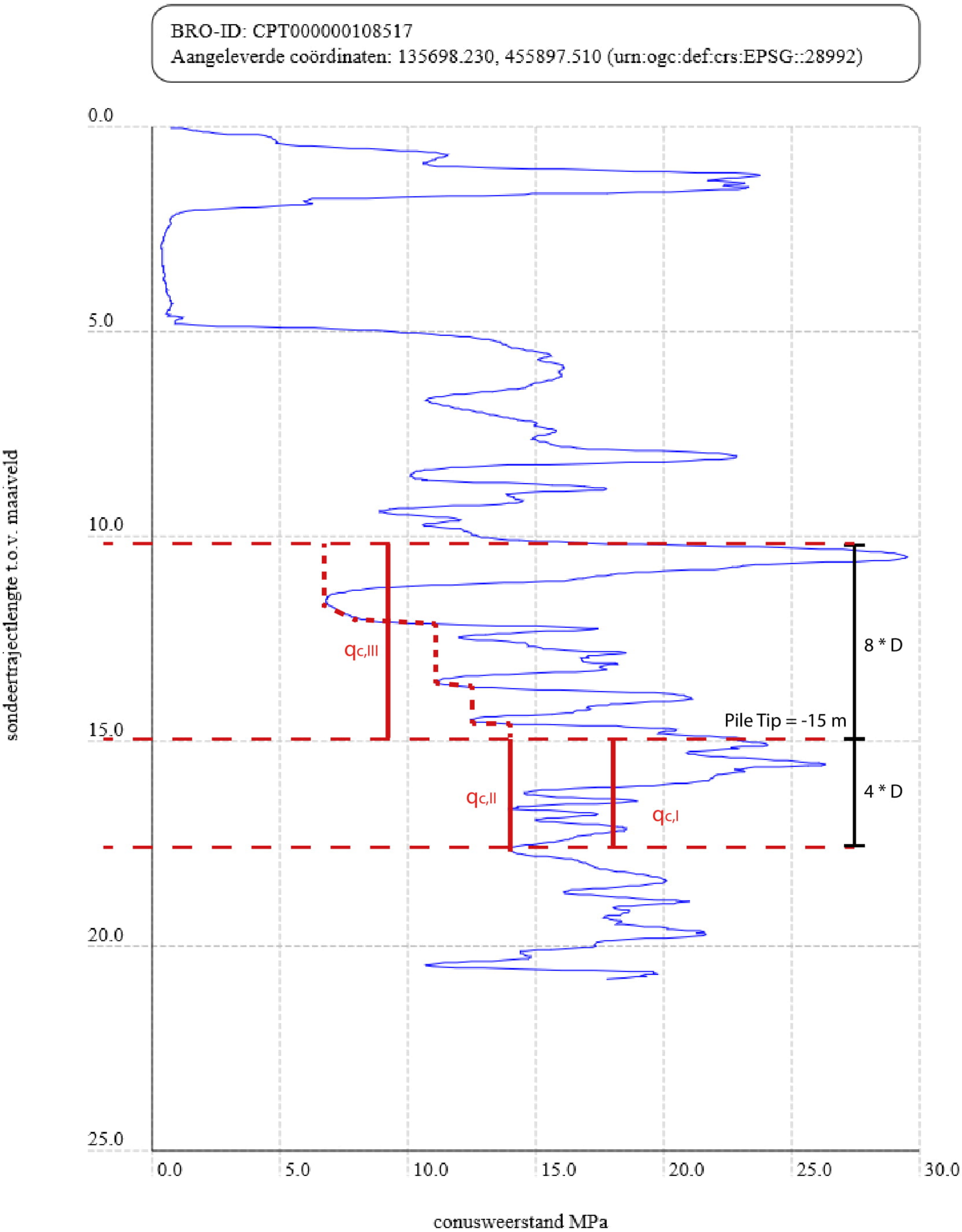


Figure B.5: Utrecht - pile tip resistance

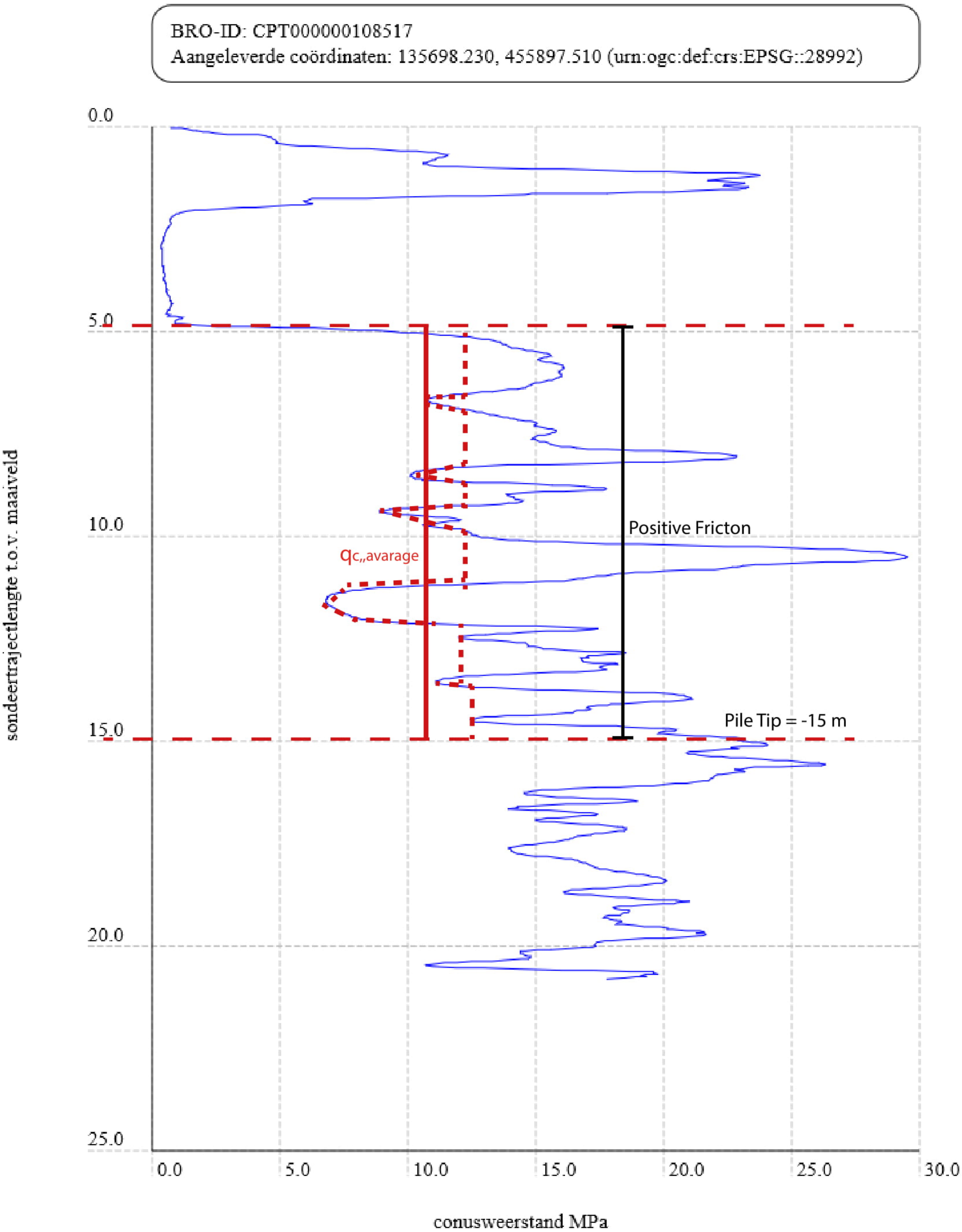
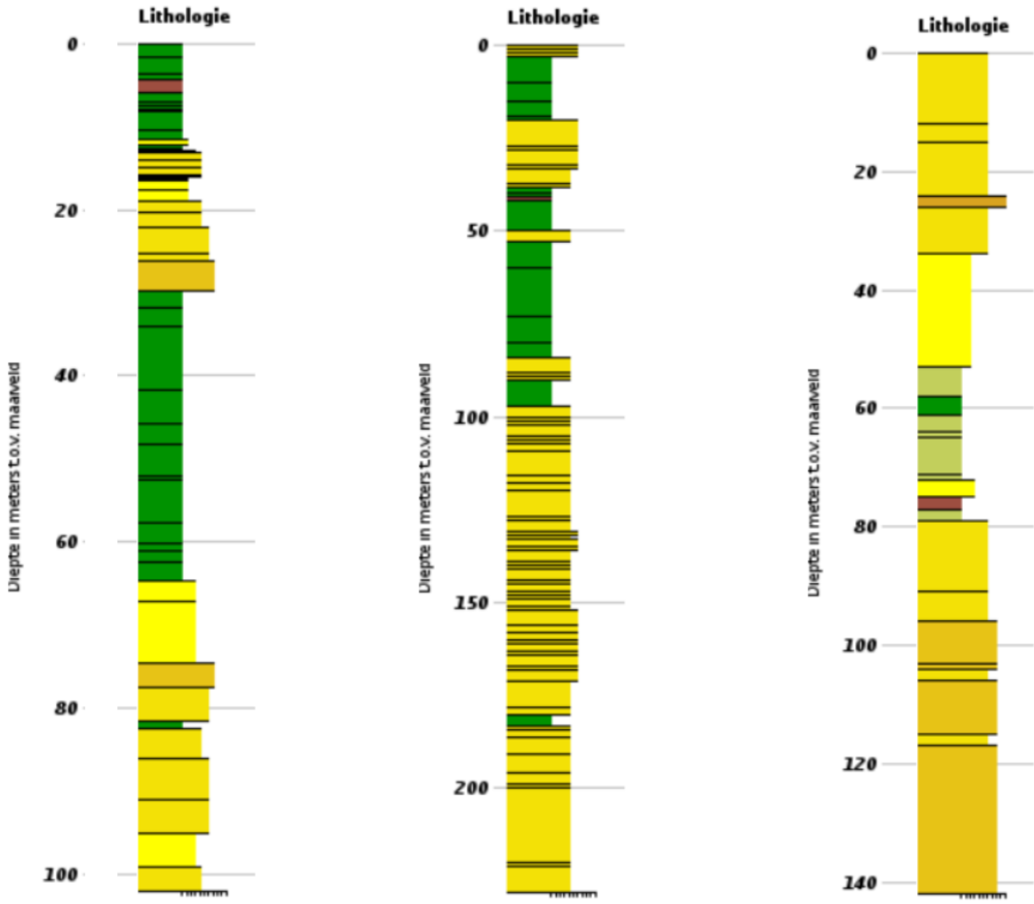


Figure B.6: Utrecht - shaft friction resistance

Drilling Sample Profiles

For clarification the total DSP of the locations are shown here.



(a) Amsterdam (b) Rotterdam (c) Utrecht

Figure B.7: Total drilling sample profiles

C

Stiffness of the Substructure

Validation Dummy Column

In order to verify if the dummy columns have the wanted springstiffness. A simple calculation is made with the program SCIA Engineer and the results are compared to the results of the hand-calculation. The shortening of the column is calculated with the following formula:

$$u_{hand} = \frac{F}{E * depth * width} * height \tag{C.1}$$

Table C.1: Properties of the dummy columns and the results of the column shortening due to an axial force of 15000 kN

k [MN/m]	E [MPa]	depth [mm]	width [mm]	height [mm]		u_{hand} [mm]	u_{SCIA} [mm]
100	1000	316.23	316.23	1000		150	150.2
200	1000	447.21	447.21	1000		75	75.1
300	1000	547.72	547.72	1000		50	49.9
400	1000	632.46	632.46	1000		37.5	37.6

Change of spring sequence

In the SCIA model the springs representing the stiffness of the soil and the stiffness of the foundation piles are swapped. The spring representing the stiffness of the foundation piles k_1 is underneath the spring which represents the stiffness of the soil k_2. In Figure C.1 the two different options are shown, with option 1 the sequence as in reality and in option 2 the spring of the soil on top of the spring of the foundation piles. The second option is favorable because it is easier to model in the program SCIA engineer.

To check if the deformation of the top is the same the k_1 is assumed as 100 MN/m, k_2 is assumed as 200 MN/m and F is 15000 kN. In option 1 the spring k_1 is modelled in the same way as the dummy column as shown in the previous paragraph and in option 2 the spring k1 is modelled as a flexible support with the springstiffness of 100 MN/m. In both option the spring k_2 is modelled as dummy column.

Table C.2: Deformation of the top due to the load F for the two options

	U_z [mm]
Option 1	225,3
Option 2	225,1

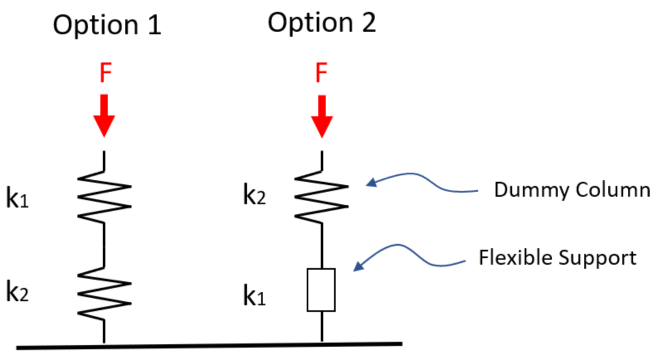


Figure C.1: Sequence of the springs

Foundation Pile plan for the Concrete Alternative

The number of foundation piles is determined based on two models. One model with the final settlement and one model with the settlement after 2 years. For the location of Amsterdam the representing stiffnesses are 300 MN/m and 600 MN/m.

The design value of the loadbearing capactiy at the location of Amsterdam is 3002 kN, as calculated in Paragraph 3.4.2.

The location of the piles are shown in Figure C.2.

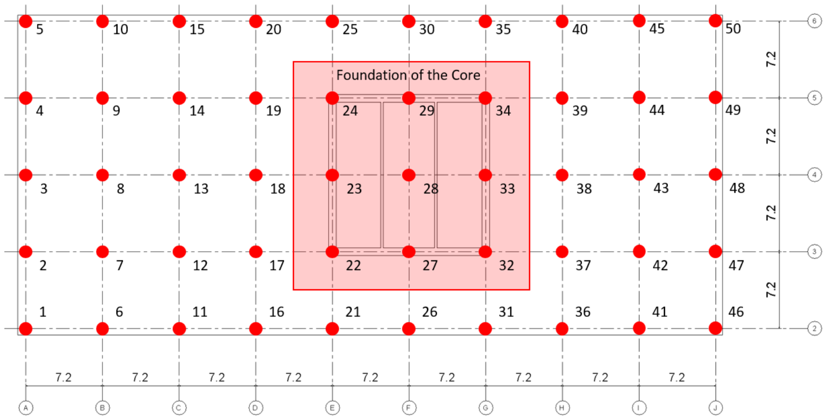


Figure C.2: The location of the foundation piles

Table C.3: The reaction forces used for the determination of the number of piles for the Concrete Alternative at the location of Amsterdam

		Amsterdam		
		Support Reaction Fz of ULS [kN]		
	Pilegroup	K = 300 MN/m	K = 600 MN/m	Number of Piles
1	P1	1032.64	1951.15	1
2	P1	2772.75	3612.42	2
3	P1	3301.74	3977.14	2
4	P1	2772.74	3612.41	2
5	P1	1032.65	1951.15	1
6	P1	4698.39	3604.32	2
7	P2	7827.39	6657.5	3
8	P2	8807.98	7454.92	3
9	P2	7827.44	6657.5	3
10	P1	4698.47	3604.33	2
11	P2	13151.99	10061.01	5
12	P2	18125.49	15592.83	7
13	P2	19660.02	17042.84	7
14	P2	18125.73	15592.95	7
15	P2	13152.36	10061.14	5
16	P3	28462.27	26616.95	10
17	P3	35768.61	36398.68	13
18	P3	38049.14	39240.18	14
19	P3	35769.32	36399.14	13
20	P3	28463.26	26617.47	10
21	P4	34681.25	31983.88	12
22	P6	49738.26	56072.73	19
23	P6	51037.17	58121.86	20
24	P6	49739.68	56073.82	19
25	P4	34682.81	31984.75	12
26	P4	34703.96	31634.27	12
27	P6	48968.24	55075.94	19
28	P6	49203.86	54880.89	19
29	P6	48969.79	55077.2	19
30	P4	34706.18	31635.6	12
31	P4	30701.6	27581.08	11
32	P6	45022.81	49333.22	17
33	P6	46217.61	51074.45	18
34	P6	45024.68	49334.77	17
35	P4	30706.53	27585.22	11
36	P2	20548.9	17541.61	7
37	P5	28460.12	27531.54	10
38	P5	30776.99	30192.07	11
39	P5	28464.87	27535.88	10
40	P2	20562.08	17555.87	7
41	P1	8464.72	6418.02	3
42	P2	13038.42	11200.13	5
43	P2	14512.32	12505.99	5
44	P2	13046.02	11207.2	5
45	P1	8487.21	6443.99	3
46	P1	216.69	841.72	1
47	P1	2575.2	2822.75	1
48	P1	3296.05	3230.63	2
49	P1	2582.28	2827.86	1
50	P1	238.96	864.71	1

D

From 3D to 2D model

For simplification a 2D model is made of the building to test the mitigating measures. In Figure D.1 the floorplan is shown with indicated the part which is modelled in the 2D model (the part between the green lines is modelled in the 2D model). The 2D model is also modelled in the program SCIA Engineer. In the 2D model the mitigating measures can be more easily applied and the effectiveness of the measures can be calculated more easily compared to the 3D model.

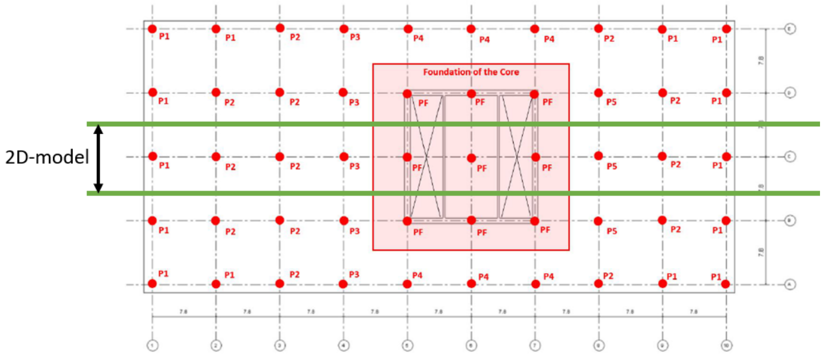


Figure D.1: Floorplan of the foundation floor with indicated which part of the structure is modelled in the 2D model.

In the 2D model not all the floors are modelled in order to shorten the calculation time. The stiffness of the floors has influence on the stiffness of the superstructure and therefore on the amount of displacement. The stiffness of the left out floors is added to the stiffness by increasing the thickness of the floors which are in the 2D model. The thickness of the floors is calculated with the following formula:

$$t_{equivalent} = n^{1/3} * t_{original} \tag{D.1}$$

With $t_{equivalent}$ is the equivalent thickness of the floor in the 2D model in m, n is the number of floors which the floor in the 2D model is representing and $t_{original}$ is the original thickness of the floors in the 3D model.

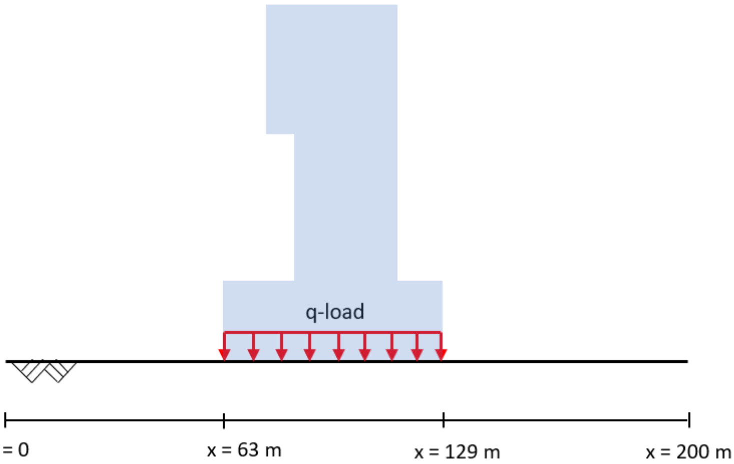
E

Problems to Adjacent Buildings

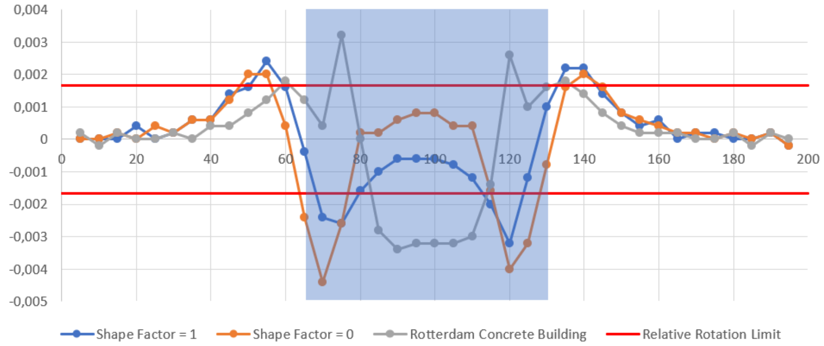
In this Appendix, the results of the calculations for exceeding the relative rotation limit of the adjacent buildings are shown.

Figure E.1 shows the relative rotation of the soil due to the weight of the building. The shape-factor defines the shape of the contact pressure [8]. When the shape factor is 0, the contact pressure is a parabolic distribution, which can be the result of a very stiff structure. When the shape factor is 1, the contact pressure is uniform over the plot of the building. In the result of Figure E.1, also the relative rotation of the concrete alternative in Rotterdam is shown. In this case, the highest contact pressure is happening under the core, and the pressure is lower to the edges.

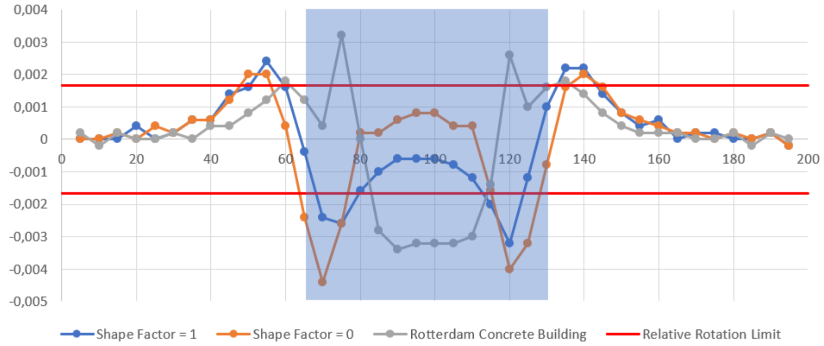
The relative rotation of the concrete building is touching the relative rotation limit in Figure E.1a, so only adjacent buildings in a range of 5-10 m to the building can experience damage based on this limit, the considered structure in this research and the soil structure used. The relative rotations of the cases with the shape factor as 1 or 0, exceeds this limit more. Therefore the adjacent buildings in a range of 15-25 meters can experience damage based on this limit. Only the distribution of the contact pressure will not be evenly distributed over the entire plot of the building. So, therefore this case is an exaggeration of the relative rotation.



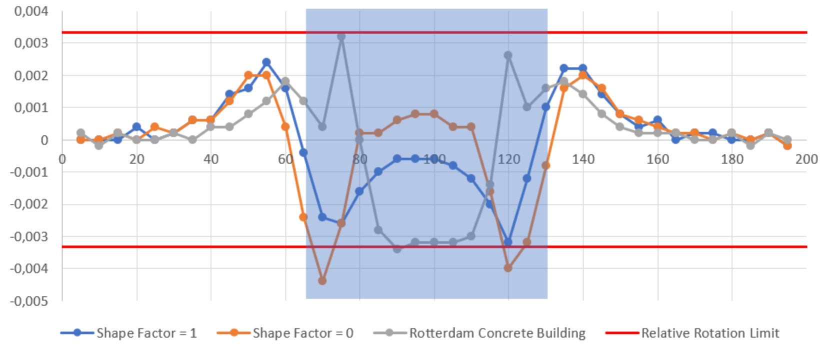
(a) q-load on the soil



(b) Relative rotation with the limit set on 1:1200



(c) Relative rotation with the limit set on 1:600



(d) Relative rotation with the limit set on 1:300

Figure E.1: Results of the calculation of the relative rotation next to the building