Pile Foundation and Soil Response to Deep Excavation

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PILE FOUNDATION AND SOIL RESPONSE TO DEEP EXCAVATION

Noord-Zuid Lijn Case Study

Msc. Thesis

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Ivan Stevanus Haryono
SUMMARY

Construction of deep excavation in densely populated area is not a trivial thing. The execution could induce disturbance to the soil and adjacent buildings. Many researches have analysed building responses to deep excavation or tunnelling projects. But the studies were mostly limited to buildings with shallow foundations.

The importance of understanding the behaviour of piled buildings is significant, especially in the analysis of buildings displacement. It is commonly found that deep excavation pits are situated close to piled buildings, especially in densely populated cities with soft subsurface. Even the buildings are founded on piles, there are no guarantees that the buildings are not affected by deep excavation activities.

Currently, a metro extension project in Amsterdam is being built. It passes the historic centre of the city and the buildings are founded on pile foundations. The project involved numerous amounts of measurements. Based on the measurement results, it has been found that due to the project, the buildings settle more than the foundation layer, but less than the surface settlement. It is suspected that the settlement of the buildings is influenced by the behaviour of the pile foundation.

This research is conducted to investigate and explain pile behaviour during the execution of deep excavation project. The focus of the study is at Ceintuurbaan station. The construction stages are modelled in 3-dimensional finite element analysis, in order to include the diaphragm wall installation phases. Before analysing the pile behaviour, back analysis of soil displacement is performed. After reasonable agreement in the soil displacement has been achieved, the pile behaviour has been analysed in more detail.

It is found that a reasonable agreement between measurement and simulation results can be obtained. Based on the analysis results, it is evident that the settlement of the building is influenced by several factors, including stress changes in the soil, neutral plane position, and the settlement of the soil adjacent to the pile. These factors are related interactively during each construction stage. The load on the pile also plays important role. Pile with higher load will experience larger settlement compared to pile with smaller load.

Based on the monitoring data and the 3D simulation results, the importance of each construction stage is demonstrated. Based on the analysis of monitoring data at Ceintuurbaan Station, it is shown that preliminary works could induce 31-59% of soil and building settlement. In the analysis of soil-structure interaction, it is important to take into account each construction stages and it should not only focus on the excavation stages.
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<th>Meaning</th>
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<tr>
<td>$A$</td>
<td>Area at the surface where surcharge load is applied (NEN9997-1+C1, 2012).</td>
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<tr>
<td>$a$</td>
<td>Effective tributary area (Zeevaert, 1973).</td>
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<tr>
<td>$A_b$</td>
<td>Area of the pile tip (NEN9997-1+C1, 2012).</td>
</tr>
<tr>
<td>$A_{shaft}$</td>
<td>Area of the pile shaft (NEN9997-1+C1, 2012).</td>
</tr>
<tr>
<td>$A_{4B}$</td>
<td>Loading area at 4B below the pile tip (NEN9997-1+C1, 2012).</td>
</tr>
<tr>
<td>$C_B$</td>
<td>The material constant.</td>
</tr>
<tr>
<td>$c_d$</td>
<td>Cohesion of the soil in drained condition (Zeevaert, 1973).</td>
</tr>
<tr>
<td>$c_i$</td>
<td>Cohesion of the embedded interface which is linked to the strength properties of the adjacent soil.</td>
</tr>
<tr>
<td>$C_c$</td>
<td>Compression index obtained from oedometer test.</td>
</tr>
<tr>
<td>$C_r$</td>
<td>Recompression index obtained from oedometer test.</td>
</tr>
<tr>
<td>$c_{soil}$</td>
<td>Cohesion of the soil.</td>
</tr>
<tr>
<td>$C_u$</td>
<td>Undrained shear strength of the soil.</td>
</tr>
<tr>
<td>$d$</td>
<td>Depth of the soil (Zeevaert, 1973).</td>
</tr>
<tr>
<td>$D_{pile}$</td>
<td>Diameter of the pile.</td>
</tr>
<tr>
<td>$d_j$</td>
<td>Thickness of the layer NEN9997-1+C1, 2012).</td>
</tr>
<tr>
<td>$E_{avg}$</td>
<td>Average soil elastic modulus at 4B below the pile tip (NEN9997-1+C1, 2012).</td>
</tr>
<tr>
<td>$E_{oed}$</td>
<td>Stiffness of the soil in 1D compression.</td>
</tr>
<tr>
<td>$E_{ref_{oed}}$</td>
<td>Reference stiffness of the soil in 1D compression.</td>
</tr>
<tr>
<td>$E_{pile,nom}$</td>
<td>Nominal value of elastic modulus of the pile (NEN9997-1+C1, 2012).</td>
</tr>
<tr>
<td>$E_{ur}$</td>
<td>Young’s modulus for unloading reloading.</td>
</tr>
<tr>
<td>$E_{ref_{ur}}$</td>
<td>Reference Young’s modulus for unloading reloading.</td>
</tr>
<tr>
<td>$E_{50}$</td>
<td>Secant modulus at 50% of failure stress.</td>
</tr>
<tr>
<td>$E_{ref_{50}}$</td>
<td>Reference secant modulus at 50% failure.</td>
</tr>
<tr>
<td>$e_0$ and $e$</td>
<td>Void ratio, in initial and final condition respectively.</td>
</tr>
<tr>
<td>$F_{found}$</td>
<td>Total load acting on the pile group (NEN9997-1+C1, 2012).</td>
</tr>
<tr>
<td>$F_{gemi}$</td>
<td>Average axial force acting on the pile (NEN9997-1+C1, 2012).</td>
</tr>
<tr>
<td>$F_{max}$</td>
<td>Maximum bearing capacity assigned on embedded pile.</td>
</tr>
<tr>
<td>$F_n$</td>
<td>Value of drag load.</td>
</tr>
<tr>
<td>$F_{ak,d}$</td>
<td>Design value for negative skin friction (NEN9997-1+C1, 2012).</td>
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<tr>
<td>$F_{ak,rep}$</td>
<td>Representative value for negative skin friction (NEN9997-1+C1, 2012).</td>
</tr>
<tr>
<td>$F_{tip}$</td>
<td>Mobilised force at embedded pile tip.</td>
</tr>
<tr>
<td>$F_{tot,j}$</td>
<td>Total load applied at the pile head (NEN9997-1+C1, 2012).</td>
</tr>
<tr>
<td>$G_{soil}$</td>
<td>Shear modulus of the soil.</td>
</tr>
<tr>
<td>$G_{ur}$</td>
<td>Shear modulus at elastic region.</td>
</tr>
<tr>
<td>$h$</td>
<td>Thickness of the consolidation layer (NEN9997-1+C1, 2012).</td>
</tr>
</tbody>
</table>
\( I_{zr} \) Influence factor in the calculation of negative skin friction in pile group (Zeevaert, 1973).

\( j \) Number of the layers (NEN9997-1+C1, 2012).

\( K_o \) Earth pressure coefficient.

\( K_{oc;j;k} \) Characteristic value of earth pressure coefficient (NEN9997-1+C1, 2012).

\( K_s \) Elastic shear stiffness of the embedded interface element.

\( K_{n}, K_i \) Elastic normal stiffness of the embedded interface element.

\( K_{tip} \) Elastic stiffness of the spring at the pile tip.

\( K_\phi \) Reduction factor at soil-pile interface.

\( L \) Pile length.

\( L \) Length of the pile above the ground level.

\( l_s \) Horizontal distance as specified in Reference source not found.

\( m \) Number of the layers where groundwater exists (NEN9997-1+C1, 2012).

\( m \) Factor of stress dependency (Hardening Soil Model).

\( m^* \) Shape factor of loaded area at 4B below the pile tip (NEN9997-1+C1, 2012).

\( n \) Number of the layers where negative skin friction acts.

\( NAP \) Normal Amsterdam Peil, reference level used in Netherlands.

\( O_{r;gem} \) Average pile diameter (NEN9997-1+C1, 2012).

\( P_o \) External load at the pile head (Flemming, 2008).

\( P_{sur;rep} \) Representative value of surcharge loading (NEN9997-1+C1, 2012).

\( P_n \) Additional load due to negative skin friction (Flemming, 2008).

\( P_u \) Ultimate allowable external load (Flemming, 2008).

\( q \) Deviatoric stress.

\( Q_u \) Ultimate bearing capacity of the pile (Flemming, 2008).

\( Q_b \) Base capacity of the pile (Flemming, 2008).

\( Q_s \) Shaft capacity of the pile (Flemming, 2008).

\( q_b \) Unit pile tip resistance (Flemming, 2008).

\( q_{b;max;j} \) Maximum unit pile tip resistance (NEN9997-1+C1).

\( q_{c;za} \) Cone resistance at depth \( z \) (NEN9997-1+C1, 2012).

\( q_{c;f;gem} \) Average cone resistance below the pile tip over 0.7-4.0 equivalent diameter (NEN9997-1+C1, 2012).

\( q_{c;f;F;gem} \) Average cone resistance following minimum path below the pile tip level over 0.7-4.0 equivalent diameter (NEN9997-1+C1, 2012).

\( q_{c;H;gem} \) Average cone resistance following the minimum path above pile tip level over a height of 8.0 equivalent diameter (NEN9997-1+C1, 2012).

\( q_{s;max;j} \) Maximum shear stress at pile shaft (NEN9997-1+C1, 2012).

\( R_{bc;j} \) Mobilised pile tip resistance (NEN9997-1+C1, 2012).

\( R_{bc;k} \) Characteristic value of pile tip resistance (NEN9997-1+C1, 2012).

\( R_{cd} \) Design value of total pile resistance (NEN9997-1+C1, 2012).

\( R_f \) Failure ratio.

\( R_{int} \) Interface strength reduction factor.

\( R_{s;k} \) Characteristic value of pile shaft resistance (NEN9997-1+C1, 2012).

\( S \) Spacing of the pile in group.
Factor of the pile tip cross section (NEN9997-1+C1, 2012).

$s_b$ Pile tip settlement obtained from load-displacement graph (NEN9997-1+C1, 2012).

$s_{el,i}$ Settlement of pile due to elastic compression of the pile (NEN9997-1+C1, 2012).

$s_p$ Total pile settlement.

$s_1$ Settlement of single pile (NEN9997-1+C1, 2012).

$s_2$ Settlement of pile group (NEN9997-1+C1, 2012).

$T_{max}$ Maximum shear stress.

$t_c$ The time to complete the primary consolidation.

$t_s$ Mobilised shear stress due to friction between pile and the soil.

$t_{n,t}$ Normal stress due to lateral displacement of the pile.

$t'$ The creep time.

$u_{n}^p, u_{t}^p$ Pile movement in lateral direction.

$u_{s}^p, u_{t}^s$ Soil movement in lateral direction.

$u_{n}^p$ Pile movement in axial direction.

$u_{s}^s$ Soil movement in axial direction.

$u_{tip}^p$ Displacement of the pile tip.

$u_{tip}^s$ Displacement of the soil at the pile tip.

$w$ Smallest width of the pile group size.

$Z$ or $z$ Depth from surface.

$z_{gw}$ Groundwater level NEN9997-1+C1, 2012).

$\alpha_p$ Factor related to type of the pile (NEN9997-1+C1, 2012).

$\alpha_s$ Material dependent factor for calculating shaft resistance (NEN9997-1+C1, 2012).

$\beta$ Factor related to the shape of the pile tip (NEN9997-1+C1, 2012).

$\delta_{hm}$ Maximum horizontal displacement.

$\delta_{j,k}$ Friction angle at the pile-soil interface (NEN9991-1+C1, 2012).

$\delta_v$ Ground surface settlement.

$\delta_{vm}$ Maximum ground surface settlement.

$\varepsilon_c$ Total strain due to primary consolidation.

$\varepsilon^H$ Logarithmic strain.

$\varepsilon_c^H$ Total logarithmic strain due to primary consolidation.

$\varepsilon_c^{He}$ Logarithmic strain in elastic part.

$\varepsilon_c^{Hc}$ Logarithmic strain in virgin area.

$\varepsilon_t$ The axial strain.

$\varepsilon_v^p$ Plastic volumetric strain.

$\gamma_b$ Partial factor for tip resistance (NEN9997-1+C1, 2012).

$\gamma_{f,sk}$ Partial factor in negative skin friction calculation (NEN9997-1+C1, 2012).
\( \gamma'_{jk} \) Characteristic value of effective soil unit weight (NEN9997-1+C1, 2012).

\( \gamma^p \) Plastic shear strain in Hardening Soil model.

\( \gamma_s \) Partial factor for shaft resistance (NEN9997-1+C1, 2012).

\( \gamma_{soil} \) Unit weight of the soil.

\( \phi_{CV} \) Critical state friction angle.

\( \phi', \phi_d \) Friction angle of the soil in effective stress condition.

\( \phi_i \) Friction angle of the embedded interface which is linked to the strength properties of the adjacent soil.

\( \phi_{jk} \) Characteristic value of friction angle in layer \( j \) (NEN9997-1+C1, 2012).

\( \phi_m \) Mobilised friction angle.

\( \phi_{soil} \) Friction angle of the soil.

\( \psi_{CV} \) Critical state dilatancy angle.

\( \psi_m \) Mobilised dilatancy angle.

\( \psi_p \) Peak dilatancy angle.

\( \sigma' \) Effective load pressure.

\( \sigma_{ref} \) Reference stress level.

\( \sigma_h \) Normal stress acting at the pile shaft (Zeevaert, 1973).

\( \sigma_{ave} \) Average normal stress at the pile shaft.

\( \sigma_{p0} \) Preconsolidation pressure before the loading.

\( \sigma_{pc} \) Achieved preconsolidation pressure after the loading.

\( \sigma_v \) Effective overburden stress of the soil.

\( \sigma'_{v,j;rep} \) Representative value of effective vertical stress at layer \( j \) (NEN9997-1+C1, 2012).

\( \sigma'_{v,j;m;rep} \) Representative value of effective vertical stress in layer \( j \) due to the effect of pile group (NEN9997-1+C1, 2012).

\( \sigma'_{v,j;sur;rep} \) Representative value of effective vertical stress due to surcharge loading (NEN9997-1+C1, 2012).

\( \sigma'_{v,4B} \) Effective stress induced by pile load at 4B below pile tip level (NEN9997-1+C1, 2012).

\( \sigma'_0 \) In situ effective stress of the soil.

\( \sigma_{0z} \) The initial stress acting at the pile shaft (Zeevaert, 1973).

\( \Delta \sigma_z \) The negative skin friction (Zeevaert, 1973).

\( \sigma'_3 \) Minor principal stress.

\( \tau_d \) The shear stress at the pile shaft in drained condition (Zeevaert, 1973).

\( \tau_s \) Unit shaft friction of the pile (Flemming, 2008).

\( \nu \) Poisson ratio.

\( \nu_{ur} \) Poisson ratio in elastic condition.

\( \Delta l \) Length of the pile below the surface where shaft resistance works (NEN9997-1+C1, 2012).
1. Introduction

1.1 Research Background

In many big cities in the world, the availability of space emerges as a common problem. Economy rises and cities grow rapidly, resulting in increasing population and increasing land price. City’s infrastructure development is often hampered by the availability of space in the area. As a result, interest in underground space development has significantly increased. Recently, there have been more and more tendencies for infrastructures, shopping malls, parking garages, etc to be constructed underground. Surely, these tendencies give better solution for future city development.

But, in a geotechnical engineering point of view, constructing underground facilities in closely spaced site is not a trivial issue. Deep excavation or tunnelling projects always induce some disturbance toward the adjacent area. On the other hand, at such densely populated cities, it is common that the density of the population and houses leaves only a small space between the underground construction site and the existing buildings. Therefore in most cases, disturbance against the adjacent buildings is inevitable.

Many attentions have been paid to the safety or the stability of the excavated area. Researches and innovations addressing this issue have been frequently produced and as a result, performance and execution of deep excavation projects are better nowadays. Nevertheless, more attention should be addressed to the building movement. Some observations suggest that even the safety and stability of the excavation has been insured, the adjacent buildings still showed damages in some cases.

Buildings affected by such activities were usually not designed to bear additional load triggered by soil movement found in deep excavation project. Such condition could lead to adverse effect and influence the performance of the buildings in its lifetime. Definitely, it is always desired that construction of underground space brings no damages to the adjacent buildings.

Researches on effect of the interaction of deep excavation and building movement have been initiated, but they were mainly intended for buildings with shallow foundation, for instance Potts and Addenbrooke [1997]. On the other hand, the response of building founded on pile foundations has not been thoroughly observed.

The importance of understanding the behaviour of piled buildings is also significant. It is commonly found that deep excavation pit is situated close to piled buildings, especially in densely populated cities with soft subsurface. Even the buildings are founded on piles, there are no guarantees that the buildings are not affected by deep excavation activities. An observation in a metro expansion project in Amsterdam, NoordZuid Lijn (North-South Line), suggests that even the buildings founded on piles, some displacements can be found.

The governing factor of the displacement is the behaviour of the pile foundation. It is of importance to understand the mechanism occurs in the pile foundation and the surrounding
soil when a deep excavation construction is executed. Therefore in the future, better understanding of this mechanism could lead to a better measure.

1.2 Problem Definition

Recently, there has been a metro expansion project in Amsterdam, which is called NoordZuid Lijn (North-South Line). The project needs to pass the old city centre where the buildings were founded on timber piles.

**Figure 1.1 Settlement at cross section 13110 east close to extensometer 06150401 at cross section 13110E**

**Figure 1.2 Settlement at cross section 13110 east close to extensometer 06150402 at cross section 13110E**
Due to the execution of the project, as observed in Figure 1.1 and Figure 1.2, the buildings settle down as the construction progressed. But interestingly, the settlement values lie between the settlement of the surface and the foundation layer (NAP -12m).

Normally, it might be expected that the settlement of the piled buildings is identical to the settlement of the foundation layer. Therefore, considering the aforementioned phenomenon, there should be some additional factors triggering the additional settlement.

It has been understood that Amsterdam subsurface experiences continuous settlement and this event causes the development of negative skin friction in the piles, even prior to the construction of the project. But the load distribution of the piles has never been monitored. Korff [2012] has conducted some analyses of the measurement data of NoordZuid Lijn. She postulated that the buildings settlement is closely related to the position of the neutral plane during the construction. Some assumptions were taken in the analysis, including that the negative skin friction and also the maximum pile resistance have been fully mobilised and they are maintained during the construction. An assumption of linear distribution for the soil settlement profile in the Holocene layers is also adopted to derive the neutral level due to the limited measurement points.

However, the truth of what occurs in the pile and the soil has not been entirely discovered with those assumptions. It is always expected that the response of the soil and the foundation adjacent to excavation can be thoroughly understood. Consequently, more accurate prediction can be derived in the future and damage of the building can be mitigated. A more detailed analysis such as finite element analysis, therefore, is required to reveal the fact.

1.3 Research Objectives

There are some researches that have been conducted earlier to analyse the interaction of pile, soil, and deep excavation in this particular case study, i.e Frankenmolen [2006], Kimenai [2011], Korff [2012]. From the results, some question marks still remain and it is expected that this study could give a clearer explanation.

The main objective of this study is to observe pile foundation behaviour subjected to deep excavation effect. This objective is expected to be better explained by answering the following questions:

- Are we able to model a deep excavation in a closely spaced urban area with piled buildings in a reasonable accuracy of the results?
- What is the triggering factor of the pile head settlement during the execution of deep excavation project?
- What has occurred in the pile load distribution during this construction period?
- How is the pile head settlement related to the soil settlement?
- What is the importance of each activity in the deep excavation project, i.e diaphragm wall installation and other preparation works?
- Do the piles experience bearing capacity problem during the excavation?
1.4 Methodology

The study will contain several stages. It consists of:

- Firstly, literature study is performed. The main focus of the literature study is the soil constitutive model, negative skin friction effect to the pile, and the calculation of pile resistance.
- Secondly, some overview and analysis of the measurement data from the project database will be conducted. The significance of each construction activity will be identified.
- Thirdly, 3-dimensional finite element analysis will be conducted to simulate the construction phases. The simulation initially will focus in the back analysis of the measured soil settlement. It is the first importance since the response of the pile will be governed by soil settlement. At the same time, piles will be modelled directly in the simulation. Once reasonable soil settlement results have been obtained, then the piles behaviour and the settlement adjacent to the project will be analysed. PLAXIS as commercial finite element analysis software is determined to be employed in the analysis. Pile foundation will be modelled using embedded pile feature which is available in the software. This feature is the extension of a beam feature accompanied by embedded interface to model pile-soil interaction.
- The study will be closed by providing some evaluations and answers to the research objectives mentioned above.

1.5 Scope of Work

In order to attain the objectives efficiently, several scopes are introduced. The main scope of the research will be summarised below:

- The case study is NoordZuid Lijn metro project in Amsterdam.
- The analysis and assessment will be based on effect of deep excavation around the station and the particular station studied in this research is Ceintuurbaan station.
- Buildings will be represented by a row of piles.
- The effect of pile cap will be disregarded in the pile settlement calculation.
- The analysis will overlook the structure’s influence (i.e weight and stiffness).
- The piles observed are driven piles, mainly timber piles, according to the actual buildings foundation around the station.

1.6 Thesis Outline

The thesis is divided into 7 chapters and it consists of:

- Chapter 1 provides the overview of the main problem, research objectives, methodology, and scope of work.
• Chapter 2 covers the theories used in the research, including basic understanding of negative skin friction, soil constitutive model, and embedded pile feature which will be used to model pile foundation.
• Chapter 3 presents the information and details about NoordZuid Lijn project. Moreover, the monitoring devices employed in the project will be discussed.
• Chapter 4 explains the observation results of the monitoring results. In this chapter, the discussion focuses on the influence of each excavation stage to the settlement of the buildings and the soils.
• Chapter 5 explains the hypotheses used as the basic concept of the analysis of pile foundation response.
• Chapter 6 contains the details of the three dimensional analysis that has been performed. Each analysis results in each construction stage will be discussed in details. The discussion covers the response of the soil and the pile foundation.
• The summary of the analysis results will be demonstrated in chapter 7. In the end, a conclusion and recommendation for future research are outlined.
2. Literature Review

This chapter will explain some of theoretical background of the subsequent analysis. Negative skin friction will be the first topic. It is understood that Amsterdam subsurface experiences continuous settlement, therefore, pile foundations in the area must have been subjected to drag load.

In relation to the subsequent finite element analysis, theory of soil constitutive model used in the simulation will be discussed. The theory will cover Hardening Soil and Soft Soil Creep model. Soft Soil Creep model is intended to be used to model the time dependent settlement occurred in Amsterdam, while Hardening Soil model is used for the excavation activities.

The pile foundation will be modelled using embedded pile. The basic of embedded pile model which will be employed to model pile foundation also will be explained.

Additional literature study might be presented in the later passages and discussions for a specific condition.

2.1 Negative Skin Friction on Pile

Cities situated in area dominated by soft soil, often encounter some problems with building foundation. Pile foundation, particularly end bearing pile, is commonly accepted as a reliable solution in this type of vicinity, especially for building with large load. Pile foundation enables the building to possess more bearing capacity and tackle the settlement problem. However, the problems are not completely solved.

In deltaic or soft soil area, the subsurface soil often experience significant consolidation settlement, due to external or self weight loading. Cities under excessive groundwater extraction might face similar problem. In such condition, pile will experience additional external load imposed by the soil displacement since the settling soil will force the pile to move down together. This event could mobilise pile resistance. Ultimately, not only changes in pile resistance will exist but also additional pile settlement will emerge.

To illustrate the load distribution in the pile subjected to negative skin friction, Figure 2.1 is presented below.
FIGURE 2.1 ILLUSTRATION OF NEGATIVE SKIN FRICTION (FLEMMING, 2008)

Where,

- $P_o$ is the external load at the pile head.
- $P_n$ is the additional load due to negative skin friction.
- $P_u$ is the ultimate allowable external load.
- $Q_u$ is the ultimate bearing capacity of the pile.
- $Q_b$ is the base capacity of the pile.
- $Q_s$ is the shaft capacity of the pile.
- $q_b$ is the unit base capacity of the pile.
- $\tau_s$ is the unit shaft friction of the pile.

Terzaghi and Peck [1948] can be regarded as the first engineers who reported the effect of negative skin friction. They observed that in Netherlands, thick layer of fill had been placed over the soft soil and the buildings, which were founded on end bearing piles, experienced some settlement. Since then, numerous reports and researches had been produced discussing about negative skin friction.

Starting in 1960s, some full scale tests were conducted as response of the increasing interest in this topic. These tests gave a new horizon for the understanding of negative skin friction concept. It was observed that negative skin friction occurred because the soil dragged the pile down during the consolidation period, when the excess pore pressure dissipated and the void ratio of the soil was reduced. Consequently, the effective stress increased and became the major factor governing the drag load magnitude.

Therefore, any activities which involve dissipation of excess pore pressure and consolidation close to pile foundation will lead to the development of negative skin friction. The causes of negative skin friction can be summarized as follows:

- On-going consolidation settlement of the soil adjacent to the pile due to surface loading.
  Bjerrum and Johannessen [1965] and Bjerrum et al [1969] showed that consolidation of soft clay in the surrounding vicinity of the piles induced shortening on pile length. After converting the pile length shortening to the axial load, evidently, it was
deduced that the positive shaft resistance acted reversely which is the main indication of negative skin friction.

- **Pile driving.**
  Zeevaert [1959] proposed a theory about the correlation of pile driving and negative skin friction. Zeevaert explained that due to pile driving, the soil will be displaced and particularly in clayey soil, the driving process increases the excess pore pressure in the soil at the outer diameter of the pile. Over the time, subsequently, the excess pore pressure dissipates and as a consequence, the effective stress of the soil in the outer diameter will increase. At the same time, consolidation will occur. Eventually, the consolidating soil will impose a drag load towards the pile, forcing it to settle down. Fellenius [1972] has confirmed this idea based on his experiment.

- **Lowering of groundwater level or excessive groundwater extraction.**
  In a full scale test by Endo et al [1969], an instrumented single steel pile with 43m length was installed into the soil with 37m of sandy silt, 4m of silt, and loose sand as the competent layer. At the moment of the test, water extraction from the sand layer was still on-going. This activity certainly created excess pore pressure and consolidation process in the upper layer, similar to Noordbergum effect (Verruijt [1969]). The Monitoring result showed that the pore pressure dissipated and settlement occurred. Due to this fact, negative skin friction was created. Thirty years later, Lee et al [1998] performed a centrifuge test to simulation a similar condition and it indicated similar tendency.

### 2.1.1 Design and Analysis Philosophy

Misconception in the design and analysis in pile foundation often occurs when negative skin friction is taken into account. The concept of negative skin friction as additional load to piles leads to a tricky understanding. Many engineers assume that negative skin friction may induce geotechnical capacity failure. Consequently, as outlined by Fellenius [1999], in the design or analysis of pile subjected to negative skin friction, engineers tend to reduce the geotechnical capacity with maximum drag load to obtain the ultimate geotechnical capacity.

Zeevaert [1959] also introduced similar concept that negative skin friction would reduce the bearing capacity of the piles. However at that moment he postulated the theory, there were no full scale test that had been performed to observe the real behaviour.

Van der Veen [1986], followed by Poulos [2008], denied this idea. They argued that geotechnical failure would only occur if the piles plunged into the soil. In that condition, shaft friction will resist the pile settlement and therefore, negative skin friction is vanished. Van der Veen also emphasized that the main problem in piles subjected to negative skin friction was the settlement of the pile.

Moreover, the other significant issue is the structural strength capacity of the pile. Due to the additional axial load, pile will potentially experience overstress condition which lead to structural failure, as explained by Chellis [1961], Kog [1987], and Davisson [1993].
2.1.2 Development of Negative Skin Friction

Currently available calculation methods can only explain the maximum value of drag load achieved when soil has reached fully consolidated condition. But, there are no explicit explanations in the formula to calculate the mobilised drag load. There were some debates about how fast drag load will accelerate with respect to the soil settlement amount.

In a full scale test by Swedish Geotechnical Institute, Bjerrum et al [1969] observed 2 different effects of soil settlement to the development of negative skin friction. In first condition in which fill was placed after pile installation, negative skin friction was fully mobilised when the soil settled 70-2000mm. The fully mobilised drag load varied from 1200-4000kN. In other condition, piles were installed at a site with no additional fill, and the soil only still experienced 1-2mm of settlement per year which can be considered as the residual settlement. Surprisingly, the measured drag load developed rapidly with only such a small ground settlement and it was approximately 2500kN.

The first condition has been supported by several observations made by others in a condition where surcharge loading was imposed. For example, Clemente [1981] reported that drag load still developed accompanied by excessive ground settlement which was higher than 1000mm. Leung et al [2004] performed a centrifuge test with similar scheme, and the result indicated that the drag load reached the maximum value after 960mm of soil settlement.

On the other hand, Fellenius [1972] opined that all piles experience negative skin friction. Means, that even with a small soil settlement, negative skin friction will develop. His idea was mainly inspired by his field experiment and also other observation from field tests by Bjerin [1977] and Bjerrum et al [1969].

Based on his field experiment observation, the ground settled 3mm in several months after pile driving, and such small settlement produced 300kN of drag load for 55m length of pile. Similar result was observed in laboratory test. Alonso et al [1984] conducted a direct shear test to measure the shear stress mobilisation at the pile-soil interface. He found that with 0.2mm relative displacement between soil and pile, 60% of drag load was mobilized. Ninety five percents of drag load was achieved by 3mm of relative displacement. This result is consistent with other laboratory tests performed by Clemence and Brumund [1975] and Chandler and Martins [1982].

The difference in drag load development between these 2 cases can be explained by the corresponding effective stress increase. Surcharge loading generally will generate large pore pressure, and as a consequence, the soil will experience longer time and large soil deformation to dissipate the excess pore pressure. This will delay the development of effective stress.

In the case of the installation of driven pile, shorter drainage path and relatively smaller excess pore pressure produced triggers faster consolidation process. As explained by Zeevaert [1973]. However, considering the installation method, such condition will only be limited to driven pile.

The cease of drag load development can be explained by the progressive failure phenomenon at the pile shaft. Jardine et al [2005] demonstrated that due to excessive strain and
rearrangement of soil fabric, strain softening occurs. In this moment, effective shear stress at pile-soil interface will also be reduced to residual value.

In order to calculate the drag load at a specific condition during the soil consolidation period, Poulos and Davis [1980] related the development of drag load to the degree of consolidation of the soil. An example of the correlation of degree of consolidation of the soil to the degree of drag load development can be observed in Figure 2.2.

![Figure 2.2 Degree of Drag Load Development Based on Degree of Consolidation of the Soil (Poulos and Davis, 1980)](image)

Based on his graphs, there are no influences of pile-soil stiffness ratio or length over depth ratio to the degree of development of drag load. Only the drainage path and consolidation parameter of the soil govern the development behaviour. This approach can be adapted for simple soil layers system.

### 2.1.3 Effect of Dead Load and Live Load to Negative Skin Friction Reduction

Dead load transferred at the top of the pile is believed can reduce or even cancel out the negative skin friction. This concept is based on the assumption that dead load will force the pile to settle relatively larger than the soil. Therefore, negative shaft friction can be reversed again.

However, a test result from Fellenius [1972] gives another perspective. It is revealed that even dead load has been added to the pile after negative skin friction developed, drag load kept emerging. Figure 2.3 presents the monitoring result.

In this field test, negative skin friction still developed in 495 days after pile installation. This event can be attributed to the pile installation effect. Later, 44 tons of dead load was applied. The previous drag load seemed to be stabilised, however, after some times, negative skin friction emerged again. Subsequently, the dead load was increased to 90 tons. Negative skin friction was eliminated, yet, it emerged again.
Based on centrifuge test, Leung et al [2004] found that in order to cancel out the negative skin friction, at least 3 times of maximum drag load should be applied as a dead load at the top of the pile as presented in Figure 2.4.

Therefore, based on these results, the aforementioned statement should be carefully considered. It is evident that elimination of negative skin friction by dead load depends on the ratio between the applied dead load and maximum drag load.

Different opinions exist among engineers with respect to the inclusion of live load to the analysis. Poulos [2008] stated that there is an expectation that incorporating live load in
negative skin friction calculation helps to reduce the negative skin friction acting on pile. However, he denied this assumption based on his calculation result. He suggested that a significantly large live load is required to reverse the shaft friction direction.

In more rigorous way, Shen [2008] observed the application effect of live load in his centrifuge test with 3 types of pile; end bearing pile, socketed pile, and floating pile. During the test, the transient live load was applied in 10 cycles. Interestingly, instead of eliminating negative skin friction, the applied live load generally buffered and increased the maximum drag load value temporarily. The drag load was recovered to the approximately original condition after the transient load has been released. A small remark can be deducted from the observation in floating pile, that is, the live load seemed to stabilize negative skin friction at the lower depth. Evidently, unlike the application of dead load, live load will hardly eliminate negative skin friction due to the transient effect. Figure 2.5 below summarises the centrifuge results.

![Figure 2.5 Influence of Transient Live Load Application to Negative Skin Friction (Shen, 2008)](image)

**2.1.4 Neutral Plane**

The illustration of neutral plane can be referred to Figure 2.1.
Buisson et al [1960] gave his opinion that the shear stress at the pile-soil interface is related to the shear strain and at a point, the shear strain will be gradually reduced to zero and no shear stress develops. This statement was confirmed by Endo et al [1969]. In the paper, it was mentioned that at a location called neutral plane, no relative displacement between the soil and pile exists. Neutral plane also shows the location in a pile where force equilibrium is reached and the negative skin friction will change to positive skin friction. Since the neutral plane indicates the inflection point of the shaft friction, that means that in this location the pile will experience maximum axial load at this location.

According to Leung et al [2004], in a pile which is socketed into dense sand, the neutral plane will be located at 90% of the length of the pile. On the other hand, neutral plane will be located much lower if the pile is constructed on a rigid stratum (i.e. hard rock). This observation has been presented by Bjerrum and Johannessen [1965], Leung et al [2004], and Shen [2008].

On the other hand, for a floating pile, shaft resistance is the only governing factor to obtain force equilibrium. Due to this fact, the neutral plane will be located shallower, sometimes close to the surface, and it depends on the mobilised shaft friction. This fact is supported by the tests performed by Shen [2008].

Provided that floating pile has higher position for neutral plane, therefore, the settlement induced in floating pile subjected to negative skin friction will be higher than the settlement in end bearing pile.

The main cause of this difference behaviour is induced by the preservation of resistance capacity of the pile. In floating pile, the resistance of the pile is dependent merely on the shaft resistance. Hence, during the exposure to soil settlement this type of pile will easily reach the maximum shaft friction, in which the pile relies the capacity on. Once this condition is reached, the neutral plane will be steadily located at the surface. On the other hand, in end bearing pile, even the soil pushes down the pile, the tip resistance of the pile assists the pile to resist. The disadvantage of this condition is the overstress in the pile.

2.1.5 Analytical Calculation Method for Negative Skin Friction

Terzaghi and Peck [1948] introduced the first method to calculate negative skin friction on the basis of the overburden pressure of the soil. Later, starting in 1960s, this concept was refined by many engineers.

It is worth to mention that the current calculation method used recently can also be attributed to the work from Zeevaert [1959]. In his book (Zeevaert [1973]), where he elaborated the theory, he presented the approach to calculate negative skin friction based on the effective stress of the soil.

Based on his theory, negative skin friction occurs due to the reduction of effective stress along the pile shaft and tip. The relief of stress at pile shaft triggers the load to increase. On the other hand at the pile tip, it even lessens the tip bearing capacity of the pile, which is denied by many engineers later. Furthermore, he also introduced that when pile-soil interface is
subjected to a stress, the adhesion will fail before the stress reaches maximum shear stress of the soil. That is the moment when slip occurs and negative skin friction cannot develop further. To incorporate this condition, Zeevaert [1959] introduced a reduction factor to the horizontal effective stress calculation. The concept is recently called as slip-method.

Zeevaert [1959] theory was consistent with the results from field tests, for example, Bjerrum and Johannessen [1965], Bjerrum et al [1969], and Bozozuk [1972]. In these results, it was revealed that negative skin friction developed proportionally to effective stress of the soil with a certain constant value.

The formula based on the effective stress analysis is concisely presented below. This approach is known as β-method. The term \( F_n \) represents the negative skin friction.

\[
F_n = \frac{\tan \delta}{\tan \varphi'} \tan \varphi' K_o \sigma_v' = \beta \sigma_v'
\]

(2.1)

Where,

- \( F_n \) is the value of negative skin friction.
- \( \delta \) is the pile-soil interface friction angle.
- \( \varphi' \) is the friction angle of the soil in effective stress condition.
- \( K_o \) is the earth pressure coefficient.
- \( \sigma_v' \) is the effective overburden stress of the soil.

Aside from the aforementioned effective stress approach, some engineers prefer to conduct calculation based on total stress approach, which is commonly known as α-method. In this method, a reduction factor \( (\alpha) \) is also employed to the formula, which is described below:

\[
F_n = \alpha C_u
\]

(2.2)

\( C_u \) is the value of the actual undrained shear strength. This method could be wisely used if the actual undrained shear strength of the soil is known. However, it is understood that the undrained shear strength of the soil changes in conjunction with the time and construction condition. Therefore, the concept of this method limits the capability to calculate the final or maximum drag load, unless the future undrained shear strength after consolidation has been predicted or investigated.

Unlike β–method, it can be concluded from many researches that the reduction factor \( \alpha \) scatters sporadically. Many attempts to determine the value and find a regular pattern have been conducted, but still the factor spread in a large range in a different soil types.

On the other hand, β–method performs more consistently. Bjerrum et al [1969] concluded that \( \beta \) varied between 0.20 and 0.25 for soft silty clay. Bjerrin [1977] reported 0.20 – 0.25 for clay with firm to medium consistency. Garlanger [1974] suggested \( \beta = 0.20 - 0.25 \), while Burland [1973] suggestion was 0.25. A recent centrifuge result performed by Leung et al [2004] recommended a value of 0.25. Evidently, β–method can be held as a firm criterion and more conveniently implemented, rather than α–method, which is prone to errors in the parameter determination and calculation.
As another alternative, Poulos and Davis [1980] proposed an elastic solution for end bearing pile using boundary element method, derived from point load in elastic half space concept (Mindlin [1936]). In this method, the soil is assumed as an elastic continuum and the pile-soil interface is considered semi-infinitely elastic. Soil displacement is imposed directly to the nodes and no consideration of slip failure at pile-soil interface is taken. As a consequence due to the semi-infinite linear interface, the shear stress produced by this method can be exaggerated and unrealistic compared to the real condition.

An improvement was introduced by Kuwabara and Poulos [1989] by including slip at the pile-soil interface. The analysis result was compared with field measurement and it turned out that slip-method gave closer result. Lee et al [2002] also performed another comparison with finite element analysis with slip method and it has been demonstrated that elastic solution’s result overestimated the drag load.

Based on these facts, slip method with effective stress approach is widely accepted. Many researchers and national codes adopt this method, for example NEN9997-1+C1 (Dutch Standard based on Eurocode), CP 4: 2003 (Singapore Standard), and Hongkong’s Foundation Design and Construction Standard.

NEN9997-1+C1 has presented different formulas for the calculation of pile subjected to negative skin friction in single arrangement or in a group. However, in relation to the observed case, only analytical solution based on single pile arrangement will be outlined here.

The analytical solution suggested by NEN9997-1+C1 adopts effective stress analysis, which is close to the Zeevaert’s analysis. Therefore, before discussing NEN9997-1+C1, it is worthwhile to revisit the approach proposed by Zeevaert.

**Zeevaert [1973] Calculation Method**

Zeevaert opined that horizontal stress from the soil at any depth of the pile will be proportional with a certain ratio to the vertical stress. He assumed that the stress development at the shaft could be described by linearly elastic and perfectly plastic behaviour, according to Mohr-Coulumb criterion.

The derivation of his concept is started from the situation of the soil after a pile has been driven. Because the soil is forced to displace, it will be remoulded and excess pore pressure will develop. This condition leads the soil to consolidate. Normally, in the case where relatively no disturbance occurs during pile installation (i.e. on bored pile), the ultimate shear strength at the pile shaft is described as:

\[
\tau_d = c_d + \sigma_h \tan \phi_d
\]  \hspace{1cm} (2.3)

Where,

- \(\tau_d\) is the shear stress at the pile shaft in drained condition.
- \(c_d\) is the cohesion of the soil in drained condition.
- \(\sigma_h\) is the normal stress at the pile shaft.
- \(\phi_d\) is the friction angle in drained condition.
Observing stress state at pile shaft in Figure 2.6. Reference source not found. and using ultimate stress condition in Mohr-Coulomb criterion, the following relationship is obtained.

\[
\frac{\sigma_h}{\sigma_v} = \frac{(\sigma_1 + \sigma_3) - (\sigma_1 - \sigma_3) \cos 2\alpha}{(\sigma_1 + \sigma_3) + (\sigma_1 - \sigma_3) \cos 2\alpha}
\]  

(2.4)

In the soil’s ultimate condition in Mohr-Circle, \(2\alpha\) is equal to \(\varphi_r + \frac{\pi}{2}\). After \(\sin \varphi_r = \frac{(\sigma_1 - \sigma_3)}{(\sigma_1 + \sigma_3)}\) is incorporated to equation 2.4, it is found that:

\[
\frac{\sigma_h}{\sigma_v} = 1 - \sin^2 \varphi_r = \frac{1}{1 + \sin^2 \varphi_r} = \frac{1}{N_\varphi}
\]  

(2.5)

Then, shaft friction can be explained by:

\[
\sigma_h = \left(1 - \sin^2 \varphi_r \right) \sigma_v = K_\varphi \sigma_v
\]  

(2.6)

Where, \(\tan \varphi_r\) is the coefficient of interaction at the pile-soil interface and the reduction factor is \(K_\varphi = \frac{\tan \varphi_r}{N_\varphi}\). 

**Figure 2.6** Stress at pile shaft (Zeevaert, 1973)
In Zeevaert’s concept, negative skin friction causes the reduction of shear strength at the pile-soil interface, this concept is illustrated in Figure 2.7. Based on the picture, negative skin friction value can be concisely described as:

\[ F_n = 2m_0 \int_0^d K_o (\sigma_{0z} - \Delta \sigma_z) \, dz \]  \hspace{1cm} (2.7)

\( r_0 \) is the radius of the pile, \( \sigma_{0z} \) is the initial stress acting at the pile shaft, and \( \Delta \sigma_z \) is the stress relief which is equal to negative skin friction (the shaded area in Figure 2.7).

Based on Figure 2.7, the vertical stress relief is calculated by:

\[ \Delta \sigma_z = \frac{1}{3} \Delta \sigma_d \, d \]  \hspace{1cm} (2.8)

Where \( d \) is the depth of the soil considered in the calculation.

Zeevaert assumed that the value of negative skin friction is equivalent with the effective tributary area \((a)\) of a pile. The effective tributary area can be regarded as the mobilised soil weight which contributes to drag the pile down. Obviously, it increases with depth proportionally with the increased soil weight. As a rough estimation, Zeevaert suggested that
can be assumed as a constant equal to \( \pi (12r_0)^2 \) for the entire depth. Hence, this theory leads to:

\[
\Delta \sigma_d = \frac{F_n}{a} \tag{2.9}
\]

Subsequently, if equation 2.9 is combined with equation 2.7, the value for negative skin friction for single pile can be obtained by:

\[
F_n = \frac{2\pi_0 K_r}{1 + \frac{d}{3} \frac{\pi (12r_0)^2}{12}} \int_0^d \sigma_{0z} dz \tag{2.10}
\]

It should be noticed that this solution is derived based on constant effective tributary area.

The importance of effective tributary area can be observed when negative skin friction calculation on pile group is required. The concept helps us to understand the phenomenon of reduced negative skin friction in piles which are closely installed.

In a pile group with a close spacing, the effective tributary area of each pile will overlap to each other, and consequently, the effective tributary area of each pile will be reduced, compared to single pile. Aside from the overlap, in the calculation of effective tributary area, Zeevaert also considered that the position of a pile in a group and the existence (amount) of the neighbour piles will also influence the value of negative skin friction for each pile. He named the influence factor as \( I \). As illustrated in the following figure, the effective tributary area of pile O will be determined, including the influence factor (\( I_{xz} \)) from the corner pile.

![Figure 2.8 Influence of Pile Group to Pile O (Zeevaert, 1973)](image)

In order to obtain the influence factor of other pile to pile O in Figure 2.8, let’s consider the vertical stress reduction at a distance \( r \) in Figure 2.9. Employing the concept from Terzaghi [1943] article 143 and Westergaard’s stress distribution, the vertical stress at a distance \( r \) can be obtained. Consequently, adapting the same way, the effect of the corner pile to the effective tributary area of pile O can also be calculated.
FIGURE 2.9 Vertical stress reduction due to friction at the pile shaft (Zeevaert, 1973)

FIGURE 2.10 Influence factor for 2 piles for calculation of effective tributary area (Zeevaert, 1973)
Zeevaert has compiled a graph in Figure 2.10, to easily calculate $I_{cz}$. As observed in the graph, there are 2 lines exist. These lines represent 2 different condition of negative skin friction. Zeevaert developed the graph based on the condition when negative skin friction is constant through the entire depth, and the other case is negative skin friction gradually increases with depth. The former case corresponds to $I_{cz/c}$, on the other hand, the latter case corresponds $I_{cz/k}$. Observing the current researches results, it is more relevant to calculate effective tributary area based on the second case.

The effective tributary area is assumed as a circular area and calculated based on the following equation.

$$a_e = \frac{\pi z^2}{\sum_n I_{cz/k}}$$  \hspace{1cm} (2.11)

Where, $z$ is the observed depth and $n$ is the number of the piles.

**NEN9997-1+C1 [2012] Calculation Method**

NEN9997-1+C1 is used as Dutch national standards to calculate geotechnical structures. NEN9997-1+C1 provides a guideline to calculate negative skin friction in chapter 7.3.2.2 (in the code). The code recommends that negative skin friction should be included in the pile load if the settlement of the soil reaches 0.02m and maximum drag load will be achieved after the settlement of the surrounding soil is 0.1m. It also recommends that negative skin friction should be addressed in serviceability condition, in accordance to chapter 2.1.1.

The code provides different methods to calculate negative skin friction on single pile and pile in a group. The determination of which method should be conducted depends on the spacing of the piles, thickness where negative skin friction works, and the diameter of the piles.

Consider a group of end bearing piles with spacing of $S$, diameter of $D$, and the consolidating layer thickness of $h$. It is indicated that if the spacing of the pile is larger than $\sqrt{100Dh}$, then calculation needs to be conducted based on single pile condition. Otherwise, the other method should be applied.

In serviceability condition, a partial factor $\gamma_{f,nk}$ is used, forming the following equation:

$$F_{nk,ld} = \gamma_{f,nk} F_{nk,rep}$$  \hspace{1cm} (2.12)

$F_{nk,rep}$ is the representative value of negative skin friction, on the other hand, $F_{nk,ld}$ is the design value of negative skin friction.

In a single pile, a partial factor from 1.0-1.4 can be taken, while, a partial factor of 1.2-1.4 is applied in pile group.

**Negative Skin Friction on Single Pile**

The formula for negative skin friction acting on single pile is concisely shown as:
The terms used in the equations above are explained as follows.

- $O_{x,\text{gem}}$ is the average pile diameter.
- $d_j$ is the thickness of the layer.
- $j$ is the number of the layers.
- $n$ is the number of the layers where negative skin friction acts.
- $m$ is the number of the layers where groundwater exists.
- $\delta_{j,k}$ is the characteristic value of friction angle at the pile-soil interface, which is $\tan\delta_{j,k}$.
- $\varphi_{j,k}$ is the characteristic value of friction angle in layer $j$.
- $\sigma'_{v,j,\text{rep}}$ is the representative value of effective vertical stress at layer $j$.
- $\gamma'_{j,k}$ is the characteristic value of effective soil unit weight.
- $z_{gw}$ is the groundwater level.
- $P_{\text{sur,rep}}$ is the representative surcharge loading.

The value of $\delta_{j,k}$ is determined by $0.75\varphi_{j,k}$ for displacement pile and $\delta_{j,k} = \varphi_{j,k}$ for bored pile. The minimum value of $K_{\alpha,j,k}\tan\delta_{j,k}$ is set as 0.25.

**Negative Skin Friction on Pile Group**

Negative skin friction in pile group is calculated based on the following formula:
\[ F_{sk,rep} = A \sum_{j=1}^{n} (\sigma'_{v,j;sur,rep} - \sigma'_{v,j;m,rep}) \]  
(2.18)

Where:

\[ \sigma'_{v,j;sur,rep} = \sigma'_{v,j-1;m,rep} + d_j \gamma'_{j;k} \]  
(2.19)

For the first layer, it applies:

\[ \sigma'_{v,j-1;m,rep} = P_{sur,rep} \]  
(2.20)

While, for the deeper layer:

\[ \sigma'_{v,j;m,rep} = \frac{\gamma'_{j;k} A}{m_j} (1 - \exp(-m_j d_j)) + \sigma'_{v,j-1;m,rep} \exp(-m_j d_j) \]  
(2.21)

The terms are explained as follows:

- \( A \) is the area at the surface where surcharge load is applied.
- \( \sigma'_{v,j;sur,rep} \) is the representative value of effective vertical stress due to surcharge loading.
- \( \sigma'_{v,j;m,rep} \) is the representative value of effective vertical stress due to the effect of pile group.

The calculation of negative skin friction for piles in a group based on this method, does not explicitly explain how to derive the value of \( A \). In regular pile group especially for the centre pile, \( A \) can be obtained simply by calculating the area where the surcharge load is imposed to the pile. But, in irregular pile group, or if engineers should deal with the peripheral and corner piles, the value of \( A \) will be need to be determined differently. Therefore in this case, the effective tributary area proposed by Zeevaert [1973] is beneficial.

### 2.2 Pile Resistance and Settlement Calculation Method

As outlined in chapter 2.1.4, pile resistance holds an important factor in determining the maximum drag load value. Hence, the calculation method for pile resistance will be highlighted in this chapter. The calculation method presented here is adopted from NEN9997-1+C1 chapter 7.6.2, as Dutch guideline for pile foundation design.

NEN9997-1+C1 has already included Eurocode 7 guideline. Compared to the previous code, NEN6743, the total pile resistance calculation is slightly changed. The values of partial factors and other factors will not be presented here and should be directly referred to the code. The partial factors can be obtained in the national annex (NEN-EN 1997-1/NB, 2008). The total pile resistance formula given by the code is:

\[ R_{v,d} = \frac{R_{b,k}}{\gamma_b} + \frac{R_{s,k}}{\gamma_s} \]  
(2.22)
Where, \( R_{\text{cd}} \) is the designed total pile resistance, \( R_{b,k} \) is the characteristic pile tip resistance, and \( R_{s,k} \) is the characteristic shaft resistance. \( \gamma_b \) and \( \gamma_s \) indicate the partial factor for tip and shaft resistance respectively.

The basic of pile tip and shaft resistance calculation method in the code adopts the approach which was initiated by van Mierlo and Koppejan’s method [1952] and later, it was improved by de Ruiter and Beringen [1979].

For pile tip resistance, the following equation is applied:

\[
R_{b,k} = A_b q_{b,\text{max},j}
\]

(2.23)

\[
q_{b,\text{max},j} = \frac{1}{2} \alpha_p \beta \left( \frac{q_{c,\text{I gems}} + q_{c,\text{II gems}}}{2} + q_{c,\text{III gems}} \right)
\]

(2.24)

Where,

- \( A_b \) is the area of the pile tip.
- \( q_{b,\text{max},j} \) is the maximum unit tip resistance.
- \( \alpha_p \) is the factor related to type of the pile, it is determined based on 7.6.2.3(f).
- \( \beta \) is the factor related to the shape of the pile tip, it is determined based on 7.6.2.3(g).
- \( s \) is the factor of the pile tip cross section, it is determined based on 7.6.2.3(h).
- \( q_{c,\text{I gems}} \) is the average cone resistance below the pile tip over 0.7-4.0 equivalent diameter, according to the Dutch averaging method and it is illustrated in Figure 2.11.
- \( q_{c,\text{II gems}} \) is the average cone resistance following minimum path below the pile tip level over 0.7-4.0 equivalent diameter, according to the Dutch averaging method and it is illustrated in Figure 2.11.
- \( q_{c,\text{III gems}} \) is the average cone resistance following the minimum path above pile tip level over a height of 8.0 equivalent diameter, according to the Dutch averaging method and it is illustrated in Figure 2.11.

![Figure 2.11 Dutch Averaging method (Xu, 2007)](image-url)
For the shaft resistance, the governing equation is:

\[ R_{s;k} = O_{s;\Delta;\text{gem}} \int_{\Delta} q_{s;\text{max}ij} \, dz \quad (2.25) \]

Where, \( O_{s;\Delta;\text{gem}} \) is the circumference of the pile shaft and \( q_{s;\text{max}ij} \) is the maximum pile shaft resistance and it can be concisely determined from:

\[ q_{s;\text{max}ij} = \alpha_s q_{c;\text{z}ij} \quad (2.26) \]

It is understood that some displacements are required to mobilise the pile resistances. The former calculation methods are limited to determine the maximum pile resistance. Therefore, some load-displacement relationships for different type of piles have been provided in NEN9997-1+C1 to calculate the mobilised pile resistances, as indicated in Figure 2.12.

\[ \text{FIGURE 2.12 LOAD-DISPLACEMENT GRAPH PROVIDED BY NEN9997-1+C1 FOR PILE TIP (LEFT) AN PILE SHAFT (RIGHT)} \]

Line 1 in the graph is to represent the load-displacement graph for ground displacement piles (driven piles), while, line 2 corresponds to auger piles with little disturbance, and line 3 is proposed for bored pile.

The total settlement of the pile is calculated by:

\[ s_p = s_1 + s_2 \quad (2.27) \]

Where,

\( s_1 \) is the settlement of single pile, as the summation of pile tip settlement (\( s_{ij} \)) which is obtained from load-displacement graph and elastic compression of the pile (\( s_{el;} \)).

\( s_2 \) is the settlement of the pile group (if the pile group is used).

The elastic compression of the pile is calculated based on:
\[ s_{el,j} = \frac{L F_{genn,j}}{A_{shaft} E_{pile,nom}} \] (2.28)

With:

\[ F_{genn,j} = \frac{IF_{tot,j} + 0.5\Delta l(F_{tot,j} + R_{b,j})}{L} \] (2.29)

Where:

- \( L \) is the length of the pile.
- \( F_{genn,j} \) is the average axial force acting on the pile.
- \( A_{shaft} \) is the area of the pile shaft.
- \( E_{pile,nom} \) is the nominal pile elastic modulus.
- \( l \) is the length of the pile above the ground level.
- \( F_{tot,j} \) is the total load applied at the pile head.
- \( \Delta l \) is the length of the pile below the surface where shaft resistance works.
- \( R_{b,j} \) is the mobilised pile tip resistance.

To calculate the settlement of the pile group, the following equation is applied.

\[ s_2 = \frac{m^* \sigma'_v;4w 0.9 \sqrt{A_{4w}}}{E_{ea,avg}} \] (2.30)

\[ \sigma'_v;4w = \frac{F_{found}}{E_{ea,avg}} \] (2.31)

With:

- \( w \) is the smallest width of the pile group size.
- \( m^* \) is the shape factor of loaded area at 4B below the pile tip, determined based on table 7.f.
- \( \sigma'_v;4w \) is the effective stress induced by pile load at 4B below pile tip level.
- \( F_{found} \) is the total load acting on the pile group.
- \( A_{4w} \) is the loading area at 4B below the pile tip.
- \( E_{ea,avg} \) is the average soil elastic modulus at 4B below the pile tip.

### 2.3 Embedded Pile Model

Earlier, modelling pile-soil interaction in finite element method can be done by some alternatives. Generally, the most representative condition is modelling the pile using a volume of non porous material and with certain strength properties. The application of such model may provide a convenient solution, particularly for single pile modelling. However, in fact, it is inevitable that a complex meshing is needed within the structure of the pile material. Clearly, in a project involving numerous piles, calculation of soil-pile interaction will take unexpected long time and it may lead to inaccuracy in the calculation. In order to
tackle these issues, PLAXIS provides a special feature in pile modelling, namely embedded pile feature. The initial development was implemented in PLAXIS3D. The principle of the model will be explained in the paragraphs below.

However, it should be noted that installation effect in driven pile has not been taken into account in the model and this might add to the limitation. Augered pile or bored pile will be more appropriate to be simulated by this model.

Embedded pile model is actually the extension of beam element. It can be arbitrarily put in the soil and cross it at any depth and orientation. In the model, embedded pile generates geometry line and a volume of elastic zone to manipulate the real volume of the pile. However, only uniform shape along the pile length can be applied. Pile-soil interaction is taken into account by introducing embedded interfaces at the shaft and the tip of the pile. Figure 2.13 gives the illustration of embedded pile model.

The behaviour of the interface of the pile and the soil at the shaft in axial direction is governed by linear-elastic model with finite strength, represented by spring and slide. On the other hand, the behaviour of the pile in lateral direction is only dictated by spring, which leads only to elastic behaviour.

The constitutive model for the shaft is explained by:
\[
\begin{bmatrix}
    t_s \\
    t_n \\
    t_t
\end{bmatrix} =
\begin{bmatrix}
    K_s & 0 & 0 \\
    0 & K_n & 0 \\
    0 & 0 & K_t
\end{bmatrix}
\begin{bmatrix}
    u_s^p - u_s^s \\
    u_n^p - u_n^s \\
    u_t^p - u_t^s
\end{bmatrix}
\]

(2.32)

Where,
- \( t_s \) is the mobilised shear stress due to friction between pile and the soil.
- \( t_n, t_t \) is the normal stress due to lateral displacement of the pile.
- \( K_s, K_t \) is the elastic shear stiffness of the embedded interface element.
- \( u_s^p \) is the pile movement in axial direction.
- \( u_n^p, u_t^p \) is the pile movement in lateral direction.
- \( u_s^s \) is the soil movement in axial direction.
- \( u_n^s, u_t^s \) is the soil movement in lateral direction.

By default, the stiffness component is determined by:

\[ K_s >> G_{soil} \]  

(2.33)

\[ K_n = K_t = \frac{2(1-\nu)}{1-2\nu} K_s \]  

(2.34)

Where, \( G_{soil} \) is the shear stiffness of the soil, and \( \nu \) is the poisson ratio of the soil.

The finite strength in axial direction at the pile shaft is defined by applying a limiting shear stress \( T_{\text{max}} \), which is arbitrarily inputted by the users. By this means, slip will be allowed if the maximum stress condition is reached. Later, the behaviour will remain plastic with no additional stress developed even the relative displacement of the pile and the soil increases.

Constitutively, the shear stress at the shaft will remain elastic in the following condition:

\[ |t_s| < (c_i + \sigma_{n}^{\text{ave}} \tan \varphi_i) \pi D_{\text{pile}} \]  

(2.35)

and

\[ |t_s| < T_{\text{max}} \]  

(2.36)

Where,
- \( c_i \) is the cohesion of the embedded interface which is linked to the strength properties of the adjacent soil.
- \( \varphi_i \) is the friction angle of the embedded interface which is linked to the strength properties of the adjacent soil.
- \( \sigma_{n}^{\text{ave}} \) is the average normal stress at the pile shaft. In case of vertical pile, it is equal to the average of normal stress in xx and yy direction.
\( D_{\text{pile}} \) is the pile diameter.

The shear strength properties of the interface is associated with the shear strength of the soil by means of strength reduction factor, \( R_{\text{int}} \). Hence:

\[
c_i = R_{\text{int}} c_{\text{soil}} \\
\phi_i = R_{\text{int}} \phi_{\text{soil}}
\]

(2.37) \hspace{1cm} (2.38)

The stress in the plastic behaviour is governed by equation 2.35, provided that the value of \( |f_s| < T_{\text{max}} \).

The response at the pile tip is also dictated by elastic-perfectly-plastic behaviour, where a bearing capacity, denoted as \( F_{\text{max}} \), is an input. with the elastic stiffness is denoted as \( K_{\text{tip}} \). The mobilisation of the pile tip capacity also depends on the relative displacement between the soil and the pile. But, unlike the force at the shaft, no tension force (negative value) as at the pile shaft is allowed. The following equation governs the constitutive model at the pile tip.

\[
F_{\text{tip}} = K_{\text{tip}} (u_{\text{tip}}^p - u_{\text{tip}}^s) \leq F_{\text{max}}
\]

(2.39)

Where,

\( F_{\text{tip}} \) is the mobilised force at the pile tip.
\( K_{\text{tip}} \) is the elastic stiffness of the spring at the pile tip.
\( u_{\text{tip}}^p \) is the displacement of the pile tip.
\( u_{\text{tip}}^s \) is the displacement of the soil at the pile tip.

Elastic region with the diamater of the pile is also introduced at the pile tip, otherwise, excessive displacement at the pile toe will occur.

**Remarks on Embedded Pile 3D Version in Axial Loading**

It should be noted that load-displacement graph is not an input in embedded pile. Load-displacement graph will be produced according to the assigned \( F_{\text{max}} \), \( T_{\text{max}} \), and \( G_{\text{soil}} \). Furthermore, installation effect has not been taken into account, and therefore, the load-displacement graph obtained is better matched with load-displacement graph for bored pile.

The accuracy of embedded pile model in single bored pile application subjected to axial loading has been investigated by Engin et al [2007, 2008, and 2009]. In these studies, the results of the calculation were matched with the field test data for bored pile. Figure 2.14 presents one of the matching result between embedded pile and field test.
2.4 Soil Constitutive Model

Earlier method involving ultimate limit state analysis is not sufficient to meet current society expectation of more detail result in geotechnical analysis. Chang and Duncan [1970] presented an example from a deep excavation with a depth of approximately 48m for pumping plant. The initial analysis indicated that in the equilibrium point of view, the excavation was stable. But, surprisingly within 1 year, the first stage of excavation induced large soil movement at the slope and the bottom of the excavation pit. The maximum measured heave at the bottom was 0.72m.

This event is not captured in the limit equilibrium analysis since it only assesses the safety against ultimate condition, in which plastic condition has been achieved. However, in some project, serviceability condition is more important. Deformation during the service could lead to large distortion in structure. Even the soil has not reached failure condition. Therefore, the necessity to use more rigorous analysis is important.

Recently, analysis in geotechnical engineering often involves numerical method, such as finite element method which before was commonly used only in the academic vicinity. It achieves its popularity since it enables engineers to analyse soil behaviour and deformation in more detail, step by step in every stage construction.

However, despite of its powerful and extraordinary capabilities compared to traditional analytical method, there is a fundamental principle that is liable for the result, it is soil constitutive model. Each constitutive model is usually developed to incorporate a specific condition. Therefore, every decision on the model could trigger significant difference in the results.

In this chapter, the background of the determination of soil constitutive model used in the analysis will be discussed.
2.4.1 Linear Elastic Perfectly Plastic vs Advanced Model

In practice, linear elastic perfectly plastic model is commonly employed in analysis of deep excavation projects as it offers a simplicity and readiness of required parameters which can be obtained from basic laboratory tests or empirical correlations. For instance, the most common used model, linear elastic perfectly plastic model with Mohr Coulomb or Tresca yield surface, only requires 5 main parameters; i.e Young’s modulus, poisson’s ratio, cohesion, friction angle, and dilatancy angle. On the other hand, more advanced model requires more parameters, which means, more laboratory or in-situ tests should be undertaken.

![Linear Elastic Perfectly Plastic Model](image)

FIGURE 2.15 LINEAR ELASTIC PERFECTLY PLASTIC MODEL (PLAXIS3D MANUAL, 2012)

However, the use of linear elastic perfectly plastic model might lead to erroneous calculation if the loading and stress state condition of the problem is not taken into account to the parameter determination process. The significance is explained in more details by the following examples.

Geomaterials behave non-linearly and in some materials, with the use of single stiffness value in linear elastic perfectly plastic model, the decision to take the correct stiffness value should receive special attention in order to give a correct result. At the time when the stress involved in the calculation is relatively small or the geomaterial has relatively large linear elastic range. Tangent modulus is a realistic representative of stiffness value. In other case, if the stress applied to the soil is relatively large, then, the stiffness value is better represented by secant modulus.

With respect to excavation cases, the determination of the stiffness value will be more complicated since different processes occur at the same time, i.e active loading and passive loading. Therefore, the model should be adjusted and calibrated in such a way that every aspect of soil behaviour can be captured in the model. Definitely, such process will consume more time and accuracy of the result might not be reliable.

Moreover, it is commonly understood that the stiffness behaviour of geomaterial is stress dependent. During the excavation project, the stress path of the soil might change and therefore the stiffness of the soil in each construction change might shift as well. For linear elastic perfectly plastic model, it only adopts constant value of stiffness. Hence, this fact hinders the use of linear elastic perfectly plastic model in a complicated case.
Ultimately, in detailed analysis of deep excavation, this solution will consume extensive analysis time, since a detail adjustment in the model should be performed for each condition occurs in the adjacent soil. In this situation, it is necessary to use a model which is capable to couple the stress changes with stiffness value changes.

Considering these facts, application of linear elastic perfectly plastic model might be good for first order approach in practice. But, more advanced model is required to simulate the excavation more conveniently. The next chapter will discuss hardening soil model which has the capabilities to overcome the aforementioned drawbacks of linear elastic perfectly plastic model.

2.4.2 Hardening Soil Model

So far, only a few soil constitutive models are designed to model unloading condition which conforms to excavation situation. Among the models, hardening soil model (HS Model) and hardening soil – small model (HSS Model) have been introduced and used widely by practicing engineers. These models are exclusively developed by PLAXIS. These advanced models have succeeded to incorporate several deficiencies in generic model, i.e Mohr-Coulumb, such as stress dependency behaviour. However, the main focus of the discussion is directed to the specific features of hardening soil model only.

The early development of Hardening Soil model was inspired by the work of Kondner and Zelasko [1963]. They introduced a hyperbolic relationship between the axial strain and the deviatoric stress development. In the model, the stiffness of the material will degrade and plastic strain will accumulate irreversibly during the development of deviatoric loading. It was succeeded by Duncan and Chang [1970], but this hyperbolic model cannot consistently simulate the transition between loading and unloading behaviour. Finally, Schanz et al [1999] refined the model and introduced hardening soil model.

Refinements are made in several aspects as follows:

- Instead of using elasticity theory, plasticity theory is used.
- Soil dilatancy is taken into account.
- And to take into account the compression hardening, a yield cap is incorporated.

The model will be explained in the following passages. As a basis, the explanation will be based on the drained triaxial test condition, where $\sigma'_3 = \sigma'_2$.

The original hyperbolic equation of the stress strain development proposed by Kondner gives infinite maximum deviatoric stress as shown in Figure 2.16. Mohr-Coulumb yield surface is introduced in the model as the failure criterion and thus, it defines the maximum deviatoric stress with the following equation:

$$ q_f = (c_{soil} \cot \varphi_{soil} - \sigma'_3) \frac{2 \sin \varphi_{soil}}{1 - \sin \varphi_{soil}} $$(2.40)
The ratio between asymptotic maximum deviatoric stress and the ultimate deviatoric stress is defined as:

\[ q_a = \frac{q_f}{R_f} \]  

(2.41)

Where,

- \( \sigma'_3 \) is the minor principal stress.
- \( c_{\text{soil}} \) is the cohesion of the soil, used in the Mohr-Coulomb failure criterion.
- \( \varphi_{\text{soil}} \) is the friction angle of the soil, used in the Mohr-Coulomb failure criterion.
- \( R_f \) is the failure ratio.

The hyperbolic stress-strain development in the model is defined by the following equation:

\[ -\varepsilon_1 = \frac{2 - R_f}{2E_{50}} \left( \frac{q}{q_a} \right) \]  

(2.42)

which is valid in the condition of \( q < q_f \). Where,

- \( \varepsilon_1 \) is the axial strain.
- \( q \) is the deviatoric stress.
- \( E_{50} \) is the secant modulus at 50% of failure stress.

It can be observed in Figure 2.16 that after the deviatoric stress imposed to the soil reaches \( q_f \), the behavior of the soil will be perfectly plastic. This is mainly due to the application of Mohr-Coulomb failure criterion in the model. By default, the failure ratio is 0.9.

As mentioned before that stress dependency behaviour of the soil stiffness has been implemented in the model. It can be observed in equation 2.43 which explains the formula of stiffness modulus in primary loading condition.
In unloading condition, the stiffness of the soil is represented by:

\[ E_{sr} = E_{ref}^r \left( \frac{\sigma' + c_{soil} \cot \varphi_{soil}}{\sigma_{ref} + c_{soil} \cot \varphi_{soil}} \right)^m \]  \hspace{1cm} (2.43)

Where,

- \( E_{ref}^{50} \) is the reference secant modulus at 50% failure.
- \( \sigma_{ref} \) is the reference stress level.
- \( m \) is the factor of stress dependency.
- \( E_{ref}^{ur} \) is the reference Young’s modulus for unloading reloading.

\( E_{oed} \), the stiffness of the soil in 1D compression, is an input in the model since hardening soil model does not adopt a fixed relationship between \( E_{50} \) and \( E_{oed} \), unlike elasticity based model. The stress dependency in the oedometer stiffness is defined as:

\[ E_{oed} = E_{oed}^{ref} \left( \frac{\sigma' / K_{NC} + c_{soil} \cot \varphi_{soil}}{\sigma_{ref} + c_{soil} \cot \varphi_{soil}} \right)^m \]  \hspace{1cm} (2.45)

Similar to the stiffness in primary loading and unloading condition, \( E_{oed}^{ref} \) explains the reference oedometer modulus in the reference stress level.

This soil model is based on the plasticity theory and therefore, to accommodate irreversible straining, there are 2 types of hardening used in the model. For irreversible straining during deviatoric loading, shear hardening is used. On the other hand, compression hardening is used to model the irreversible straining in primary compression. It should be noted that the yield surfaces can expand, unlike elastic perfectly plastic model.

The shear hardening yield function is defined by:

\[ f = \frac{2 - R}{E_{50}} \cdot \frac{q}{1 - \frac{q}{q_u}} - \frac{2q}{E_{ur}} - \gamma^p \]  \hspace{1cm} (2.46)

Where,

\[ \gamma^p = -(2\varepsilon^p_1 - \varepsilon^p_2) \]  \hspace{1cm} (2.47)
$\gamma^p$ is defined as hardening parameter which explains the plastic shear strain in the model. Using the aforementioned equations, the plastic strain can be obtained in any conditions during primary loading.

The elastic strain which can occur during primary and unloading – reloading condition is given by:

$$ -\varepsilon_1^e = \frac{q}{E_{ur}} \quad (2.48) $$

$$ -\varepsilon_2^e = -\varepsilon_3^e = -\nu_{ur} \frac{q}{E_{ur}} \quad (2.49) $$

The shear stiffness of in the elastic region is defined by:

$$ G_{ur} = \frac{1}{2(1+\nu_{ur})} E_{ur} \quad (2.50) $$

If $\gamma^p$ is held as a constant, the yield condition ($f = 0$) can be plotted in $p$-$q$ plane. Due to the influence of stress dependency in equation 2.43 and 2.44, the shape of the yield locus will depend on the value of $m$. For very soft soil, in which the value of $m = 0$, the yield locus tends to be a straight line. On the other hand, for a value of $m = 0.5$ as in hard soil, the yield locus tends to curve.

Observing a specific material behavior, i.e $m = 0.5$, with various value of plastic shear strain, Figure 2.17 can be obtained. It can be observed that originally the yield loci of the model is slightly curved and with the developing plastic shear strain, the model develops shear resistance of the soil until it reaches the specified linear failure condition (equation 2.40).

![Figure 2.17](Image)

**Figure 2.17 The influence of various values $\gamma^p$ to the shape of the yield loci (Schanz et al., 1999)**

Similar to other plasticity theory based models, the rates of plastic strain are related. The relationship between the rate of plastic shear strain and the rate of plastic volumetric strain is governed by:
\[ \varepsilon_v^P = \sin \psi_m \dot{\varepsilon}_p \] (2.51)

Where, \( \psi_m \) is the mobilised dilatancy angle and it is determined by considering the following conditions:

- For \( \sin \phi_m < \frac{3}{4} \sin \phi_{soil} \), the value of \( \psi_m \) is 0.
- For \( \sin \phi_m > \frac{3}{4} \sin \phi_{soil} \) and \( \psi > 0 \), the value of \( \psi_m \) is:

\[
\sin \psi_m = \max \left( \frac{\sin \phi_m - \sin \phi_{cv}}{1 - \sin \phi_m \sin \phi_{cv}}, 0 \right)
\] (2.52)

- For \( \sin \phi_m > \frac{3}{4} \sin \phi_{soil} \) and \( \psi \leq 0 \), the value of \( \psi_m \) is \( \psi \).
- For \( \phi_{soil} = 0^\circ \), the value of \( \psi_m \) is \( \psi \).

\( \phi_{cv} \) is the critical state friction angle, which is a the material constant and independent on the density. On the other hand, \( \phi_m \) is the mobilised friction angle of the material. The determination of \( \phi_m \) follows:

\[
\sin \phi_m = \frac{\sigma'_{1} - \sigma'_{3}}{\sigma'_{1} + \sigma'_{3} - 2c_{soil} \cot \phi_{soil}}
\] (2.53)

The aforementioned conditions are based on Rowe’s [1962] stress – dilatancy theory. Using the friction angle (\( \phi_{soil} \)) and the dilatancy angle (\( \psi_p \)) at failure condition, the critical state dilatancy angle is calculated by:

\[
\sin \psi_{cv} = \frac{\sin \phi_{soil} - \sin \psi_p}{1 - \sin \phi_{soil} \sin \psi_p}
\] (2.54)

However, a special attention should be paid to the dilating material. After extensive shearing, the dilatancy should be ended. Otherwise, it is not realistic to let the dilatancy of the material continues infinitely. It is understood that the dilatants behaviour is closely related to the void ratio of the material. Therefore, PLAXIS allows the users to apply dilatancy cut off by specifying the initial and maximum void ratio. Using such a way, the dilatancy can be limited as indicated in Figure 2.18.
For void ratio smaller than maximum void ratio, the mobilised dilatancy angle is calculated by:

\[
\sin \psi_m = \frac{\sin \varphi_m - \sin \varphi_{cv}}{1 - \sin \varphi_m \sin \varphi_{cv}} \quad (2.55)
\]

If the maximum void ratio has been reached, the mobilised dilatancy angle will be set to 0°.

### 2.4.3 Soft Soil Creep Model

In addition to hardening soil model, soft soil creep model will be used to model the autonomous settlement of the Amsterdam subsurface (see chapter 4.2). Therefore, it is worthwhile to discuss the model.

The capability of hardening soil model in modelling soil subjected primary loading has been recognised. From hardening soil model, the oedometer stiffness can be defined as:

\[
E_{oed} = E_{oed}^{ref} \left( \frac{-\sigma'}{p^{ref}} \right)^m \quad (2.56)
\]

Considering soft soil with cohesion of 0kPa and linear stress dependency behaviour \((m = 1)\), it leads to:

\[
E_{oed} = -\sigma' \frac{E_{oed}^{ref}}{p^{ref}} \quad (2.57)
\]

The incremental strain due to primary loading then:

---

Figure 2.18 The application of dilatancy cut off in drained triaxial test (PLAXIS 3D manual, 2012)
\[
\dot{\varepsilon} = \frac{p^{\text{ref}}}{E_{\text{ov}}^{\text{ref}}} \dot{\sigma}'_1
\]  

(2.58)

Where, \( \dot{\sigma}'_1 \) is the incremental major principal stress.

However, the model is no longer appropriate in some conditions, for instance, when engineers need to deal with a very soft soil or even stiff soil which behaviour is influenced by creep. This drawback can be modelled properly using soft soil creep model.

The following formula was derived by Buisman [1936], which was proposed to model creep behaviour under constant effective stress:

\[
\varepsilon = \varepsilon_c - C_B \log \left( \frac{t_c + t'}{t_c} \right)
\]  

(2.59)

Where,

- \( \varepsilon_c \) is the total strain due to primary consolidation.
- \( t' \) is the creep time
- \( t_c \) is the time to complete the primary consolidation.
- \( C_B \) is the material constant.

Garlanger [1972] proposed a slightly different approach. He defined similar formula, but the engineering strain was replaced by void ratio. Moreover, \( \tau_c \) was used instead of \( t_c \).

Butterfield [1979] replaced the strain in Buisman formula with logarithmic strain and similar to Garlanger, \( \tau_c \) was used instead of \( t_c \) as shown below:

\[
\varepsilon_H^H = \varepsilon_c^H - C \log \left( \frac{\tau_c + t'}{\tau_c} \right)
\]  

(2.60)

Where,

- \( \varepsilon_H^H \) is the logarithmic strain, defined as \( \ln \left( \frac{1 + \varepsilon}{1 + e_0} \right) \)
- \( \varepsilon_c^H \) is the total logarithmic strain due to primary consolidation.
- \( e_0 \) and \( \varepsilon \) are the void ratios, in initial and final condition respectively.

This equation will be the basis of the soft soil creep model in PLAXIS.

For small strain:

\[
C = \frac{C_a}{(1 + e_0) \ln 10} = \frac{C_B}{\ln 10}
\]  

(2.61)
One might question, how to obtain the creep time while in compression test such as oedometer test, it is always considered that only primary consolidation occurs during the entire period of the test. This assumption is not true. For a test period of 1 day, Vermeer and Neher [1999] argued that merely an hour is required to complete the dissipation of the pore pressure even in soil with low permeability. Thus, primary consolidation is accomplished and during the rest of the test, pure creep occurs. However, the determination of $t'$ and $\tau_c$ is not a trivial.

The background of the creep model has been explained. But, it should be noted that the aforementioned theories are explained based on the constant loading. In order to model the entire transient loading conditions, the constitutive law will be explained. To avoid the complexity in the determination of $t'$ and $\tau_c$ from oedometer test, both will be re-defined.

The time dependent settlement of Amsterdam subsurface will modelled in 1D. Therefore for an efficient explanation, only 1D constitutive law will be explained.

The classical consolidation theory describes that the total strain in the end of consolidation can be calculated by:

\[
\varepsilon^e_c = \varepsilon^H_e - \varepsilon^H_c = -A \ln \left( \frac{\sigma'}{\sigma'_0} \right) - B \ln \left( \frac{\sigma_{pc}}{\sigma_{p0}} \right)
\]  

(2.62)

The variable A and B are described by:

\[
A = \frac{C_r}{(1 + e_0) \ln 10}
\]

(2.63)

\[
B = \frac{C_e - C_r}{(1 + e_0) \ln 10}
\]

(2.64)

Where,

- $\varepsilon^H_e$ is the logarithmic strain in elastic part.
- $\varepsilon^H_c$ is the logarithmic strain in virgin area.
- $\sigma'$ is the effective load pressure.
- $\sigma'_0$ is the in situ effective stress of the soil.
- $\sigma_{p0}$ is the preconsolidation pressure before the loading.
- $\sigma_{pc}$ is the achieved preconsolidation pressure after the loading.
- $C_r$ is the recompression index obtained from oedometer test.
- $C_e$ is the compression index obtained from oedometer test.

Therefore, the equation for total strain can be expressed by:
\[ \varepsilon'' = \varepsilon_{c}^{H} - \varepsilon_{c}^{H} = -A \ln \left( \frac{\sigma_{t}}{\sigma_{0}} \right) - B \ln \left( \frac{\sigma_{pc}}{\sigma_{p0}} \right) - C \log \left( \frac{\tau_{c} + t'}{\tau_{c}} \right) \]  

(2.65)

It is evident that this equation still contains \( t' \) and \( \tau_{c} \). The definition of \( t' \) has been given, but not \( \tau_{c} \). Theoretically, \( \tau_{c} \) can be defined by means of differentiating equation 2.60 with respect to time, resulting in:

\[-\dot{\varepsilon}' = \frac{C}{\tau_{c} + t'} \]  

(2.66)

Thus, \( \tau_{c} \) can be obtained directly, if \( C \) and \( t' \) are known. The value of \( C \) can be obtained using the oedometer test result, as shown in Figure 2.19. Subsequently, \( \tau_{c} \) is obtained by intercepting the creep line with the non-logarithmic time axis as shown by Janbu [1969] in Figure 2.19 (b).

In order to give an analytical solution for \( \tau_{c} \), several assumptions are taken:

- All inelastic strains are time dependent.
- No instantaneous plastic strains occur in non failure condition as in oedometer test.
- Preconsolidation stress depends on the accumulated strain, as postulated by Bjerrum [1967].

These assumption leads to the modification of equation 2.62:

\[ \varepsilon_{c}^{H} = \varepsilon_{c}^{H_{r}} - \varepsilon_{c}^{H_{c}} = -A \ln \left( \frac{\sigma_{t}}{\sigma_{0}} \right) - B \ln \left( \frac{\sigma_{p}}{\sigma_{p0}} \right) \]  

(2.67)

Where,
\[ \sigma_p = \sigma_{p0} \exp\left(-\frac{e^c}{B}\right) \quad (2.68) \]

This equation implies that the longer the soil is subjected to creep, the larger the preconsolidation stress. Furthermore, the time dependency of preconsolidation pressure can be defined as:

\[ \varepsilon^{H_c} - \varepsilon^{H_c} = -B \ln\left(\frac{\sigma_p}{\sigma_{p0}}\right) = -C \log\left(\frac{\tau_e + \tau'}{\tau_c}\right) \quad (2.69) \]

In the condition of oedometer test, a load step is maintained for a constant period of \( \tau' \), which is commonly 1 day. Therefore, \( \tau = t_c + \tau' \). Assuming that the sample is in normally consolidation line, the value of \( \sigma_p = \sigma' \). Equation 2.69 then can be modified into:

\[ B \ln\left(\frac{\sigma'}{\sigma_{p0}}\right) = C \log\left(\frac{\tau_e + \tau - t_c}{\tau_c}\right) \quad (2.70) \]

It should be noted that the equation above is valid for normally consolidated condition.

It is assumed that \( \tau_c - t_c << \tau \). It leads to the definition of \( \tau_c \):

\[ \frac{\tau}{\tau_c} = \left(\frac{\sigma'}{\sigma_{p0}}\right)^{B/C} \Rightarrow \tau_c = \tau \left(\frac{\sigma'}{\sigma_{p0}}\right)^{B/C} \quad (2.71) \]

The aforementioned assumption is tested by the following concept. It has been understood that merely one hour is required to achieve the end of primary consolidation period. On the normally consolidated line, we will have overconsolidation ratio of 1, both in the beginning and the end of the test. As observed in Figure 2.20 during the primary consolidation, \( \sigma_{p0} \) changes to \( \sigma_{pc} \) and subsequently, \( \sigma_{pc} \) changes to \( \sigma' \) during the creep period. Eventually, the sample will be in the normally consolidation state again in the end of the test. But, the sample will be in the under-consolidated condition after short consolidation period (\( \sigma_p < \sigma' \)). Commonly, the ratio of B/C is very high. With B/C \( \geq 15 \) applied in equation 2.71, the value of \( \tau_c \) will be very small. With a small value of \( \tau_c \) and \( t_c \), therefore \( \tau_c - t_c << \tau \) is correct.

Ultimately, the complete differential creep equation is presented below:

\[ \varepsilon_c^H = \varepsilon_c^{H_c} - \varepsilon_c^{H_c} = -A \frac{\dot{\varepsilon}'}{\sigma'} - \frac{C}{\tau} \left(\frac{\sigma'}{\sigma_p}\right)^{B/C} \quad (2.72) \]
FIGURE 2.20 ILLUSTRATION OF STRESS-STRAIN CURVE FROM OEDOMETER TEST
3. The Project: Noord-Zuid Lijn Amsterdam

A metro project in Amsterdam, Noord-Zuid Lijn, is the focus of the study. This chapter gives an introduction and some details of the project. The explanation will contain the description of the project, subsurface condition of the soil and the groundwater, the buildings adjacent to the project and its foundation. During the execution of the project, there was an enormous amount of measurement devices installed to monitor the displacement of the soil and the adjacent buildings. The type of measurement devices is also presented. Eventually, this chapter describes the details of the construction of Ceintuurbaan Station, which is the primary focus of the study.

3.1 Project Description

Noord-Zuid Lijn is an extension metro project in Amsterdam. The extension stretched from northern part (Buikslotermeerplein) to southern part of Amsterdam (World Trade Centre), as shown in Figure 3.1. The total length of the project was 9.5km and 3.8km was constructed underground. Among 3.8km long underground segment, tunnel boring machine was used to drill 3.1km of the metro line and the rest consisted of the underground stations.

Eight metro stations were constructed in total, in which 4 of them were built underground; Amsterdam Central Station, Rokin, Vijzelgracht, and Ceintuurbaan. Due to the limited space, Amsterdam Central Station was built using immersing technique. The tunnel segment needs to be inserted under the old station from River IJ which is situated next to the Amsterdam Central Station. On the other hand, the next 3 stations were constructed using top down method. Figure 3.2 and Figure 3.3 present the illustration of the different methods employed in the project.

Rokin station stretches about 200m long with 24.5m wide and the lowest point of the excavation is NAP -26.5m, approximately. Vijzelgracht station has been the longest part, about 260m long. The width of the station is 22m with NAP -30.5m as the final depth of the excavation. These dimensions of the 2 stations allow the metro line to be constructed at the same level, side by side. But, it is not applicable to Ceintuurbaan station. Due to the narrow space in the neighbourhood, the optimum width of the station box was only 12m wide, forcing the metro line to be built in 2 levels as shown in Figure 3.3. The length of the station is approximately 210m and the final depth of the excavation is NAP -31m. The dimension presented above is the approximate value since some local adjustments in some cross section were done.
Figure 3.1 Illustration of Noord-Zuid Lijn Project (COB, 2011(a))

Figure 3.2 Impression of immersed tunnel under Amsterdam Central station (Hasse et al. (1999))
To maintain the stability of the soil during the excavation, diaphragm wall is chosen as the retaining wall, with a thickness varies from 1.2 – 1.4m in those 3 stations. It was embedded to NAP -36m at Rokin and NAP -45m at 2 other stations. The construction of the station was meant to be executed in dry condition. Hence, after the diaphragm wall was accomplished to be installed, dewatering was performed locally in the inner side of the station. During the excavation of the deeper layer, the stability against basal heave at Vijzelgracht and Ceintuurbaan, was not sufficient. To anticipate this issue, the excavation then was executed with pressurised chamber.

It was realised that the project might cause disturbance to the surrounding area. Especially, it is because the alignment of the project would pass through the historic centre of Amsterdam, where most of the buildings were built approximately 100 years ago. It was considered that the area would be sensitive to such disturbance. Therefore, to anticipate and monitor the response of the buildings and the soils, an enormous amount of monitoring instruments were installed. Further explanation of the monitoring devices will be discussed in chapter 3.4.

It is worth to mention that 3 leakages in the diaphragm wall right after installation occurred at Vijzelgracht station and caused several buildings settled at once. The first leakage occurred close to Vijzelgracht 20-26, resulted in a settlement of 15cm. The second leakage triggered Vijzelgracht 8,6, and 4 to settle 20cm. It was suspected that the leakage occurred at the foundation tip level. The leakage allowed the water to erode the soil and consequently, it piped out. Certainly, the soil beneath the houses lost its strength referring to the CPT results, as shown in Figure 3.5. Eventually, the pile foundations of the buildings were influenced adversely.
However, this issue will not be addressed in this research. Further discussion and analysis will focus on Ceintuurbaan station. It was concluded that the most reliable measurement results are at Ceintuurbaan station.

![Image: Vijzelgracht 22 after the incident (Korff et al., 2011)](image1.png)

**Figure 3.4 Vijzelgracht 22 after the incident (Korff et al., 2011)**

![Image: CPT results before (left) and after (right) the first leakage (Korff et al., 2011)](image2.png)

**Figure 3.5 CPT results before (left) and after (right) the first leakage (Korff et al., 2011)**
3.2 Subsurface Condition of The Netherlands and Amsterdam

The most influential part of the geological sediments in The Netherlands extends to 50-60m deep below surface level. The nature of this shallow subsurface is very young in geological period. The age of the formation is approximately less than 1.81 million years old since it was formed after Neogene era (Tertiary era). It is mostly dominated by Quaternary sediment, including Holocene and Pleistocene deposits. Only 2% of the outcrop found at the surface is traced back to Tertiary period. Table 3.1 explains the order of the geology age dominating The Netherlands shallow subsurface.

**Table 3.1 Quaternary period dominating The Netherlands (van der Woude, 2011)**

<table>
<thead>
<tr>
<th>System</th>
<th>Series</th>
<th>Subseries</th>
<th>Stages</th>
<th>Ages [million years]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quaternary</td>
<td>Holocene</td>
<td>Flandrien</td>
<td></td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td>Pleistocene</td>
<td>Late</td>
<td>Wechselian</td>
<td>0.13</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pleistocene</td>
<td>Eemian</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Middle</td>
<td>Saalian</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pleistocene</td>
<td>Holstenian</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Elsterian</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Cromerian</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Early</td>
<td>Menapian</td>
<td>1.81</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pleistocene</td>
<td>Waalian</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Eburonian</td>
<td></td>
</tr>
</tbody>
</table>

When Quaternary period started, Early and Middle Pleistocene material were deposited and mainly formed by massive fluvial deposits from River Rhine and Maas. During the Late Pleistocene, the temperature of the earth dropped and glaciers were stacked. The glaciers from northern part of Europe invaded Netherlands during Saalian Age (Late Pleistocene). No wonder the soil layers under and including third sand layer are overconsolidated, due to the effect of preloading by the glaciers in the past.

Furthermore, this glacial movement triggered some moraines formation in some parts of the Netherlands, for instance Utrechtse Heuvelrug and Veluwe. However, any moraines were not formed in Amsterdam, only fluvio-glacial deposits were brought and deposited. The first layer of fluvio-glacial deposits of this era consists of 5-30m laminated clay, called Warven Clay. Intermediate Sand layer lied over Warven Clay and generally it was formed by very fine sand.

After this period, during the Eemian age, the sea level rose and caused more fine material to be transported and deposited. In this period, Eem Clay was formed. It was dominated by laminated, crumbly, to massive clay with some quartz, calcite and illite mineral. Then, the last period of Late Pleistocene (Wechselian period) commenced. During this period, Aeolian Sand was transported over large area of The Netherlands, and subsequently, it became the main material to form first and second sand layer, which can be found in general Amsterdam condition. Both sand layers consist of fine to medium coarse sand. Between these layers,
Allerød layer can be found. It consists of silty and sandy clay. The thickness of this package is generally 10-15m.

Over those soil layers, the rest of the deposits were formed during the Holocene period. Rapid transgression happened in the early period, and due to this event, old Strandwallen (sea ridges) and Basis Veen (peat layer) were deposited. Gradually, the gradient of the river was reduced and tidal flats existed. Consequently, fine materials were transported and Hydorbia Clay was formed. Later, brackish environment dominated and it resulted in the Wad deposits sedimentation. Old Sea Clay and Holland Peat were deposited then. Lately, Ophooglag or Antropogene sand became the first layer found from the surface.

The geohydrology condition of typical Amsterdam subsurface is dominated by 3 different phreatic levels. The first phreatic level is influenced by Holocene layers. First Sand layer, Allerød, and Second Sand layer form another phreatic level, while, the last phreatic level is governed by Third Sand layer. The determined 3 phreatic levels for each station are included in Figure 3.6 (Projectbureau NoordZuidLijn, 2008b).

In general, the phreatic level of the Holocene layer remains consistently from the north to south part of the project, about NAP -0.5m. The second and the third phreatic level are different at the north part of Amsterdam. They gradually share almost the same phreatic level as we move to the south. Interestingly, both indicate declination of phreatic level from north to south side of the project. It can be clearly observed in Figure 3.6. The plausible explanation for this phenomenon is the location of river Ij which is next to Amsterdam Central station. Consequently, high pore pressure can be expected. But, as we go deeper, we can observe that the influence of river Ij to third sand layer is reduced.
3.3 Buildings Adjacent to the Project

Mostly, buildings in the historic centre of Amsterdam are approximately 110 years old. The majority was constructed in 1800s, only a few buildings were established in 1600s. These old buildings use masonry walls and are founded on timber piles. Typically, the buildings consist of 4-storey houses, in a few cases, some houses consist of 5 storeys. In addition, some houses also have one level of basement. Figure 3.7 shows a typical old Amsterdam house.

In Figure 3.8, we can observe the detail of the connection between superstructure and timber pile. The thickness of the wall close to the foundation level is generally 440mm and in the upper structure, it reduces. The thickness at the ground level is 330mm and 220mm for the higher floor.

![Figure 3.7 An example of typical old Amsterdam building - Ferdinand Bolstraat 95](image)

To support the building, timber piles were usually driven and arranged in row as shown in the illustration. The distance between a pair of timber piles is approximately 0.8m. The pile itself has 180-200mm diameter at the top. Normally, the timber pile used in for foundation is tapered with a gradient of 8mm/m, resulting in smaller tip diameter. It is found that the minimum tip diameter is 80-85mm. Over the timber piles, a cross beam is used to connect both piles. The thickness of this beam is about 2.5cm. Then, the final connection between the foundation and the wall is a long plate with 10cm thickness. Pine wood was used as the timber pile material, while cross beam and plate were made of oak wood. This type of foundation is typically called as Amsterdamse Funderingen, means Amsterdam foundation. The tip of the pile is generally located at the first sand layer, approximately NAP -12m to -13m.
Some defects were found in the Amsterdam foundation, especially in the upper part of the foundation as shown in Figure 3.9. It is evident that the cross beam has cracked. It is suspected that overload of the foundation is responsible for it. Besides, it is also possible that the cross beam has experienced degradation due to bacteria invasion, which is commonly found in timber material. Further explanation of the degradation is explained in chapter Error! Reference source not found., which also exposes the current condition of the timberile in The Netherlands.
Certainly by having this problem, the load from the superstructure is not fully transferrable to the timber pile and the soil-structure interaction in the building changes. As an adverse result, buildings could settle unequally and structural damage might happen subsequently.

Houses built in the early 1900s experienced a slightly different foundation system. Reinforced concrete beam was introduced, replacing the timber cross beam. Timber piles were not used in pair as before. Instead, they were installed single in a row. Starting from around 1945, concrete pile became more popular and as a result, new houses at that time had started to use driven concrete pile.

Van Tol [2012] presented a graph comparing poor foundations in Amsterdam and Rotterdam houses which were built in different years, as shown below. It can be concluded that in both cities, the new application of foundation system had successfully reduced the number of poorly founded buildings. Furthermore, we can expect better performance of newer building in the adjacent area of the project.
3.4 Monitoring Instruments

To measure and monitor the movement of the buildings, the groundwater level and wall deformation, some instruments have been installed. These chapters will give some overview about the instruments used and also some specific information related to the measurement results.

3.4.1 Monitoring Soil Displacement

Extensometer and inclinometers were installed and used as the instruments to monitor soil displacement in the project. Extensometer is employed to measure vertical soil displacement and inclinometer is utilized to monitor horizontal soil displacement. In each station, there are 4 sets of measurement arrays for extensometers and inclinometers. Several packers of the extensometers were installed in several depths and the datum of the measurement is at the third sand layer but with various depths, as shown in Figure 3.11. As explained in chapter 3.2, at a depth below 50m, we can find the overconsolidated soil layer and it can be assumed as a stable point. Consequently, the results of the measurement from extensometers will give absolute settlement of each layer. The inclinometer measurement levels also refer to the same reference datum of the extensometers. Both measurements were undertaken automatically in every several days.

It should be noted that the extensometers and inclinometers were installed in the same hole and this cause an accuracy problem. This is caused mainly due to the friction between the extensometers and the inclinometers. De Nijs and Buyx [2009] concluded that the accuracy of the results of the extensometers can vary from 2 to 5 mm, while theoretically, it can be optimised to 1mm.
As a corrective measure, manual measurements for the soil settlement were also performed. The reference datum of the manual measurements was the third sand layer. The results of the measurements then can be compared to the extensometers results. But the comparison is only limited to surface, because the manual measurement only recorded the settlement of the surface level.

3.4.2 Monitoring Building Displacement

The monitoring of building displacement in this project relies on automatic monitoring system employing several robotic total stations and prisms. The total stations were installed in the facade of several buildings which were considered not to be influenced by the excavation and possessed a decent foundation based on the inspection report. Consequently, the displacement of other building measured will be a relative value, since the reference buildings might also displace. An example of the location of the robotic total station location is presented in Figure 3.12.

The prisms were installed in every building adjacent to the pit. Generally, there were 4 prisms installed at least in each building and each prism was monitored by 2 robotic total stations. Seven robotic total stations were located at each station and they regularly monitored 50-100 prisms.
In addition, manual levelling was also performed in each building to give a back up data for the building settlement measurement results. Unlike the prism measurement, the datum for manual levelling point is based on the deep point, approximately in the third sand layer. However, the manual levelling is performed much less frequent compared to total robotic station. The interval of the measurement is about 6-12 months. Figure 3.13 shows the measurement point of the building.

**Figure 3.12 Robotic total stations with prisms at Ceintuurban Station (COB, 2011(c))**
FIGURE 3.13 MEASUREMENT POINTS AT FERDINAND BOLSTRAAT 69 (COB, 2012)
3.4.3 Other Measurements

Inclinometers were also installed in the diaphragm wall panels to measure the wall deflection during the excavation. But, the references in the measurement are not fixed and therefore, relative value is obtained from the measurement. The base of the inclinometer, which is located at the tip of the diaphragm wall, can be assumed as the reference point as it is expected that it experiences the least movement.

Groundwater level is also surveyed by means of several different instruments, e.g. standpipes and piezometers (vibrating wire and special BAT). The measurement of the struts load during the excavation was taken by employing vibrating wire strain gauges.

3.4.4 Reference Datum – Normaal Amsterdam Peil

The depth value in this research is always accompanied by NAP. This sign means *Normaal Amsterdam Peil*, which is the ordnance datum for vertical measurement applied in The Netherlands. Figure 3.14 shows the monument of Amsterdam ordnance datum in Amsterdam city council building.

![Figure 3.14 Monument for Amsterdam Ordnance Datum](http://commons.wikimedia.org/wiki/File:NAP_reference2.jpg)
3.5 Construction of Ceintuurbaan Station

Ceintuurbaan station is located at Ferdinand Bolstraat. Figure 3.15 shows the location of Ceintuurbaan station with respect to the entire project alignment. There are 2 major cross sections at Ceintuurbaan station, 13044 and 13110. 13044 is situated at Govert Flinkstraat which is at the northern part of the station, and 13110 is located at Eerste Jan van der Heijdenstraat, southern side of the station. The impression of the station and the cross section are presented in Figure 3.16 to Figure 3.18.

As described before, this station is the narrowest station among other stations. The length of the station is approximately 210m. The width of the station is 13855mm 12752mm, at 13044 and 13110 respectively. This number is the total width measured from the outer side of the diaphragm wall. The station was constructed by employing top-down method, with concrete diaphragm wall and 4 levels of steel struts as the supporting structures. The thickness of the diaphragm wall itself is 1200mm and it was embedded down to NAP -45m. The final excavation depth at 13044 is NAP-30.9m and NAP -30.5m at 13110. The overview of dimension of the excavation is presented in Table 3.2.

<table>
<thead>
<tr>
<th></th>
<th>13044</th>
<th>13110</th>
</tr>
</thead>
<tbody>
<tr>
<td>Final excavation depth [m]</td>
<td>NAP -30.9m</td>
<td>NAP -30.5m</td>
</tr>
<tr>
<td>Total width (including wall thickness) [m]</td>
<td>13.855m</td>
<td>12.752m</td>
</tr>
<tr>
<td>Total net width (excluding wall thickness) [m]</td>
<td>11.455m</td>
<td>10.372m</td>
</tr>
<tr>
<td>Wall thickness [m]</td>
<td>1.2m</td>
<td>1.2m</td>
</tr>
<tr>
<td>Top level of diaphragm wall [m]</td>
<td>NAP -0.8m</td>
<td>NAP -0.8m</td>
</tr>
<tr>
<td>Embedment of diaphragm wall [m]</td>
<td>NAP -45m</td>
<td>NAP -45m</td>
</tr>
</tbody>
</table>

Figure 3.15 Location of Ceintuurbaan station (taken from HTTP://WWW.HIERZIJNWIJ.NU/MAP/)
Figure 3.16 Layout of Ceintuurbaan station including cross section and measurement points
The overview of Ceintuurbaan station has been presented. The later passage will focus in more details to cross section 13110E (East), which is the main interest of the research.
3.5.1 Soil Condition and Phreatic Level – 13110E

The soil condition in Ceintuurbaan is driven by the local geology of Amsterdam that has been explained earlier. The general soil layers for cross section 13110 are presented in Figure 3.19.

In more detail, Table 3.3 lists the subsurface of Ceintuurbaan materials.

<table>
<thead>
<tr>
<th>Layer name</th>
<th>Dutch name</th>
<th>Top Level [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandfill</td>
<td>Ophooglaag</td>
<td>NAP +0.4</td>
</tr>
<tr>
<td>Peat</td>
<td>Hollandveen</td>
<td>NAP -2.3 to -3.0</td>
</tr>
<tr>
<td>Old sea clay</td>
<td>Oudezeeklei</td>
<td>NAP -5.0</td>
</tr>
<tr>
<td>Wad deposit</td>
<td>Wadzandlaag</td>
<td>NAP -6.4</td>
</tr>
<tr>
<td>Hydrobia clay</td>
<td>Hydrobiaklei</td>
<td>NAP -10.3 to -11.0</td>
</tr>
<tr>
<td>Peat</td>
<td>Basiveen</td>
<td>NAP -10.5 to -11.5</td>
</tr>
<tr>
<td>First sand layer</td>
<td>Eerstezandlaag</td>
<td>NAP -11.0 to -12.0</td>
</tr>
<tr>
<td>Loam</td>
<td>Allerød</td>
<td>NAP -14.0</td>
</tr>
<tr>
<td>Second sand layer</td>
<td>Tweedezand lag</td>
<td>NAP -15.5 to -16.0</td>
</tr>
<tr>
<td>Eemian clay</td>
<td>Eemclay</td>
<td>NAP -24.7 to -25.0</td>
</tr>
<tr>
<td>Intermediate sand layer</td>
<td>Tussenzandlaag</td>
<td>NAP -38.0 to -38.2</td>
</tr>
<tr>
<td>Glacial Drenthe clay</td>
<td>GlacialeDrentheklei</td>
<td>NAP -41 to -42</td>
</tr>
<tr>
<td>Glacial Warven clay</td>
<td>GlacialeWarvenklei</td>
<td>NAP – 45.5 to -49</td>
</tr>
<tr>
<td>Third sand layer</td>
<td>Derdezandlaag</td>
<td>NAP -47.3 to -50</td>
</tr>
</tbody>
</table>

**FIGURE 3.19 CROSS SECTION AT 13110 (KORFF, 2012)**
APPENDIX A presents the results of the CPTs performed during the investigation stage around this cross section. The engineering parameters derived and adopted in the design of the station are presented in APPENDIX B.

At Ceintuurbaan station, Holocene layer generally has average phreatic level of NAP -0.44 to -0.5m. The first sand layer’s phreatic level is located at NAP -2.75m, while for the second sand layer, it is NAP -2.84m. The phreatic levels between these 2 sand layers do not fluctuate dramatically. In addition, the distance between these 2 sand layers are not significant. Considering those facts, therefore, the First Sand layer, Allerød, and the Second Sand layer can be considered sharing the same phreatic level. Third Sand layer has similar phreatic level, which is at NAP -2.99m, but it is considered as a layer with different phreatic level due to the existence of thick clay layer over it. There are no available data for the intermediate sand layer, consequently, the phreatic level for the intermediate sand layer is presumably assumed similar to third sand layer.

3.5.2 Project Sequence – 13110E

In this chapter, the total sequence in the project will be explained. The project itself consists of 3 parts, including the preparation, preliminary works, and the main excavation.

The preparation stage was conducted mainly to prepare and complete the data required. It should be remarked that the monitoring devices, especially for the buildings and soils, were installed in this stage. It was beneficial as in this stage the settlement of Amsterdam soils and buildings without the influence of the project could be captured.

For the preliminary work, Korff [2012] has listed some related works involved in the project which consists of:

- The removal of obstacles.
  In this stage, pipelines, sewers, and other infrastructures located at the top of the proposed excavation were removed.
- Filling of additional anthropogenic sand.
  At this stage, additional sand fill was filled as a working platform for diaphragm wall installation. The sand fill was installed only at the top of the station box.
- Diaphragm wall installation.
- Jet grouting installation.
  Jet grout struts were installed as part of the excavation system to maintain the wall deflection in the range of the allowable value.
- Pumping test.

Actually, Korff mentioned 1 additional activity to be included in the preliminary work, which is the first excavation and roof construction. Since the proposed work method would employ top-down method, it required a shallow excavation to allow the roof of the station to be installed. The first excavation was done until NAP -1.95m.

Korff included the first of excavation to the preliminary work in her work to avoid some confusion in the transition of the schedule between preliminary work and main excavation.
period in both cross sections. But, in the further consideration of this research, this stage will be included in the main excavation stage.

The details of the activities involved in the preliminary works and the main excavation for 13110E are summarised in Table 3.4. They also included the period when each activity was performed and for each cross section. APPENDIX C provides the complete illustration of the project sequence. It should be noted that the depths, both in the tables and in the APPENDIX C, refer to NAP value.

**Table 3.4 Activities schedule for Ceintuurbaan 13110**

<table>
<thead>
<tr>
<th>Stage</th>
<th>Activities</th>
<th>Start</th>
<th>Finish</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Additional fill (+70cm)</td>
<td>15-11-2003</td>
<td>08-04-2004</td>
</tr>
<tr>
<td>2</td>
<td>Diaphragm wall installed</td>
<td>20-08-2004</td>
<td>13-09-2004</td>
</tr>
<tr>
<td>3</td>
<td>Jetgrout strut installed</td>
<td>14-10-2005</td>
<td>08-11-2005</td>
</tr>
<tr>
<td>4</td>
<td>Excavation to -1.95m and construction of the roof</td>
<td>26-06-2006</td>
<td>24-08-2006</td>
</tr>
<tr>
<td>5</td>
<td>Groundwater lowering for 1st and 2nd sand layer to NAP-10m.</td>
<td>15-07-2007</td>
<td>16-06-2007</td>
</tr>
<tr>
<td>6</td>
<td>Excavation to -6.2m</td>
<td>14-05-2007</td>
<td>18-06-2007</td>
</tr>
<tr>
<td>7</td>
<td>Strut installation at -5.6m</td>
<td>24-08-2007</td>
<td>25-08-2007</td>
</tr>
<tr>
<td>8</td>
<td>Excavation to -10.3m</td>
<td>08-10-2007</td>
<td>14-10-2007</td>
</tr>
<tr>
<td>9</td>
<td>Strut installation at -9.6m</td>
<td>07-11-2007</td>
<td>08-11-2007</td>
</tr>
<tr>
<td>10</td>
<td>Excavation to -15.3m</td>
<td>17-12-2007</td>
<td>23-12-2007</td>
</tr>
<tr>
<td>11</td>
<td>Strut installation at -14.7m</td>
<td>09-01-2008</td>
<td>10-01-2008</td>
</tr>
<tr>
<td>12</td>
<td>Excavation to -19.6m</td>
<td>11-02-2008</td>
<td>17-02-2008</td>
</tr>
<tr>
<td>13</td>
<td>Floor installation</td>
<td>30-03-2008</td>
<td>27-04-2008</td>
</tr>
<tr>
<td>14</td>
<td>Strut release at -14.7m</td>
<td>20-07-2008</td>
<td>21-07-2008</td>
</tr>
<tr>
<td>15</td>
<td>Excavation to -25.6m</td>
<td>25-08-2008</td>
<td>31-08-2008</td>
</tr>
<tr>
<td>16</td>
<td>Strut installation at -25m</td>
<td>04-11-2008</td>
<td>05-11-2008</td>
</tr>
<tr>
<td>17</td>
<td>Excavation to – 31.1m with pressurized chamber (air pressure)</td>
<td>04-06-2009</td>
<td>12-06-2009</td>
</tr>
<tr>
<td>18</td>
<td>Drainage layer (sand) installation</td>
<td>12-06-2009</td>
<td>16-09-2009</td>
</tr>
<tr>
<td>19</td>
<td>Installation of base floor, chamber was depressurised</td>
<td>16-10-2009</td>
<td>17-02-2010</td>
</tr>
<tr>
<td>20</td>
<td>Strut release at -25m</td>
<td>17-02-2010</td>
<td>∞</td>
</tr>
</tbody>
</table>

**3.5.3 Diaphragm Wall Properties**

The diaphragm wall used in the excavation was 1.2m thick. At Ceintuurbaan station, the top of the diaphragm wall was located at NAP -0.8m and it was embedded to NAP -45m. The tabulated installation sequence at Ceintuurbaan station can be found in APPENDIX D.
The engineering properties of the diaphragm wall installed are summarized in Table 3.5.

**Table 3.5  Engineering properties of diaphragm wall**

<table>
<thead>
<tr>
<th>Engineering Properties</th>
<th>Lower Bound Value</th>
<th>Average Value</th>
<th>Upper Bound Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete quality</td>
<td>B25, characteristic $f'_c$ is 25MPa</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stiffness modulus $E_b$ [kN/m²]</td>
<td>$1.0 \times 10^7$</td>
<td>$2.0 \times 10^7$</td>
<td>$3.0 \times 10^7$</td>
</tr>
<tr>
<td>Axial stiffness $EA$ [kN/m]</td>
<td>$9.6 \times 10^6$</td>
<td>$1.2 \times 10^7$</td>
<td>$1.8 \times 10^7$</td>
</tr>
<tr>
<td>Bending stiffness $EI$ [kNm²/m]</td>
<td>$1.2 \times 10^6$</td>
<td>$1.4 \times 10^6$</td>
<td>$2.2 \times 10^6$</td>
</tr>
<tr>
<td>Volumetric unit weight $\gamma_b$ [kN/m³]</td>
<td>23</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 3.5.4 Struts Properties

Cylindrical steel struts were used as a measure to maintain the stiffness of the excavation system, so that, the wall displacement could be limited in the desired range. The struts were installed in every 5m of horizontal distance in average, from centre to centre. There were 4 levels of struts, at NAP-5.6m, NAP -9.6m, NAP -14.7m, NAP -25.0m. In average, the smallest diameter is 660mm with 22mm wall thickness installed at NAP -5.6m and the largest is 1067mm with 20mm thickness. The maximum yield stress of the steel struts is 240MPa with stiffness value of 210GPa. Prior to the installation, the struts were pre-stressed. The properties are given in the following table.

**Table 3.6  Struts properties**

<table>
<thead>
<tr>
<th>Struts</th>
<th>A [mm²]</th>
<th>$EA$ [kN/m]</th>
<th>Prestress force [kN]</th>
<th>$F_{max}$ tension = $F_{max}$ compression [kN]*</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st level of struts</td>
<td>44073</td>
<td>9255338</td>
<td>3000</td>
<td>10580</td>
</tr>
<tr>
<td>2nd level of struts</td>
<td>57993</td>
<td>12178459</td>
<td>5500</td>
<td>13920</td>
</tr>
<tr>
<td>3rd level of struts</td>
<td>65752</td>
<td>13807836</td>
<td>8250</td>
<td>15780</td>
</tr>
<tr>
<td>4th level of struts</td>
<td>65752</td>
<td>13807836</td>
<td>4000</td>
<td>15780</td>
</tr>
</tbody>
</table>

*) obtained by using $\sigma_{yield}A$. $\sigma_{yield}$ of the struts is 240MPa and the stiffness is 210GPa.

The concrete floors installed for the platform of the metro line can be considered as additional struts. The exact engineering properties of the concrete floor are not available, but in the subsequent analysis, similar concrete properties as in the diaphragm wall will be applied.

### 3.5.5 Jet Grout Strut

In addition to the steel struts, jet grout struts were designed to be an extra measure to limit soil displacement adjacent to the pit. Yet, the design of the jet grout strut was not intended to provide very stiff excavation system. Too stiff jet grout strut will result in excessive bending moment in the diaphragm wall, which ultimately can crack the concrete. On the other hand,
too flexible behaviour was not expected since it will cause large soil deformation. Somehow, the installed jet grout strut should balance both aspects. Therefore, the expected stiffness was in the range of 1000-2200MPa.

The design level of the jet grout strut installation was NAP -33m to NAP -34.5m, approximately 1.5m thick column. The diameter of each column is 2.2m and the centre to centre distance is 1.8m. The layout of the applied jet grout strut in the project is illustrated in Figure 3.20. The gaps observed in the figure were intentionally left as a way to reduce the stiffness of the grout body, in the purpose to give a balanced stiffness.

Even this technique has been widely used to improve soil condition, many uncertainties still remain, especially the stiffness behaviour obtained after the installation. This parameter will give significant impact to the deformation analysis of the excavation. The strict requirement of the stiffness added some complexities in the execution.

![Figure 3.20 Jet Grout Strut Layout](image)

To eliminate the uncertainties, some field trials were performed and 14 grout body was cored and tested in the laboratory. According to the results, it turned out that the creep behaviour can be observed. The stiffness of the sample was reduced with time, as shown in Figure 3.21. The reduction could reach up to 80% of the initial value. It should be noted that the tests were conducted with a relatively small sample, 33-100mm diameter.

Later, it was decided to prove the behavior in the field with the design dimension. Back analysis by comparing the calculated and measured wall deflection, was performed to re-check the stiffness parameter obtained in the field. From the observation and the back analysis, a conclusion was drawn that the creep behavior did not occur in the field. This means that in a large diameter of jet grout column, creep can be able to be eliminated.

Based on the analysis of Delfgaauw et al [2009], elastic modulus of 2000-3800MPa can be expected. Observing the requirement of the stiffness value, the obtained result has certainly exceeded. Alternatively as mentioned above, several gaps between jet grout column were intentionally left, to lower the overall stiffness.
3.5.6 Air Pressure

It can be observed in Table 3.4, that the main excavation involved a pressurized excavation with air pressure, particularly at the deepest level. This is due to the reason that the thickness of the layer from Eem clay down to Warvan clay will not be sufficient to withstand the phreatic water pressure of the third sand layer, and warrant the stability of the excavation against basal heave. The air pressure applied in the excavation was 0.65bar. However, it was only a temporary measure until the drainage layer and the deepest floor were installed.
4. Soil and Buildings Movements at 13110E Based on Monitoring Results

In this chapter, the measurement data around 13110E (East) will be peeled. The displacement of the soils and buildings will be discussed as a basis for the respective analysis. In the discussion, the total settlement induced by different project stages will be separated, and it is expected that it will enable us to observe each phenomenon in detail. Consequently, further analysis result will be able to be simply confirmed for each stage.

4.1 Measurement Points

Figure 4.1 shows cross section 13110, which stretches along Eerste Jan van der Heijdenstraat, and the locations of the extensometers installed. It can be observed that 4 extensometers are located at each side of the street. For further convenience, the west side of the street will be referred to 13110W and the east side will be 13110E. In the picture, the identification of each extensometer is also mentioned. These extensometers identification will be used in the later discussion. The identification of prisms measurement point, ground surface manual levelling points, or building manual levelling points, will be explained exclusively when it is necessary.
4.2 Autonomous Settlement Prior to Project Execution

Generally, the subsurface of Amsterdam still experiences settlement. This is triggered by the time dependent settlement occurring in Holocene layers. Due to the continuous settlement, in order to maintain the surface level, the local government needs to increase the sand fill several times. Certainly, this leads to the larger settlement. The proof of this phenomenon can be clearly observed by the presence of the thick anthropogenic layer (Ophooglaag) and the compressed peat layer. The peat layer was originally several meters thick, and recently, it remains to approximately 1m thick.

Tracing back to the geological knowledge explained in chapter 3.2, the stable layer can be found starting from Third Sand layer down to deeper strata, which was consolidated in the glacial period. Theoretically, these layers do not have any movements. It also explains the reason why third sand layer became the measurement reference datum.

Cook et al [2007] suggested that surface settlement at Ceintuurbaan prior to the construction is approximately 2.2mm/year, while the first sand layer’s settlement is 0.4mm/year. The autonomous building settlement at Ceintuurbaan is approximately 2.0mm/year. Korff [2012] suggested slightly different values. She reported that the average surface settlement at Ceintuurbaan is approximately 3.5mm/year. While, the buildings settle down approximately 10mm over 8 years, which is equal to 1.25mm/year.

Hogenes [1998] analysed the stability of the NAP levels which is the basic of the measurement for the buildings and soils settlement. He discovered that NAP levels settled down with a rate of 0.9mm/year between 1927 and 1998. He noted as well that the Eem clay also experiences creep with a rate of 0.35mm/year. As a conclusion, Hogenes took the third sand layer as the reference level and he suggested that the general rate of building settlement in Amsterdam is approximately 4mm/year with reference to the third sand layer. This value is indeed larger than the value of the surface settlement at Ceintuurbaan. Since the value suggested by Hogenes was derived from general Amsterdam condition, not for a specific location, it will only be treated as a qualitative comparison and not be taken as a representative value for Ceintuurbaan station.

The autonomous settlement of the soils and the buildings can be summarized as in Table 4.1.

<table>
<thead>
<tr>
<th>Range</th>
<th>2.2-3.5 [mm/year]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface settlement</td>
<td></td>
</tr>
<tr>
<td>First sand layer</td>
<td>0.4 [mm/year]</td>
</tr>
<tr>
<td>Eem clay</td>
<td>0.35 [mm/year]</td>
</tr>
<tr>
<td>Buildings</td>
<td>1.25-2.0 [mm/year]</td>
</tr>
</tbody>
</table>

The surface settlement discussed above can be observed in the extensometer measurements, which are presented in Figure 4.2. The data was selectively taken from 13110E. From the figure, a clear pattern can be observed that in a period of almost 2 years, the settlement of the
surface level is approximately 5-8mm, with 6mm in average. This leads to a surface settlement of 3mm/year which conforms to the value in Table 4.1.

**Figure 4.2 Surface settlement at 13110E prior to project execution**

Different from the previous result, various values are obtained in the measurement at the first sand layer, as observed in Figure 4.3. Some extensometers suggested that heave phenomenon can be found, but on the other hand, the other extensometers indicate that the first sand layer settles about 0.5-1.4mm in almost 2 years. No clear pattern can be concluded here and it is suspected that the accuracy problem occurs in this depth. Considering the value suggested in Table 4.1, the most representative extensometers are 06150402 and 06150403. In the period prior to the project execution, 06150402 suggested 0.5mm of settlement (0.25mm/year) and 06150403 suggested 1.4mm of settlement (0.7mm/year).

Autonomous settlement can also be observed in the second sand layer. Figure 4.4 demonstrates the settlement pattern. Despite of the outlier from 06150404, the settlement in nearly 2 years of monitoring is between 0.2-1.0mm, which is equal to 0.5mm/year of maximum settlement. These values from both sand layers are close to 0.4mm/year in Table 4.1. Furthermore, the values are also close to the creep settlement of Eem clay, which leads to the conclusion that the settlement of the layers between third sand layer and Holocene layers is governed by Eem clay behavior.
It can be deducted that the most significant settlement occurs in the Holocene layers. Considering the type of the material in this shallow subsurface depth (chapter 3.5.1), it can be speculated that the peat layers, Hollandveen and Basisveen, are the main driving factor for the settlement.
4.3 Soil Settlement Induced by The Project

4.3.1 Overview of ground displacement by Korff [2012]

A rigorous study by Korff [2012] has successfully analysed the surface settlement of the soil. Figure 4.5 presents the normalized settlements obtained from the manual surface levelling at Ceintuurbaan station. As observed, Korff has compiled the surface settlement in different stages, including preliminary work. It should be noted that the preliminary work defined by Korff [2012] includes the first stage of the excavation. It was suggested that the shape of the total normalised settlements conforms to the shape suggested by Peck [1969], particularly to category I. According to Peck, category I includes sand and soft to stiff clay condition, which conforms to the condition of the project.

![GroundSurface CTB](image)

**FIGURE 4.5 NORMALIZED SURFACE SETTLEMENT DURING THE ENTIRE PERIOD (KORFF [2012])**

Furthermore, Korff emphasised that 70-75% of the settlement of the soils occurred during the preliminary works. Table 4.2 shows the percentage of settlement due to the preliminary work in different stations. Vijzelgracht shows abnormal behaviour, but, it can be argued that the settlement during the excavation increased significantly due to the effect of the leakage incident. To observe the proportion of the settlement induced by the project, next several chapters describes it in more detail.

<table>
<thead>
<tr>
<th>Stations</th>
<th>Percentage of settlement due to preliminary work</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rokin</td>
<td>70 %</td>
</tr>
<tr>
<td>Vijzelgracht</td>
<td>55%</td>
</tr>
<tr>
<td>Ceintuurbaan</td>
<td>75%</td>
</tr>
</tbody>
</table>
### 4.3.2 Settlement due to Additional Fill

The placement of additional fill prior to the excavation is intended to provide a working platform for the installation of the diaphragm wall. Due to the nature of the Holocene layers, certainly, additional settlement can be expected. Officially, this period occurred from 15 November 2003 to 8 April 2004 for entire Ceintuurbaan station. The settlement induced by this event can be observed in Figure 4.6.

Figure 4.6 also clearly explains that the influence of the additional sand fill reduced with distance. The most influenced extensometers are 06150401 and 06150402, which are 3m and 8m away from the edge of the excavation (Figure 3.11), respectively. In total, the surface settlement induced at 06150401 is 10mm, while at 06150402 is 2mm. On the other hand, no influence of this event can be observed at 06150403 and 06150404. The observation for this period was taken based on the settlement measured from the additional fill placement until the installation of first diaphragm panel related to 13110E was installed.

Further observation is made by pointing the attention to the deeper layer, especially the response of first and second sand layer (Figure 4.7 and Figure 4.8). Surprisingly, both layers responded differently from Holocene layers. Measurement data shows that the layers heaved, exactly starting on 1st February 2004. The heave gradually reduces and returns to the original position. In overall, no noticeable settlement occurs in the layers below Holocene layers.

Table 4.3 is presented below and it summarizes the settlement value for measurement points at the surface, first and second sand layer.

<table>
<thead>
<tr>
<th>Extensometers</th>
<th>Surface settlement</th>
<th>First sand layer settlement</th>
<th>Second sand layer settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>06150401</td>
<td>10.0mm</td>
<td>0.0mm</td>
<td>0.0mm</td>
</tr>
<tr>
<td>06150402</td>
<td>2.0mm</td>
<td>0.0mm</td>
<td>0.0mm</td>
</tr>
<tr>
<td>06150403</td>
<td>1.0mm</td>
<td>0.0mm</td>
<td>0.0mm</td>
</tr>
<tr>
<td>06150404</td>
<td>1.0mm</td>
<td>0.0mm</td>
<td>0.0mm</td>
</tr>
</tbody>
</table>
Figure 4.6 Surface settlement during sand fill placement

Figure 4.7 Response of first sand layer to sand fill placement
4.3.3 Settlement due to Diaphragm Wall Installation

After the sand fill placement period, the diaphragm wall panels were installed. However, the period of this event presented in chapter 3.5.2 includes the installation period for the entire diaphragm wall panels' adjacent to 13110. Therefore, to assess soil settlement at 13110E more accurately, attention will be paid to the installation schedule of the closest panels to 13110E.

The layout of the panels is presented Figure 4.9. It is believed that the settlement of the soil at 13110E does not solely depend on the installation of panel 21, rather, there are contributions from several panel installations. Therefore, the effect of several panel installations was observed in the settlement monitoring and it turned out that the influential panel installations were from panel 19 to panel 23 (5 panels).

The schedule of the panel installation is presented in Table 4.4. To give clearer understanding, the table is sorted based on the schedule, not based on the panel’s number.

<table>
<thead>
<tr>
<th>Panel</th>
<th>Start Excavation</th>
<th>Start concrete pouring</th>
<th>Finish concrete pouring</th>
</tr>
</thead>
<tbody>
<tr>
<td>22</td>
<td>20-08-2004</td>
<td>24-08-2004</td>
<td>24-08-2004</td>
</tr>
<tr>
<td>23</td>
<td>30-08-2004</td>
<td>01-09-2004</td>
<td>01-09-2004</td>
</tr>
<tr>
<td>19</td>
<td>06-09-2004</td>
<td>07-09-2004</td>
<td>07-09-2004</td>
</tr>
<tr>
<td>21</td>
<td>10-09-2004</td>
<td>13-09-2004</td>
<td>13-09-2004</td>
</tr>
<tr>
<td>20</td>
<td>14-09-2004</td>
<td>16-09-2004</td>
<td>16-09-2004</td>
</tr>
</tbody>
</table>
Figure 4.9 Layout of Diaphragm Wall Panels at 13110

Figure 4.10 to Figure 4.12 demonstrate the settlement occurred in the measurement points during and after installation. Further later, it can be observed that the settlement caused by diaphragm wall installation did not occur directly, rather, it needed some time. Hence, two figures are presented for each measurement points. The overall figures are intended to show the long term condition including the period of installation and after installation. While the detail figures represent the short term effect of installation.

In short term observation, evidently, the soils at the surface behave differently with respect to each panel installation. The soils at the surface merely respond to installation of panel 19 and 22. This is contradictory to the deeper layers behaviour. These deeper layers responded to each panel installation shortly after the installation. There are no certain explanations about it and accuracy issue might explain this different behaviour. Furthermore, the direct influence of the installation seems to be substantial in 06150401 and 06150402, which are closer to the diaphragm wall, and the influence is eliminated in further distance.

Even the direct effect of panel installation only influences the closest extensometers, but the figures for long term condition suggest that settlement occurred in all of the extensometer points. Understandably, the long term settlement of the soils is triggered by the unloading during the panel excavation. Pore water pressure will be dramatically reduced and as it dissipates soil will experience consolidation settlement. As observed particularly in Figure 4.10 (measurement at the surface), the settlements of the soils flatten in the period close to
the jet grouting strut installation period. The flattened soils settlements are closely related to the small excess negative pore pressure left. Based on this measurement result, it can be concluded that greatest effect of diaphragm wall installation effect is not in short term period, rather, long term effect will be more significant.

The summary of the long term settlement is presented in Table 4.5. These long term settlements are measured from the moment when first panel was installed until the installation of jet grouting. Some extensometer points measured show unexpected and odd values, i.e sudden jump which is considered unrealistic, hence, this will be noted as not applicable (NA) in the table.

**Table 4.5 Soils settlements caused by diaphragm wall installation**

<table>
<thead>
<tr>
<th>Extensometers</th>
<th>Surface long term settlement</th>
<th>First sand layer long term settlement</th>
<th>Second sand layer long term settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>06150401</td>
<td>7.0mm</td>
<td>2.0mm</td>
<td>1.0mm</td>
</tr>
<tr>
<td>06150402</td>
<td>8.0mm</td>
<td>1.5mm</td>
<td>NA</td>
</tr>
<tr>
<td>06150403</td>
<td>5.0mm</td>
<td>1.0mm</td>
<td>0.0mm</td>
</tr>
<tr>
<td>06150404</td>
<td>5.0mm</td>
<td>NA</td>
<td>0.0mm</td>
</tr>
</tbody>
</table>

13110E - Surface: Diaphragm wall installation
20 Aug 2004 - 14 Oct 2005
Figure 4.10 Surface settlement during diaphragm wall installation, overall (above) and in detail (below)
Figure 4.11 First sandlayer settlement during diaphragm wall installation, overall (above) and in detail (below).
4.3.4 Settlement due to Jet Grout Strut

Jet grout strut is mainly intended to reduce deformation of soil adjacent to the excavation. It is installed by eroding soil with hydraulic fracturing. Paradoxically, instead of reducing soil deformation, the fracturing process potentially induces excessive soil deformation and stress relief. Sometimes, it is not uncontrollable due to complexity of the installation process and the nature of the soil.

However, in some deep excavation projects involving jet grout column installation, the main goal is feasible to be achieved and the settlement of the soils behind the wall can be limited, as observed and reported by Wong and Poh [2000] in a deep excavation project in Singapore Post Centre. Jet grout columns were installed roughly 2 weeks after the closest panel was cast. Increasing pore water pressure was observed in the soils behind the panel, and it was accompanied by heaving and increasing earth pressure. This event is understandable since the installation of jet grouting involves high fluid pressure applied to the soil. Moreover, the panel installed earlier is potentially still ‘soft’ and not completely cured within 2 weeks after installation.

Figure 4.13 to Figure 4.15 demonstrate that this case did not occur in North/South Line case. Practically, no vertical displacements are observed in the measurement data. The main reason is that, the interval between the installation of diaphragm wall panels and jet grouting column was fairly long, approximately 1 year. By the time when the jet grout column would be installed, the diaphragm wall panels had fully been cured. Certainly, the increasing stress due to the jet grouting process could be absorbed by diaphragm wall panels, and therefore,
no stress attenuates to the soils behind the diaphragm wall. Moreover, the hydraulic fracturing process in the soil is also hindered by the diaphragm wall, resulting in no stress relief in the soil behind the excavation.

**Figure 4.13** Surface settlement in jet grout strut installation

**Figure 4.14** First sand layer settlement during jet grout strut installation
4.3.5 Settlement due to Pumping Test

Following the jet grout strut installation, there was a ‘waiting period’ before the excavation of the pit was executed. During this period, pumping test was conducted in the (future) excavated area and the targets were the first, second, and intermediate sand layer. This was intended to simulate the future pumping activity which took place later in the actual excavation scheme. In chapter 3.5.1, it has been explained that the aforementioned sand layers have relatively high phreatic levels which is able to exert uplift force and ultimately induce basal instability. Therefore, dewatering was required to secure the vertical equilibrium of the base of the pit when the excavation proceeded to greater depth.

The tests were done twice, on 7-10 March 2006 and 5-10 April 2006 for 13110E. The scheme is summarised as follows:

- 7-10 March 2006:
  In the first pumping test, the First and Second Sand layer’s phreatic levels were lowered from NAP -0.5 to NAP -0.6m. Also, pumping test was performed in Tussenzandlaag. It was dewatered from NAP -3m to NAP-7m. Then, the phreatic level returned to the normal condition.

- 5-10 April 2006:
  The second pumping test was performed in First and Second Sand layer from NAP -0.5m to NAP -10.5m and in Tussenzandlaag from NAP -3m to NAP -30m.
The results of the observation on the extensometer suggest that only the surface reacted to this activity as indicated in Figure 4.16. On the other hand, relatively no displacements occur in first and second sand layer as shown by Figure 4.17 Figure 4.18. It implies that the only Holocene layers were affected.

But, it should be noted that the diaphragm wall acts as a seal and therefore, there should be no direct effect of the pumping test. Rather, the deeper layer should have been influenced more significantly and then the source of the settlement comes from the deeper layer. Using this point of view, the settlement of the surface, First and Second sand layer should have been identical.

There are no clear explanations about the behaviour of the First and Second sand layer according to the extensometers. One possible explanation is the inaccuracy of the extensometer especially when it is subjected to a very small settlement value.

![Figure 4.16 Surface settlement during pumping test](image-url)

**Figure 4.16 Surface settlement during pumping test**
**Figure 4.17 First sand layer settlement during pumping test**

**Figure 4.18 Second sand layer settlement during pumping test**
The summary of the settlement during pumping test is tabulated in a table below.

**Table 4.6 Summary of Soil Settlement due to Pumping Test**

<table>
<thead>
<tr>
<th>Extensometers</th>
<th>Surface settlement</th>
<th>First sand layer settlement</th>
<th>Second sand layer settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>06150401</td>
<td>3mm</td>
<td>1mm</td>
<td>0mm</td>
</tr>
<tr>
<td>06150402</td>
<td>4mm</td>
<td>1mm</td>
<td>0mm</td>
</tr>
<tr>
<td>06150403</td>
<td>2mm</td>
<td>0mm</td>
<td>0mm</td>
</tr>
<tr>
<td>06150404</td>
<td>2mm</td>
<td>0mm</td>
<td>0mm</td>
</tr>
</tbody>
</table>

There is a slight remark about the condition before pumping test was performed. Surface layers indicate small magnitude of settlement between the end of jet grouting period and the start of pumping test period. There is no clear information about some activities performed in 4 months period. Observing the effect of diaphragm wall installation, it can be concluded that consolidation settlement has been accomplished. Jet grouting period also gives no significant influences. Therefore, in further analysis, this event will be overlooked.

### 4.3.6 Settlement due to Main Excavation

The settlements during the main excavation are presented in Figure 4.19 to Figure 4.21. From these figures, it is clear that the settlements of the soils reduce with distance. The settlements from the extensometers are tabulated in the table below. The observation is started from 26 June 2006 when the first excavation was started, and it is ended until 21 August 2009.

Despite some odd spikes, the extensometers show similar pattern for the entire period of the main excavation.

**Table 4.7 Settlements of Soils during Main Excavation**

<table>
<thead>
<tr>
<th>Extensometers</th>
<th>Surface settlement</th>
<th>First sand layer settlement</th>
<th>Second sand layer settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>06150401</td>
<td>22mm</td>
<td>12mm</td>
<td>4mm</td>
</tr>
<tr>
<td>06150402</td>
<td>28mm</td>
<td>10mm</td>
<td>4mm</td>
</tr>
<tr>
<td>06150403</td>
<td>25mm</td>
<td>6mm</td>
<td>0mm</td>
</tr>
<tr>
<td>06150404</td>
<td>15mm</td>
<td>2mm</td>
<td>0mm</td>
</tr>
</tbody>
</table>
FIGURE 4.19 SURFACE SETTLEMENT DURING MAIN EXCAVATION

13110E - Surface: Main Excavation
26 June 2006 - 21 Aug 2009

FIGURE 4.20 FIRST SAND LAYER SETTLEMENT DURING MAIN EXCAVATION

13110E - 1st Sand Layer: Main excavation
26 June 2006 - 21 Aug 2009
### 4.3.7 Summary of Soil Settlements

Table 4.8 tabulates the total settlement from different levels discussed above. It can be concluded that there is a distance effect on the influence of the excavation to the soil settlement.

<table>
<thead>
<tr>
<th>Level</th>
<th>06150401</th>
<th>06150402</th>
<th>06150403</th>
<th>06150404</th>
</tr>
</thead>
<tbody>
<tr>
<td>NAP 0.4m</td>
<td>45mm</td>
<td>44mm</td>
<td>36mm</td>
<td>23mm</td>
</tr>
<tr>
<td>NAP -12m</td>
<td>15mm</td>
<td>12.5mm</td>
<td>7mm</td>
<td>2mm</td>
</tr>
<tr>
<td>NAP -20m</td>
<td>5mm</td>
<td>4mm</td>
<td>1mm</td>
<td>0mm</td>
</tr>
</tbody>
</table>

The total settlement listed above can be broken down in order to observe the most influential stages in the project. The percentage can be observed in Table 4.9 to Table 4.11.

Based on the table, the biggest influence among the project activities still relies on the main excavation part. However, it cannot be denied that the preparation part has taken quite a large portion, especially at the surface. The percentage of settlement at the surface induced by the preparation stage ranges from 31% to 51%, on the other hand, at the deeper layer, the main excavation period dominates the influence. The preparation stage here is defined as the period from the additional fill placement to the period before the first excavation was executed.
Interestingly, the influence of diaphragm wall installation period is quite significant in overall, except at 06150401 which is close to the location of additional fill placement. This phenomenon suggests that diaphragm wall installation should be carefully considered during the project design and execution beside the main excavation part. Certainly this period cannot be overlooked and it will be included in the further analysis and some specific issues related to this activity will be discussed later.

### Table 4.9 Influence of each construction activity to soil settlement at the surface

<table>
<thead>
<tr>
<th>Period</th>
<th>06150401</th>
<th>06150402</th>
<th>06150403</th>
<th>06150404</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total settlement</td>
<td>%</td>
<td>Total Settlement</td>
<td>%</td>
</tr>
<tr>
<td>Additional fill placement</td>
<td>10mm</td>
<td>22%</td>
<td>2mm</td>
<td>5%</td>
</tr>
<tr>
<td>Diaphragm wall installation</td>
<td>7mm</td>
<td>15%</td>
<td>8mm</td>
<td>18%</td>
</tr>
<tr>
<td>Jet grout column installation</td>
<td>0mm</td>
<td>0%</td>
<td>0mm</td>
<td>0%</td>
</tr>
<tr>
<td>Waiting period</td>
<td>2mm</td>
<td>4%</td>
<td>2mm</td>
<td>5%</td>
</tr>
<tr>
<td>Pumping test</td>
<td>3mm</td>
<td>7%</td>
<td>4mm</td>
<td>9%</td>
</tr>
<tr>
<td>Waiting period</td>
<td>1mm</td>
<td>2%</td>
<td>0mm</td>
<td>0%</td>
</tr>
<tr>
<td>Excavation</td>
<td>22mm</td>
<td>49%</td>
<td>28mm</td>
<td>64%</td>
</tr>
</tbody>
</table>

### Table 4.10 Influence of each construction activity to soil settlement at foundation level (NAP-12m)

<table>
<thead>
<tr>
<th>Period</th>
<th>06150401</th>
<th>06150402</th>
<th>06150403</th>
<th>06150404</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total settlement</td>
<td>%</td>
<td>Total Settlement</td>
<td>%</td>
</tr>
<tr>
<td>Additional fill placement</td>
<td>0mm</td>
<td>0%</td>
<td>0mm</td>
<td>0%</td>
</tr>
<tr>
<td>Diaphragm wall installation</td>
<td>2mm</td>
<td>13%</td>
<td>1.5mm</td>
<td>12%</td>
</tr>
<tr>
<td>Jet grout column installation</td>
<td>0mm</td>
<td>0%</td>
<td>0mm</td>
<td>0%</td>
</tr>
<tr>
<td>Waiting period</td>
<td>0mm</td>
<td>0%</td>
<td>0mm</td>
<td>0%</td>
</tr>
<tr>
<td>Pumping test</td>
<td>1mm</td>
<td>7%</td>
<td>1mm</td>
<td>8%</td>
</tr>
<tr>
<td>Waiting period</td>
<td>0mm</td>
<td>0%</td>
<td>0mm</td>
<td>0%</td>
</tr>
<tr>
<td>Excavation</td>
<td>12mm</td>
<td>80%</td>
<td>10mm</td>
<td>80%</td>
</tr>
</tbody>
</table>

### Table 4.11 Influence of each construction activity to soil settlement at NAP-20m

<table>
<thead>
<tr>
<th>Period</th>
<th>06150401</th>
<th>06150402</th>
<th>06150403</th>
<th>06150404</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total settlement</td>
<td>%</td>
<td>Total Settlement</td>
<td>%</td>
</tr>
<tr>
<td>Additional fill placement</td>
<td>0</td>
<td>0%</td>
<td>0</td>
<td>0%</td>
</tr>
<tr>
<td>Diaphragm wall installation</td>
<td>1</td>
<td>20%</td>
<td>0</td>
<td>0%</td>
</tr>
<tr>
<td>Jet grout column installation</td>
<td>0</td>
<td>0%</td>
<td>0</td>
<td>0%</td>
</tr>
<tr>
<td>Waiting period</td>
<td>0</td>
<td>0%</td>
<td>0</td>
<td>0%</td>
</tr>
<tr>
<td>Pumping test</td>
<td>0</td>
<td>0%</td>
<td>0</td>
<td>0%</td>
</tr>
<tr>
<td>Waiting period</td>
<td>0</td>
<td>0%</td>
<td>0</td>
<td>0%</td>
</tr>
<tr>
<td>Excavation</td>
<td>4</td>
<td>80%</td>
<td>4</td>
<td>100%</td>
</tr>
</tbody>
</table>
4.4 Buildings Settlement Subjected to Project Activities

4.4.1 Overall Buildings Settlement During The Project Execution

Figure 4.22 demonstrates the overall responses of the buildings, from the first 2 years of monitoring until the end of the excavation period. The settlement values in the figure are taken from the manual levelling data for each building, which were recorded regularly. Consequently, the values presented in the figure show the absolute settlement.

**Figure 4.22 Overall buildings settlement at Ceintuurbaan station measured between week 20 2001 to week 52 2010 (taken from www.webgis.mobonz.nl)**

The figure above shows us that the most of the closest buildings suffer the largest settlement compared to other buildings. Even it is not clear, there are some buildings in further distance to the station experience smaller settlement than the closer ones. Nevertheless, there are some ‘outliers’. Several buildings in further distance performed as poor as the closest buildings. The plausible explanation is the condition of the timber pile foundation itself, either it has been heavily overloaded or broken prior to the construction of the project.

4.4.2 Buildings Responses in Specific Project Activities

Before discussing further, it should be noted that buildings settlements in this particular chapter are assessed based on the prism monitorings which cannot offer the absolute settlement value. As mentioned earlier in chapter 3.4.2, prism monitoring depends on the reference building, thus, the data obtained from the measurements are relative displacement values. Consequently, one cannot compare it directly to the settlement in the previous subchapter.

However, this chapter is presented to give an insight about the percentage of building settlement induced by each project activities. Similar to chapter 4.3.7, it is expected that we can observe the most influential period during the project execution. In the subsequent
analysis, data obtained from manual levelling on buildings will be the basis on the validation.

Prism monitoring results are chosen due to the intensity of the measurement. Comparing to the manual levelling, prism measurements were undertaken frequently, this fact enables us to observe buildings behaviour in a short period in detail.

In order to look into the details more conveniently, several buildings are chosen exclusively. They are located in cross section 13110E. The observation does not focus on every prism monitoring points installed in the buildings, rather, it has been narrowed down to only 3 monitoring points from 3 different buildings. The buildings and the measurement points are:

- Ferdinand Bolstraat 93, based on prism monitoring point 608100930304.
- Ferdinand Bolstraat 95, based on prism monitoring point 607100950302.
- Eerste Jan van der Heijdenstraat 92, based on prism monitoring point 607920301.

Each location of the buildings can be observed in Figure 4.1. Ferdinand Bolstraat 93 and 95 correspond to extensometer 06150401. Eerst Jan van der Heijdenstraat 92 corresponds to extensometer 06150404.

The settlement of the buildings can be observed in Figure 4.23 to Figure 4.28. In the initial condition, some movements can be observed in 2 prism measurement points, 607100930304 and 607100950302. Both measurement points were lifted up 2.5-5mm. But, these values are questionable and it can be considered that the buildings settled relatively 0mm. The uplift might have occurred due to the settlement of the reference buildings. In this particular event, the settlement values of the monitored buildings is regarded as invalid. Remarkable displacements are observed during the diaphragm wall installation, pumping test, and main excavation period. No significant settlements occurred during the other period. The settlement of the buildings are tabulated in Table 4.12.

<table>
<thead>
<tr>
<th>Table 4.12 Building settlements during project execution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Period</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Additional fill placement</td>
</tr>
<tr>
<td>Diaphragm wall installation</td>
</tr>
<tr>
<td>Jet grout column installation</td>
</tr>
<tr>
<td>Waiting period</td>
</tr>
<tr>
<td>Pumping test</td>
</tr>
<tr>
<td>Waiting period</td>
</tr>
<tr>
<td>Excavation</td>
</tr>
</tbody>
</table>

From the table above, it can be deducted that similar to the influence in soil settlement, main excavation plays major role in inducing buildings settlement. 41-67% of buildings settlements are obtained during the main excavation. The data, again, show that the percentage of the preparation period is dominated by diaphragm wall installation period.
**Figure 4.23** Buildings settlements in initial condition

**Figure 4.24** Buildings settlements due to the placement of additional sandfill
FIGURE 4.25 BUILDING SETTLEMENTS DUE TO DIAPHRAGM WALL INSTALLATION, OVERALL (ABOVE) AND IN DETAIL (BELOW)
Figure 4.26 Building settlement during jet grout column installation

Figure 4.27 Building settlement during pumping test period
4.5 Conclusion

Based on the results above, several conclusions can be made. It is evident that preliminary works are indispensable factor to be included in the analysis and design of deep excavation, especially in relation to the soil-structure interaction. As observed, building movement has been induced since the period of preliminary works, therefore, any damages could be triggered not only by the main excavation phases, but also there is a contribution of preliminary works.

Surface settlement induced by preliminary works is between 31%-51% of total settlement, while the contribution of preliminary works in the settlement of foundation layer (NAP - 12m) is 20% maximum. Based on the prism monitoring, building settlement induced during preliminary works period is between 33%-59%.

The most significant activity among preliminary works is the installation of diaphragm wall. In the surface, this activity contributes between 15%-22%, while at the foundation layer, the contribution is 13% maximum. The influence of diaphragm wall installation is more significant to the building settlement, with a range between 32%-41%.
5. Hypothesis

It has been observed in the previous analysis that the settlement of the buildings does not simply follow the settlement of the First Sand layer as the foundation layer. Rather, the buildings settle more than the foundation layer, but less than the surface.

Several hypotheses are summarised in this chapter which could explain the cause of building settlement and provide a basis for the analysis on the finite element results.

Korff [2012] postulated in her study that piled buildings are subjected to settlement as a response to deep excavation due to several effects:

- Settlement of the foundation layer.
- Reduction of pile capacity due to reduced stress level.
- Remobilisation of shaft friction, either positive or negative skin friction.
- Redistribution of pile load from the building.
- Horizontal pile deformation.

In this study, the effect of horizontal pile deformation and also redistribution of pile load will not be taken into account.

To give more comprehensive explanation the following passages will describe the possible triggering factors of pile settlement due to deep excavation.

5.1 Settlement at the foundation layer

This factor is indispensable in analyzing pile settlement. If the foundation layer settles down directly due to a specific event, driven pile will follow the settlement of the foundation layer before finding a new equilibrium in the load distribution and also the new position of the neutral plane. Therefore, in any case, the assessment of pile settlement should come to this particular point firstly. Further changes in pile shaft and the neutral plane will cost an additional settlement.

5.2 Influence of Stress Relief

The resistance of pile is obtained by the confinement of the adjacent soil to mobilise the shaft and the tip resistance. This confinement is governed by the normal stress acting on the pile surface.

During the construction stages, unloading will occur in the soil body, for instance during the diaphragm wall installation. It is inevitable that trenching is necessary to construct the diaphragm wall panels. Sometimes the trenches are left open for several days before concreting takes place. During this unloading period, it is likely that the stress in the soil decreases and finally the normal stress acting on the pile will be reduced. Regardless the
magnitude, the stress relief will definitely shift the pile tip resistance and shaft friction. Consequently, the pile will be required to find a new equilibrium and ultimately additional settlement will occur.

In a certain condition, the settlement due to this particular effect is not necessarily accompanied by the changes in neutral plane. The positive and negative skin friction could be reduced simultaneously in a same rate without changing the neutral plane.

5.3 Influence of Neutral Plane Position and Remobilisation of Pile Resistance

The position of the neutral level might not remain at the same level. The changes of the neutral plane can be triggered by the remobilisation of pile resistance, either at the tip or the shaft, as illustrated in a figure below.

![Figure 5.1 Illustration of change in neutral plane due to remobilisation of pile resistance](image1)

On the other hand, in case maximum pile resistance has been fully mobilised, the shift of the neutral plane is caused mainly by additional external load introduced to the pile as indicated in Figure 5.2.

![Figure 5.2 Rise of neutral plane](image2)
In other case, it is possible that neutral plane position remains at the same level and at the same time, additional settlement occurs. This basically means that the incremental settlements at the neutral plane, surface and the foundation layer are identical.

![Figure 5.3 Neutral plane remains at the same level due to a specific event](image)

Based on the basic concept of piles subjected to negative skin friction, the settlement of the pile subjected to a specific event will be equal to the soil settlement at the neutral level. Therefore, observing the potential changes in neutral plane and the relation of pile-soil displacement at the neutral plane, the understanding of the position of neutral plane is of importance.

Korff [2012] has analysed the ratio between the soil settlement and building settlement at several measurement points during the execution of the project at Ceintuurbaan Station. Korff assumes that the maximum pile resistance and negative skin friction have been mobilized due to time dependent settlement of Amsterdam subsurface.

In Figure 5.4, it is evident that the neutral level is initially located down close to the tip. The level is shifted up when the project progresses. Working with similar assumption adopted by Korff [2012] where positive and negative skin friction remain constantly in the maximum value through the entire project construction, the raise of interaction level/neutral level in Figure 5.4 should coincide with additional external load. The main basic of this statement is that in the condition where positive and negative skin friction have been fully mobilised, the shaft friction cannot be forced to increase. Theoretically, the only way to raise the neutral plane is to increase the external load at the pile head as illustrated in the following picture. However, it is unlikely that several buildings increased the loads in the same period.
Therefore, another factor must play a significant role and it is suspected that the changes in the neutral plane and remobilisation of pile resistance are 1 of the possible governing factor of pile settlement.
6. Three Dimensional Finite Element Analysis

The measured soil and building settlement have been presented and analysed. In order to give the explanation to the research objectives, finite element analysis will be employed to simulate the construction of Ceintuurbaan station. This chapter exclusively discusses about the finite element analysis and the results. Finite element analysis will be employed using commercial software, PLAXIS3D 2012.

As indicated in the scope of work, the buildings will be represented by a row of piles. Therefore, in this simulation, it is assumed that the settlement of the buildings is equal to the settlement of the piles.

In general, there are several major excavation stages modelled in the analysis. The simulation will start from 100 years ago to model the time dependent settlement and developed pile’s load distribution. After the simulation of 100 years period, the simulation will enter the construction stages of the excavation. The stages are explained as follows:

- First stage – modelling the settlement of 100 years of time dependent settlement.
- Second stage – modelling the sand fill placement period.
- Third stage – modelling diaphragm wall installation period.
- Forth stage – modelling pumping test period.
- Fifth stage – modelling the main excavation.

The initial focus of the analysis is to back analyse of the soil displacement. It has been understood that soil displacement has a close relationship to the pile response and hence, it is important to have the correct result in advance prior to analysis of pile behaviour. The available measurement data at 13110E will be the basis of the back analysis.

The original parameter sets provide a range of parameter values for each layer, as observed in APPENDIX B. In order to find the best match, several analyses were performed using different values of parameters. Two back analysis results are presented below. The first one is based on the mean value of the parameters. On the other hand, the second one is based on the adjusted parameter values.

Pile behaviour will be discussed based on the final analysis results.

6.1 The Model

The complete model is presented in Figure 6.3 below. The model has 31902 soil elements. The size of the model is 45m wide in x-direction, 70m long in y-direction, and 70m deep in z-direction. The boundary condition of the model is explained by Figure 6.2.
Figure 6.1 Complete 3D model

Figure 6.2 Boundary Condition (retrieved from `http://kb.plaxis.nl/tips-and-tricks/fixities-and-deformation-boundary-conditions-plaxis-3d`)

Figure 6.3 illustrates the plan view of the model. It can be observed that the piles are incorporated directly in the model. The piles are named according to the point load applied
at the pile head (first number) and also the distance to the diaphragm wall edge (last number). The piles are arranged so that the axis of the pile rows is situated at the real buildings’ edge. In the subsequent analysis, attention will be mostly drawn to pile 70-3 and 100-3.

![Plan View of the Model and Sign Convention](image)

**Figure 6.3 Plan View of the Model and Sign Convention**

The external load is applied using point load at the pile head. However, the actual transferred load from the building to the pile is uncertain. Based on the study of Frankenmolen [2006] and Korff [2012], it was decided to apply 70kN and 100kN to the pile. Embedded pile model is used to model the pile. In such a way, it will allow us to observe the full soil-structure interaction in deep excavation.

The supporting structure of the excavation consists of diaphragm wall and several layers of struts. The diaphragm wall will be modelled using volume material with linear elastic behaviour. While the struts are modelled using node to node anchors.

More detailed parameters of the soil and the other structures will be explained in chapter 6.5 and 6.6.

### 6.2 Adopted Soil Layers and Phreatic Levels

The soil layers adopted in the model are presented in a table below. The parameters of the soils will be discussed in detail in chapter 6.5 and 6.6.
### Table 6.1 Adopted soil layers and phreatic levels in simulation

<table>
<thead>
<tr>
<th>Layer name</th>
<th>Dutch name</th>
<th>Top Level [m]</th>
<th>Phreatic Level [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandfill</td>
<td>Ophooglaag</td>
<td>NAP + 0.4</td>
<td>NAP -0.5</td>
</tr>
<tr>
<td>Peat</td>
<td>Hollandveen</td>
<td>NAP - 3.0</td>
<td>NAP -0.5</td>
</tr>
<tr>
<td>Old sea clay</td>
<td>Oudezeeklei</td>
<td>NAP - 5.0</td>
<td>NAP -0.5</td>
</tr>
<tr>
<td>Wad deposit</td>
<td>Wadzandlaag</td>
<td>NAP - 6.4</td>
<td>NAP -0.5</td>
</tr>
<tr>
<td>Hydrobia clay</td>
<td>Hydrobiaklei</td>
<td>NAP - 10.5</td>
<td>NAP -0.5</td>
</tr>
<tr>
<td>Peat</td>
<td>Basisveen</td>
<td>NAP - 11.0</td>
<td>NAP -0.5</td>
</tr>
<tr>
<td>First sand layer</td>
<td>Eerstezandlaag</td>
<td>NAP - 11.5</td>
<td>NAP -3.0</td>
</tr>
<tr>
<td>Loam</td>
<td>Allerød</td>
<td>NAP - 14.0</td>
<td>NAP -3.0</td>
</tr>
<tr>
<td>Second sand layer</td>
<td>Tweedezand lag</td>
<td>NAP - 16.0</td>
<td>NAP -3.0</td>
</tr>
<tr>
<td>Eemian clay</td>
<td>Eemclay</td>
<td>NAP - 25.0</td>
<td>NAP -3.0</td>
</tr>
<tr>
<td>Intermediate sand layer</td>
<td>Tussenzandlaag</td>
<td>NAP - 38.0</td>
<td>NAP -3.0</td>
</tr>
<tr>
<td>Glacial Drenthe clay</td>
<td>GlacialDrentheklei</td>
<td>NAP - 42.0</td>
<td>NAP -3.0</td>
</tr>
<tr>
<td>Glacial Warven clay</td>
<td>GlacialWarvenklei</td>
<td>NAP - 45.0</td>
<td>NAP -3.0</td>
</tr>
<tr>
<td>Third sand layer</td>
<td>Derdezandlaag</td>
<td>NAP - 50.0</td>
<td>NAP -3.0</td>
</tr>
</tbody>
</table>

### 6.3 Pile Resistance and Material properties

This chapter explains about the determination of the input parameter for the pile resistance in embedded pile feature. In embedded pile model, the maximum tip and shaft resistance value is not obtained from calculation, but rather, it is inputted by user. Then, the development of load – displacement graph will depend on the shear modulus of the adjacent soil.

The calculation of the maximum tip resistance and shaft resistance will follow the CPT-based method as indicated in Eurocode 7 with Dutch guideline (NEN9997-1+C1). Since we are interested in calculating the real serviceability, material and partial factors in the calculation are set to 1.0.

Because the piles used are timber piles, the actual dimension of the pile tends to be tapered. From a test in Dapperbuurt (Korff [2012]), it was understood that the diameter of the pile head ranges from 220mm to 250mm and the tip has approximately 100-110mm of diameter in average. The shaft then is tapered with a rate of 9.1-10.5mm/m. In this analysis, the geometry of the pile is simplified as having constant diameter along the pile length. It is decided to use 180mm of pile diameter.

The calculation of shear force at the pile shaft adopts the aforementioned diameter value. However, in order to calculate the maximum pile tip resistance, it is decided to use the original pile tip diameter (100mm). The head level of the piles is set at NAP +0.4m and the tip level is set at NAP -12m, which is -0.5m below the top of the first sand layer. This dimension is summarized in Table 6.2.
Table 6.2 Pile dimension

<table>
<thead>
<tr>
<th>Level</th>
<th>06150404</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall diameter (embedded pile input)</td>
<td>180mm</td>
</tr>
<tr>
<td>Shaft diameter (calculation purpose)</td>
<td>180mm</td>
</tr>
<tr>
<td>Tip diameter (calculation purpose)</td>
<td>100mm</td>
</tr>
<tr>
<td>Head level</td>
<td>NAP +0.4m</td>
</tr>
<tr>
<td>Tip level</td>
<td>NAP -12m</td>
</tr>
</tbody>
</table>

There is no clear information about the stiffness value of timber pile at the site. The back analysis performed by Korff suggested that old timber pile might have elastic stiffness of 8.10^6 kPa. While, the elastic stiffness value for new timber pile is 20.10^6 kPa. In the subsequent simulation, 15.10^6 kPa is taken as the value for the elastic stiffness value.

The details of the calculated maximum resistance will be explained in the following paragraph. Three different CPTs; 14910S, 14911S, and 14912S; were compared and used as the basis of the calculation parameters. These CPTs are included in APPENDIX A.

![Figure 6.4 Typical CPT result - 14911S](image-url)
6.3.1 Maximum Pile Tip Resistance Calculation

The calculation of pile is summarised as follows:

- Pile class factor \( \alpha_p = 1 \)
- Factor of pile base shape \( \beta = 1 \)
- Factor that takes into account the pile base cross section \( s = 1 \)

In CPT-based design, averaging method is commonly accepted to include the cone resistance from the adjacent level to the tip of the pile. In Dutch method, the averaging of cone resistance follows the illustration in Figure 6.5. The main difference between the conditions shown in the picture is the existence of softer layer beneath pile tip level. In this analysis, the second condition (b) is considered due to the existence of Allerød layer which has lower cone resistance value than first sand layer.

![Figure 6.5 Dutch averaging method (Xu, 2007)](image)

Therefore the calculation of the influence depth is:

- Below pile tip level: \( 4D_{pile} = 4 \cdot 0.18m = 0.72m \)
- Above pile tip level: \( 8D_{pile} = 8 \cdot 0.18m = 1.44m \)

With this level, the obtained average cone resistance is:

- \( q_{c1} = \frac{25 + 11}{2} MPa = 18 MPa \)
- \( q_{c2} = 18 MPa \)
- \( q_{c3} = \frac{11 MPa \cdot 0.5m + 2 MPa \cdot 0.5m + 1 MPa \cdot 0.44m}{1.44m} MPa = 4.89 MPa \)

The area of the tip is:

- \( A = \frac{\pi}{4} D_{tip}^2 = \frac{\pi}{4} \cdot 0.1^2 = 0.00785 m^2 \)
Based on the aforementioned parameters, the maximum tip resistance is:

\[
F_{\text{max},\text{tip}} = \frac{1}{2} \alpha_p \beta_s \left( \frac{q_{c1} + q_{c2}}{2} + q_{c3} \right) A = 78.81 \text{kN}
\]

### 6.3.2 Maximum Shaft Resistance Calculation

The following formula from NEN9997-1+C1 is used to calculate the maximum shaft friction:

\[
T_{\text{max}} = \alpha_s q_c \pi D_p \rho
\]

There is a friction coefficient factor \( \alpha_s \) which is used to correct the value of cone resistance in a specific material. The value of \( \alpha_s \) can be obtained from NEN9997-1+C1. The summary of the calculation results are presented in the table and figure below. This value is used for both positive and negative skin friction.

**Table 6.3 Maximum shaft friction**

<table>
<thead>
<tr>
<th>Layer name</th>
<th>Dutch name</th>
<th>( \alpha_s )</th>
<th>( q_c ) [MPa]</th>
<th>( T_{\text{max}} ) [kN/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandfill Ophooglaag</td>
<td>0.01</td>
<td>2</td>
<td>11.30</td>
<td></td>
</tr>
<tr>
<td>Peat Hollandveen</td>
<td>0.00</td>
<td>0.5</td>
<td>0.00</td>
<td></td>
</tr>
<tr>
<td>Old sea clay Oudezeeklei</td>
<td>0.02</td>
<td>1</td>
<td>11.30</td>
<td></td>
</tr>
<tr>
<td>Wad deposit Wadafzetting</td>
<td>0.01</td>
<td>1</td>
<td>5.70</td>
<td></td>
</tr>
<tr>
<td>Hydrobia clay Hydrobiaklei</td>
<td>0.02</td>
<td>1</td>
<td>11.30</td>
<td></td>
</tr>
<tr>
<td>Peat Basisveen</td>
<td>0.00</td>
<td>0.5</td>
<td>0.00</td>
<td></td>
</tr>
<tr>
<td>First sand layer top Eerste zandlaag</td>
<td>0.01</td>
<td>1&amp;18</td>
<td>56.55</td>
<td></td>
</tr>
</tbody>
</table>

**Figure 6.6 Maximum shear force profile at the soil-pile interface**
Based on the analysis of pile test at Dapperbuurt, it was deducted in the case of a new-driven-timber pile, the calculated shaft capacity based on NEN9997 will produce only 70% of the achieved shaft capacity. On the other hand, the shaft capacity of old timber pile is 70% of the actual shaft capacity of new timber pile. Therefore, both discrepancies are cancelled out and the aforementioned calculation is valid.

6.4 Undrained vs Drained Analysis

This chapter discusses about the adopted drainage type in the simulation. The use of drainage type will influence the results. Hence, it is necessary to assess the required drainage condition.

The application of undrained or drained analysis completely depends on the desired observed behaviour. The main difference between both types of analysis lies on the excess pore pressure accounted in the analysis. In undrained analysis, the load is sustained by the total stress of the soil, which includes the pore pressure and soil skeleton. With the assumption that water is incompressible, volume changes in this condition is relatively small. This is also related to the fact that excess pore pressure is not dissipated yet. In contrast, in drained analysis, no pore pressure exists, and therefore any loading applied to the soil will be sustained by the soil skeleton.

According to Schweiger [2010], undrained analysis tends to be used in the analysis of:
- Material with low permeability,
- High loading rate and,
- Short term behaviour is of interest.

On the other hand, drained analysis is applied in the analysis of:
- Material with high permeability,
- Low loading rate and,
- Long term behaviour is the main attention

To check the appropriate condition in case of deep excavations, Vermeer and Meier [1998] suggested the following equations.

\[
T = \frac{k E_{oed}}{\gamma_w D^2} t
\]  

(6.1)

Where,
- \(T\) is dimensionless time factor.
- \(k\) is the permeability of the material.
- \(E_{oed}\) is the stiffness for consolidation calculation obtained from oedometer test.
- \(\gamma_w\) is the unit weight of the water.
- \(D\) is the length of the drainage path.
- \(t\) is the consolidation time.
If the value of $T$ is less than 0.01, undrained analysis tends to be used. On the other hand, if the value of $T$ is larger than 0.4, drained analysis is the main interest.

Considering the entire project periods (approximately 7 years), drained analysis is likely to be performed. A preliminary check in Oudezeeklei and Eem clay by using the aforementioned formula is conducted. Both are considered the most governing material since the permeability values of these layers are the lowest among other layers. The parameters are taken from datasets which is presented in APPENDIX B.

- **Oudezeeklei**
  $$T = \frac{k_E \cdot \gamma_{w}D^2}{10kN/m^3 \cdot 1.4m} \cdot \frac{0.8695 \cdot 10^{-3} m/day \cdot 1000kPa}{7 \text{years} \cdot 365 \text{days}} = 159$$

- **Eem clay**
  $$T = \frac{k_E \cdot \gamma_{w}D^2}{10kN/m^3 \cdot 6.5m} \cdot \frac{0.1296 \cdot 10^{-3} m/day \cdot 5500kPa}{7 \text{years} \cdot 365 \text{days}} = 48$$

Oudezeeklei has only 1 boundary to sandy layer (Wadzandlaag), therefore for preliminary check, the drainage path of Oudezeeklei is taken as the total thickness of the layer, which is 1.4m. On the other hand, the thickness of Eem clay is taken as half of the thickness, since it is confined by sandy material (Second Sand layer and Tussenzandlaag). Definitely, these values are used only for rough estimation. More rigorous analysis is required to determine the length of drainage path. With an assumption that the permeability of the soil remains constant during the project construction, it can be found that undrained behaviour can be ignored.

However, as observed in chapter 4.3.3, there is a delay in soil settlement after installation of diaphragm wall panels. This might be induced by the undrained behaviour of the soil. Using the suggested formula by Vermeer and Meier, it can be simply checked as follows:

- **Oudezeeklei**
  $$T = \frac{k_E \cdot \gamma_{w}D^2}{10kN/m^3 \cdot 1.4m} \cdot \frac{0.8695 \cdot 10^{-3} m/day \cdot 1000kPa}{4 \text{days}} = 0.24$$

- **Eem clay**
  $$T = \frac{k_E \cdot \gamma_{w}D^2}{10kN/m^3 \cdot 6.5m} \cdot \frac{0.1296 \cdot 10^{-3} m/day \cdot 5500kPa}{4 \text{days}} = 0.008$$

Based on these results, it is evident that undrained behaviour governs the behaviour of Eem clay during the period of diaphragm wall installation, assuming that 4 days are required to complete the installation of 1 panel. Consequently, it is evident that draining analysis is not appropriate to be conducted. Rather, undrained analysis should be employed.

In the subsequent analysis, drainage type of Undrained (A) is chosen. This option will allow us to use effective stress analysis and at the same time, PLAXIS will calculate the development of excess pore pressure due to a specific activity. Consequently, short term and long term behaviour of the project can be captured completely. However, only fine grained material
will be set to undrained (A). On the other hand, the drainage type of coarse grained material will be still set to drained, since coarse grained material tends to have a relatively small or even no excess pore pressure.

6.5 Preliminary Back Analysis

This chapter explains the simulation and the results. There are 5 major stages will be modelled:

- First stage – modelling the settlement of 100 years of time dependent settlement.
- Second stage – modelling the sand fill placement period.
- Third stage – modelling diaphragm wall installation period.
- Forth stage – modelling pumping test period.
- Fifth stage – modelling the main excavation.

It should be noted that in APPENDIX B, the parameter sets provide a range of parameter values for each layer. However, to back analyse the actual soil displacement, some trial and errors in the parameter determination should be conducted. In this back analysis, mean values of the parameters are used.

The results of the soil settlement as well as the piles settlement will be presented and compared to the measurement. The discrepancies in the results of the simulation and measurement will be the basis of the subsequent analysis with improved parameters.

6.5.1 Initial Soil Parameters

There are 2 different models used in the simulation, Soft Soil Creep and Hardening Soil model. During the simulation of 100 years time dependent settlement, Soft Soil Creep is used to model the fine grained soils, while the coarse grained soils are kept using Hardening Soil parameters. Entering the main construction stages, all of the soil layers will use Hardening Soil model.

The parameter sets for simulation of 100 years time dependent settlement is presented in Table 6.4. The initial hardening soil parameter adopted for the preliminary back analysis is presented in Table 6.5. It should be noted that the value of the POP or OCR is kept 0, since the preconsolidation pressure will be provided in the end of 100 years simulation.

The parameter values are adopted from the mean values presented in APPENDIX B. These parameter values will be evaluated and discussed in the subsequent passages. Some changes will be applied based on the back analysis results.
TABLE 6.4  INITIAL PARAMETER SETS FOR SIMULATION OF 100 YEARS SETTLEMENT (SOFT SOIL CREEP AND HARDENING SOIL MODEL)

<table>
<thead>
<tr>
<th>Material</th>
<th>( \gamma_{sat} ) [kN/m³]</th>
<th>( \gamma_{sat} ) [kN/m³]</th>
<th>( C' )</th>
<th>( C'' )</th>
<th>( \sigma_0 )</th>
<th>( C_0 )</th>
<th>( C_1 )</th>
<th>( k_{sat} )</th>
<th>( k_{sat} )</th>
<th>POP or OCR</th>
<th>( R_{interface} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anwoolting</td>
<td>15</td>
<td>15</td>
<td>0.1</td>
<td>0.25</td>
<td>Hardening Soil Model</td>
<td>8.64x10⁴</td>
<td>POP = 0</td>
<td>0.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hollandseid</td>
<td>10.5</td>
<td>10.5</td>
<td>5</td>
<td>0.29</td>
<td>2.64x10⁴</td>
<td>0.34</td>
<td>0.021</td>
<td>8.64x10⁴</td>
<td>POP = 0.7</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Oudendrecht</td>
<td>11</td>
<td>16.5</td>
<td>7</td>
<td>0.33</td>
<td>6.68x10⁴</td>
<td>0.038</td>
<td>0.018</td>
<td>1.26x10⁴</td>
<td>POP = 0.7</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Waddinxing</td>
<td>13.3</td>
<td>17.9</td>
<td>2</td>
<td>0.35</td>
<td>Hardening Soil Model</td>
<td>8.64x10⁴</td>
<td>POP = 0</td>
<td>0.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hydroblokken</td>
<td>13</td>
<td>15.2</td>
<td>8</td>
<td>0.34</td>
<td>0.840</td>
<td>0.1</td>
<td>0.0061</td>
<td>8.64x10⁴</td>
<td>POP = 0.7</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Restroom</td>
<td>11.7</td>
<td>11.7</td>
<td>6</td>
<td>0.21</td>
<td>2.18x10⁴</td>
<td>0.2</td>
<td>0.0123</td>
<td>8.64x10⁴</td>
<td>POP = 0.7</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Fine Sand Layer</td>
<td>15.8</td>
<td>19.8</td>
<td>0.1</td>
<td>0.35</td>
<td>Hardening Soil Model</td>
<td>8.64x10⁴</td>
<td>POP = 0</td>
<td>0.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Artificial Sand</td>
<td>14.6</td>
<td>18.5</td>
<td>0.1</td>
<td>0.35</td>
<td>Hardening Soil Model</td>
<td>8.64x10⁴</td>
<td>POP = 0</td>
<td>0.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Second Sand Layer</td>
<td>15</td>
<td>19.1</td>
<td>0.1</td>
<td>0.35</td>
<td>Hardening Soil Model</td>
<td>8.64x10⁴</td>
<td>POP = 0</td>
<td>0.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kim Clay</td>
<td>13.1</td>
<td>17.9</td>
<td>20</td>
<td>0.32</td>
<td>4.35x10⁴</td>
<td>0.313</td>
<td>0.0994</td>
<td>1.25x10⁴</td>
<td>OCR = 2.0</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Tusenronding</td>
<td>16</td>
<td>19.1</td>
<td>0.1</td>
<td>0.35</td>
<td>Hardening Soil Model</td>
<td>8.64x10⁴</td>
<td>POP = 0</td>
<td>0.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Glacial Deformed Clay</td>
<td>15.0</td>
<td>19.7</td>
<td>15</td>
<td>0.34</td>
<td>4.14x10⁴</td>
<td>0.11</td>
<td>0.0144</td>
<td>8.64x10⁴</td>
<td>OCR = 1.5</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Glacial Waried Clay</td>
<td>15</td>
<td>18.5</td>
<td>5</td>
<td>0.32</td>
<td>4.14x10⁴</td>
<td>0.11</td>
<td>0.0144</td>
<td>8.64x10⁴</td>
<td>OCR = 1.5</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Third Sand Layer</td>
<td>17</td>
<td>19.6</td>
<td>0.1</td>
<td>0.35</td>
<td>Hardening Soil Model</td>
<td>8.64x10⁴</td>
<td>POP = 0</td>
<td>0.7</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

6.5.2 Diaphragm Wall, Struts, and Jet Grout Column Parameters

Concrete Parameter for Diaphragm Wall, Roof, and Floor of The Station Box

The properties of the concrete for diaphragm wall, roof and the floor of the stations are summarised in a table below. The determination of the parameter is based on chapter 3.5.3.

**Table 6.5 Initial parameter sets for main construction stages (Hardening Soil Model)**

<table>
<thead>
<tr>
<th>Material</th>
<th>( \gamma_{sat} ) [kN/m³]</th>
<th>( \gamma_{sat} ) [kN/m³]</th>
<th>( C' )</th>
<th>( C'' )</th>
<th>( \sigma_0 )</th>
<th>( C_0 )</th>
<th>( C_1 )</th>
<th>( k_{sat} )</th>
<th>( k_{sat} )</th>
<th>( \mu )</th>
<th>( R_{interface} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anwoolting</td>
<td>15</td>
<td>15</td>
<td>0.1</td>
<td>0.25</td>
<td>Hardening Soil Model</td>
<td>8.64x10⁴</td>
<td>POP = 0</td>
<td>0.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hollandseid</td>
<td>10.5</td>
<td>10.5</td>
<td>5</td>
<td>0.29</td>
<td>2.64x10⁴</td>
<td>0.34</td>
<td>0.021</td>
<td>8.64x10⁴</td>
<td>POP = 0.7</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Oudendrecht</td>
<td>11</td>
<td>16.5</td>
<td>7</td>
<td>0.33</td>
<td>6.68x10⁴</td>
<td>0.038</td>
<td>0.018</td>
<td>1.26x10⁴</td>
<td>POP = 0.7</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Waddinxing</td>
<td>13.3</td>
<td>17.9</td>
<td>2</td>
<td>0.35</td>
<td>Hardening Soil Model</td>
<td>8.64x10⁴</td>
<td>POP = 0</td>
<td>0.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hydroblokken</td>
<td>13</td>
<td>15.2</td>
<td>8</td>
<td>0.34</td>
<td>0.840</td>
<td>0.1</td>
<td>0.0061</td>
<td>8.64x10⁴</td>
<td>POP = 0.7</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Restroom</td>
<td>11.7</td>
<td>11.7</td>
<td>6</td>
<td>0.21</td>
<td>2.18x10⁴</td>
<td>0.2</td>
<td>0.0123</td>
<td>8.64x10⁴</td>
<td>POP = 0.7</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Fine Sand Layer</td>
<td>15.8</td>
<td>19.8</td>
<td>0.1</td>
<td>0.35</td>
<td>Hardening Soil Model</td>
<td>8.64x10⁴</td>
<td>POP = 0</td>
<td>0.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Artificial Sand</td>
<td>14.6</td>
<td>18.5</td>
<td>0.1</td>
<td>0.35</td>
<td>Hardening Soil Model</td>
<td>8.64x10⁴</td>
<td>POP = 0</td>
<td>0.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Second Sand Layer</td>
<td>15</td>
<td>19.1</td>
<td>0.1</td>
<td>0.35</td>
<td>Hardening Soil Model</td>
<td>8.64x10⁴</td>
<td>POP = 0</td>
<td>0.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kim Clay</td>
<td>13.1</td>
<td>17.9</td>
<td>20</td>
<td>0.32</td>
<td>4.35x10⁴</td>
<td>0.313</td>
<td>0.0994</td>
<td>1.25x10⁴</td>
<td>OCR = 2.0</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Tusenronding</td>
<td>16</td>
<td>19.1</td>
<td>0.1</td>
<td>0.35</td>
<td>Hardening Soil Model</td>
<td>8.64x10⁴</td>
<td>POP = 0</td>
<td>0.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Glacial Deformed Clay</td>
<td>15.0</td>
<td>19.7</td>
<td>15</td>
<td>0.34</td>
<td>4.14x10⁴</td>
<td>0.11</td>
<td>0.0144</td>
<td>8.64x10⁴</td>
<td>OCR = 1.5</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Glacial Waried Clay</td>
<td>15</td>
<td>18.5</td>
<td>5</td>
<td>0.32</td>
<td>4.14x10⁴</td>
<td>0.11</td>
<td>0.0144</td>
<td>8.64x10⁴</td>
<td>OCR = 1.5</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Third Sand Layer</td>
<td>17</td>
<td>19.6</td>
<td>0.1</td>
<td>0.35</td>
<td>Hardening Soil Model</td>
<td>8.64x10⁴</td>
<td>POP = 0</td>
<td>0.7</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table 6.6 Diaphragm wall properties in the analysis**

<table>
<thead>
<tr>
<th>Properties</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material model</td>
<td>Linear elastic</td>
</tr>
<tr>
<td>Drainage type</td>
<td>Non-porous</td>
</tr>
<tr>
<td>Unit weight [( \gamma )]</td>
<td>23 kN/m³</td>
</tr>
<tr>
<td>Elastic stiffness [E]</td>
<td>20.10³ kPa</td>
</tr>
<tr>
<td>Possion ratio [( v )]</td>
<td>0.15</td>
</tr>
<tr>
<td>( R_{interface} )</td>
<td>0.7</td>
</tr>
</tbody>
</table>
Struts Properties

Table 6.7 indicates the properties of the struts implemented in the analysis. Chapter 3.5.4 is the basis of the input.

<table>
<thead>
<tr>
<th>Properties</th>
<th>1st Strut</th>
<th>2nd Strut</th>
<th>3rd Strut</th>
<th>4th Strut</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material type</td>
<td>Elastoplastic</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EA [kN/m]</td>
<td>9255338</td>
<td>12178459</td>
<td>13807836</td>
<td>13807836</td>
</tr>
<tr>
<td>$F_{\text{max tension}} = F_{\text{max compression}}$ [kN]</td>
<td>10580</td>
<td>13920</td>
<td>15780</td>
<td>15780</td>
</tr>
<tr>
<td>Pre-stress force [kN]</td>
<td>3000</td>
<td>5500</td>
<td>8250</td>
<td>4000</td>
</tr>
</tbody>
</table>

Jet Grout Column Properties

Based on chapter 3.5.5, the following properties of the jet grout column are adopted.

<table>
<thead>
<tr>
<th>Properties</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Material model</td>
<td>Mohr-Coulumb</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Drainage type</td>
<td>Undrained (A)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\gamma_{\text{sat}} = \gamma_{\text{unsat}}$ [kN/m$^3$]</td>
<td>17</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Elastic stiffness [E]</td>
<td>$2.10^6$ kPa</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Possion ratio [v]</td>
<td>0.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$c'$</td>
<td>3.6 kPa</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\phi'$</td>
<td>26.5°</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$R_{\text{interface}}$</td>
<td>0.7</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

6.5.3 100 Years Time Dependent Settlement and The Period of Sand Fill Placement

The finite element analysis is commenced by simulating 100 years time dependent settlement. The simulation of 100 years time dependent settlement is implemented to:

- Gain a correct in situ stress of the soil prior to the main analysis of the main construction phases. This issue is substantial since the response of the soil is greatly influenced by the initial in situ stress or preconsolidation pressure. In addition, pile response has a close relationship to the soil displacement. Thus, having a correct initial in situ stress will enable us to gain accurate results.
- Create the developed pile load distribution, including drag load and pile resistance. Similar to the necessity of in situ stress in the soil, the initial condition of the pile is also substantial to the response of pile to deep excavation activities.

For the purpose of parameter verification, the results from the simulation of 100 years time dependent settlement and sand fill placement are observed simultaneously. By checking the gained preconsolidation pressure after 100 years and the response of the soil during sand fill placement, correct parameter values can be obtained.
It is realised that there are little observations or monitoring data on the settlement to compare with the calculation results. However, the observation results from several sources as mentioned in chapter 4.2 can be regarded as a reasonable comparable data for the soil settlement calculation results.

The observations on pile distribution since 100 years ago do not also exist. Yet, by comparing soil settlement and pile settlement to the observation, a conclusion on pile load distribution could be drawn.

Based on Table 4.1:

- The annual surface settlement rate is 2.2-3.5mm/year. If it is assumed that the settlement developed linearly, after 100 years the total settlement is 220 – 350mm.
- The First Sand layer’s settlement rate is 0.4mm/year. Consequently, it leads to 40mm of total settlement after 100 years.
- On the other hand, annual rate for building settlement is 1.25-2.0mm/year. The total building settlement 125 – 200mm.

The comparison of the simulation results after sand fill placement will be based on the measurement data.

6.5.3.1 Model and schedule of 100 years of Time Dependent Settlement and Addition Sand Fill Placement Period

To model 100 years of time dependent settlement, some facts about the actual field condition are considered.

Based on some general information, it is understood that for every several years local government raises the surface layer by adding more sand fill to maintain the level of the road. The thickness of the additional sand fill is unknown, but generally for maintenance purpose, 1m is the maximum thickness.

On the other hand, from the investigation at several buildings, the head of the pile foundation is situated approximately at NAP -0.5m to NAP -1.0m. It is definitely plausible if an assumption was taken that the surface level 100 years back was close to pile head level.

Eventually, in the subsequent analyses, it is assumed that the 100 years back level was at NAP-0.5m, and the surface was maintained by adding 30cm of additional fill for every 30years. This decision has considered that the current surface level is at NAP 0.4m.

The aforementioned considerations become the basis of the staged constructions in the finite element analysis, which are explained in the table below.
**Table 6.8 Staged constructions in the initial analysis**

<table>
<thead>
<tr>
<th>Stage Number</th>
<th>Activity</th>
<th>Type</th>
<th>Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>K₀ procedure – in situ stress generation</td>
<td>Ko-procedure</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>Pile installation</td>
<td>Plastic</td>
<td>1 day</td>
</tr>
<tr>
<td>3</td>
<td>Application of external load</td>
<td>Plastic</td>
<td>1 day</td>
</tr>
<tr>
<td>4</td>
<td>Additional 30cm of fill</td>
<td>Plastic</td>
<td>1 day</td>
</tr>
<tr>
<td>5</td>
<td>Consolidation for 30 years</td>
<td>Consolidation</td>
<td>10950 days</td>
</tr>
<tr>
<td>6</td>
<td>Additional 30cm of fill</td>
<td>Plastic</td>
<td>1 day</td>
</tr>
<tr>
<td>7</td>
<td>Consolidation for 30 years</td>
<td>Consolidation</td>
<td>10950 days</td>
</tr>
<tr>
<td>8</td>
<td>Additional 30 cm of fill</td>
<td>Plastic</td>
<td>1 day</td>
</tr>
<tr>
<td>9</td>
<td>Consolidation for 40 years</td>
<td>Consolidation</td>
<td>14657 days</td>
</tr>
<tr>
<td>10</td>
<td>Additional sandfill placement</td>
<td>Plastic</td>
<td>1 day</td>
</tr>
<tr>
<td>11</td>
<td>Consolidation for 285 days</td>
<td>Consolidation</td>
<td>285 days</td>
</tr>
<tr>
<td>12</td>
<td>Change from SSC to HS model</td>
<td>Plastic⁽¹⁾</td>
<td>-</td>
</tr>
<tr>
<td>13</td>
<td>Addition sand fill placement</td>
<td>Plastic⁽²⁾</td>
<td>-</td>
</tr>
<tr>
<td>14</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>285 days</td>
</tr>
</tbody>
</table>

⁽¹⁾ ignored undrained behaviour
⁽²⁾ reset displacement to zero

Figure 6.7 indicates the staged construction when the fill is built up. Figure 6.8 shows the placement of the sand fill.
Several different analyses and iterations were performed to investigate the best fit parameter sets. From these analyses, it was found that the most significant layers are Hollandveen and Oudezeeklei. It is believed that these layers are the main triggering factor of consolidation in the soil. Other consolidation-triggering layers, such as Hydrobiaklei or Basisveen, can be considered insignificant due to its thin thickness. Deeper layers are believed to be more relatively stable compared to Holocene layers.

To achieve similar settlement to the observation, some changes in the parameters were performed. For further convenience in the comparison, the original parameter sets will be called as analysis set – 1 and the adjusted parameters are called analysis set – 2. The parameters of Hollandveen and Oudezeeklei in both sets are presented in Table 6.9.
### Table 6.9 Differences in Hollandveen and Oudezeeklei in Analysis Set – 1 and Analysis Set – 2

<table>
<thead>
<tr>
<th></th>
<th>Analysis set - 1</th>
<th>Analysis set - 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hollandveen</td>
<td>Oudezeeklei</td>
</tr>
<tr>
<td>Cohesion [kPa]</td>
<td>5</td>
<td>7</td>
</tr>
<tr>
<td>Friction angle [°]</td>
<td>20</td>
<td>33</td>
</tr>
<tr>
<td>$\gamma_{\text{unsat}}$ [kN/m$^3$]</td>
<td>2.5</td>
<td>11</td>
</tr>
<tr>
<td>$\gamma_{\text{sat}}$ [kN/m$^3$]</td>
<td>10.5</td>
<td>16.5</td>
</tr>
<tr>
<td>$e_0$</td>
<td>7.78</td>
<td>1.35</td>
</tr>
<tr>
<td>POP [kPa]</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>$C_c$</td>
<td>2.644</td>
<td>0.606</td>
</tr>
<tr>
<td>$C_r$</td>
<td>0.24</td>
<td>0.086</td>
</tr>
<tr>
<td>$C_\alpha$</td>
<td><strong>0.023</strong></td>
<td>0.0049</td>
</tr>
<tr>
<td>$k_x$, $k_y$, $k_z$ [m/day]</td>
<td>8.64x10$^{-4}$</td>
<td>1.296x10$^{-4}$</td>
</tr>
</tbody>
</table>

It should be noted that other soil layers are kept using the original parameter sets as explained in Table 6.4.

The most significant changes took place in the creep parameter and preconsolidation pressure of Hollandveen. The original values tend to underestimate the total value of time dependent settlement after 100 years. It is evident that the original $C_\alpha$ from Hollandveen and Oudezeeklei are close. This is contrast with the common fact that peat layer behaves far softer than soft grain material. Hence, most efforts were concentrated on the creep parameter.

The adjustment of the consolidation parameter followed the rule of thumb regarding the ratio between modified compression index ($\lambda^*$) and modified creep index ($\mu^*$), which is commonly applied in practice. In which, the ratio between those ($\mu^*/\lambda^*$) normally lies on 0.1-0.04. With 0.2 as the value of $C_\alpha$ value of 0.2, the ratio of $\mu^*/\lambda^*$ is 0.13.

Using analysis set – 2, better results can be obtained. The comparison of the results with the observation will be explained in the following passages.

#### 6.5.3.2 Soil Response After 100 years Settlement and Additional Sand Fill Placement Period

The calculation results from 2 analysis sets give a considerable difference. Analysis set – 1 tends to underestimate the soil settlement after 100 years. The comparison between those 2 analyses sets are presented in Figure 6.9. Analysis set – 1 gives less than 20mm of surface settlement, on the other hand, 428mm of surface settlement was calculated using analysis set – 2.
The difference between the analysis sets is mainly caused by the application of different value of $C_\alpha$. Based on the result of analysis set – 2, it is evident that most settlement occurs in Hollandveen layer, while other layers contribute less.

Comparing the simulation results to the measurement data, analysis set – 2 gives more reasonable results. Figure 6.10 gives another comparison during the period of sand fill placement. The figure suggested us that analysis set – 2 again gives closer result to the measurement. Analysis set – 1 clearly behaves softer and overestimates the settlement profile.

The main reason behind this difference lies in the basic concept of creep model. Bjerrum [1967] originally suggested that the accumulated strain due to creep will affect the preconsolidation pressure. Based on the equation 2.68, the longer the soil is exposed to creep strain, the higher the preconsolidation pressure will be achieved in the end of the calculation.

In this sense, the preconsolidation pressure in Hollandveen in analysis set – 2 should achieve higher value after 100 years of settlement, as clearly shown in Figure 6.11. Given this fact, therefore it is logical that the soils in analysis set – 2 behave slightly ‘stiffer’ than analysis set – 1.

Eventually, parameters from analysis set – 2 are considered as the reasonable parameters set. But, this conclusion is limited only the verification of Soft Soil Creep model parameters. More evaluation will be given later for Hardening Soil model parameters in the construction phases.
FIGURE 6.10 CALCULATED SURFACE SETTLEMENT AFTER SANDFILL PLACEMENT PERIOD FROM DIFFERENT ANALYSES USING SSC AND HS MODEL

FIGURE 6.11 PRECONSOLIDATION PRESSURE ACHIEVED AFTER 100 YEARS PERIOD
6.5.3.3 Pile Settlement and Behaviour after 100 years Settlement

To discuss the pile response, pile 70-3 and pile 100-3 are presented as the representatives of both pile groups. Pile load distribution, mobilized shear friction, and the settlement plot are presented in Figure 6.12 to Figure 6.18.

The results suggest that both piles behave differently. Total pile settlements after 100 years settlement are 44mm and 62mm, from pile 70-3 and pile 100-3 respectively. Assuming that pile settlement is equal to building settlement, these values are less than the general observation result as discussed in chapter 4.2. The observation suggested that the settlement of the buildings at Ceintuurbaan should be between 12.5cm to 20 cm after 100 years. But, it should be noted that the values are obtained based on the observation to buildings with different foundation condition. In fact, prior to the construction of the project, foundation investigation found that many buildings were founded on poor foundation condition. In contrast, the piles in the model are in perfect condition, disregarding damages.

Based on Figure 6.13 and Figure 6.17, the position of the neutral planes of the piles after 100 years can be determined. The neutral plane level in pile 70-3 lies at NAP -7m within Wadafzetting layer, on the other hand in pile 100-3, the neutral plane is located slightly higher in Oudezeeklei at NAP -6m.

A slight difference in the mobilised shear force in the piles exists. In pile 100-3, maximum shear force has been fully mobilised along the pile. Pile 70-3 has also mobilised the maximum shear force, except in Wadafzetting layer. The reason behind this partial mobilization of shear force is that the required relative displacement between pile and soil to achieve maximum shear force has not been reached. In Figure 6.14 and Figure 6.15, it can be observed that the difference between pile and soil settlement in Wadazetting layer is less than 2mm. From several researches, it has been understood that full friction will be mobilised by relative displacement between 2-20mm.

As a consequence, the pile load distribution in pile 70-3 forms a slight curve at the depth where maximum drag load is reached. Definitely, pile 100-3 that has significant difference in relative displacement of pile and soil has sharper form. The zone where the shaft force is partially mobilised is called transition zone.
**Figure 6.12 Load Distribution in Pile 70-3 After 100 Years**

**Figure 6.13 Mobilised Shear Force in Pile 70-3 After 100 Years**
**Figure 6.14** Settlement of pile 70-3 and the adjacent soil after 100 years

**Figure 6.15** Relative displacement of pile 70-3 and the adjacent soil inducing transition zone (zoomed in)
Figure 6.16 Load distribution in pile 100-3 after 100 years

Figure 6.17 Mobilized shear force in pile 100-3 after 100 years
6.5.3.4 Pile and Soil Response to Sand Fill Placement

Figure 6.19 presents the results of soil and pile settlement after sand fill placement. Small discrepancies can be observed in the surface and First Sand layer settlement from the simulation and the measurement.

However, the difference is not significant. On the other hand, the simulation overestimates piles’ settlement by 2-4mm. It can be possibly influenced by the settlement of First Sand layer. Another possible explanation is the behaviour of the interface of the embedded pile at the tip. It is understood that the behaviour of the interface at this location is ‘soft’, which can lead to the overestimated settlement. The effect of installation is also not taken into account. Densification should occur at the tip of driven pile and hence, the stiffness of the soil should be relatively high. In this simulation, the stiffness of the interface is merely governed by the adjacent soil.
Some adjustments in the parameters will be conducted to obtain closer results to the measurement. Therefore, pile behaviour after sand fill placement will be presented later in chapter 6.6.1 after the adjustment.

6.5.4 Diaphragm Wall Installation

In this chapter, the modelling of diaphragm wall installation phases and the simulation results are presented. To give clearer explanation of the model, some additional information on the bentonite pressure and fresh concrete pressure are provided based on the previous researches. In the end, the simulation results will be compared to the measured data.

6.5.4.1 Model and Schedule

In practice, the installation of diaphragm wall involves 2 main steps, trenching and concreting. These steps will be implemented in the finite element analysis to model the case as accurate as possible.

In total there will be 9 panels’ installation that will be modelled. This number is based on the prior analysis in chapter 4.3.3. Based on the observation of the extensometers, it is understood that approximately 5 panels influence the soil at cross section 13110E. Since the piles groups are located with a specific distance to cross section 13110E, the number of panels’ installation in the model should be increased to accommodate the installation influence to the piles. It was decided to increase to 9 panels. The plan view of the diaphragm wall can be observed in Figure 6.3. The adjacent panels to panel 25 and 17 will be modelled as wished-in-place concrete volume.

The steps involved in the model for the installation of a panel are:
• Deactivating the soil inside the panel. The deactivated volume will be set as dry material, so no pore pressure will exist.

• To model the trenching, surface load is applied to the surface of diaphragm wall. Hydrostatic bentonite pressure is used, as shown in Figure 6.20. It will be explained in more detail in the subsequent passages.

• During concreting, surface load is again implemented, but with different profile. It has been understood that during concreting, bilinear surface profile is more appropriate to model the fresh concrete pressure. It will be explained in more detail in the subsequent passages.

• After the trenching and concreting processes have been accomplished, the excavated volume will be filled by concrete as a non-porous volume material, blanketed by interfaces at the boundary of the wall and the soil. The surface load will be deactivated.

![Figure 6.20 Applied surface load during trenching (light blue line)](image)

This process is repeated based on the schedule of each panel’s installation. Table 6.10 lists the installation schedule for 9 panels.

<table>
<thead>
<tr>
<th>Panel</th>
<th>Start Excavation</th>
<th>Start concrete pouring</th>
<th>Finish concreting</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>05-08-2004</td>
<td>10-08-2004</td>
<td>10-08-2004</td>
</tr>
<tr>
<td>24</td>
<td>16-08-2004</td>
<td>19-08-2004</td>
<td>19-08-2004</td>
</tr>
<tr>
<td>22</td>
<td>20-08-2004</td>
<td>24-08-2004</td>
<td>24-08-2004</td>
</tr>
<tr>
<td>23</td>
<td>30-08-2004</td>
<td>01-09-2004</td>
<td>01-09-2004</td>
</tr>
<tr>
<td>19</td>
<td>06-09-2004</td>
<td>07-09-2004</td>
<td>07-09-2004</td>
</tr>
<tr>
<td>21</td>
<td>10-09-2004</td>
<td>13-09-2004</td>
<td>13-09-2004</td>
</tr>
<tr>
<td>20</td>
<td>14-09-2004</td>
<td>16-09-2004</td>
<td>16-09-2004</td>
</tr>
<tr>
<td>17</td>
<td>16-09-2004</td>
<td>17-09-2004</td>
<td>17-09-2004</td>
</tr>
<tr>
<td>18</td>
<td>21-09-2004</td>
<td>24-09-2004</td>
<td>24-09-2004</td>
</tr>
</tbody>
</table>

In Table 6.10, we can find that panels’ installation consumes several days. In Noord-Zuid Lijn, the excavation usually took 1 day and another 1 day was for concreting. The rest of the
period of the installation of 1 panel was dedicated to some other activities related to the
installation, for instance preparation of concreting, pulling out tremie pipe, or insertion of
steel cage. Following the schedule, Table 6.11 shows the sequence of finite element analysis
conducted arranged for this specific construction activity.

**Table 6.11 Finite element analysis sequence**

<table>
<thead>
<tr>
<th>Stage Number</th>
<th>Activity</th>
<th>Type</th>
<th>Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>Activate concrete volume adjacent to panel 25</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>16</td>
<td>Trenching of panel 25 – bentonite pressure</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>17</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>5 days</td>
</tr>
<tr>
<td>18</td>
<td>Concreting of panel 25 – concrete pressure</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>19</td>
<td>Applying concrete volume in the panel</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>20</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>6 days</td>
</tr>
<tr>
<td>21</td>
<td>Trenching of panel 24 – bentonite pressure</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>22</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>3 days</td>
</tr>
<tr>
<td>23</td>
<td>Concreting of panel 24 – concrete pressure</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>24</td>
<td>Applying concrete volume in the panel</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>25</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>1 day</td>
</tr>
<tr>
<td>26</td>
<td>Trenching of panel 22 – bentonite pressure</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>27</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>4 days</td>
</tr>
<tr>
<td>28</td>
<td>Concreting of panel 22 – concrete pressure</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>29</td>
<td>Applying concrete volume in the panel</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>30</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>2 days</td>
</tr>
<tr>
<td>31</td>
<td>Trenching of panel 23 – bentonite pressure</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>32</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>6 days</td>
</tr>
<tr>
<td>33</td>
<td>Concreting of panel 23 – concrete pressure</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>34</td>
<td>Applying concrete volume in the panel</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>35</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>1 day</td>
</tr>
<tr>
<td>36</td>
<td>Trenching of panel 19 – bentonite pressure</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>37</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>1 day</td>
</tr>
<tr>
<td>38</td>
<td>Concreting of panel 19 – concrete pressure</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>39</td>
<td>Applying concrete volume in the panel</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>40</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>3 days</td>
</tr>
<tr>
<td>41</td>
<td>Trenching of panel 21 – bentonite pressure</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>42</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>3 days</td>
</tr>
<tr>
<td>43</td>
<td>Concreting of panel 21 – concrete pressure</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>44</td>
<td>Applying concrete volume in the panel</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>45</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>1 day</td>
</tr>
<tr>
<td>46</td>
<td>Trenching of panel 20 – bentonite pressure</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>47</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>2 days</td>
</tr>
<tr>
<td>48</td>
<td>Concreting of panel 20 – concrete pressure</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>49</td>
<td>Applying concrete volume in the panel</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>50</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>1 day</td>
</tr>
<tr>
<td>51</td>
<td>Trenching of panel 17 – bentonite pressure</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>52</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>1 day</td>
</tr>
<tr>
<td>53</td>
<td>Concreting of panel 17 – concrete pressure</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>54</td>
<td>Applying concrete volume in the panel</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>55</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>4 days</td>
</tr>
</tbody>
</table>
Following the installation of diaphragm wall, wished-in-place jet grout column is activated. The modelling of jet grout column installation is not completely performed here because, based on the analysis and observation discussed in chapter 4.3.4, no significant displacement of the soil has occurred.

The position of jet grout column is presented in the figure below.

![Figure 6.21 Jet Grout Column Position](image)

**6.5.4.2 Bentonite Pressure**

The bentonite pressure is governed by the unit weight of the bentonite used in the installation. However, there is no available information related to the unit weight of the bentonite used in the construction. Hence, some trial-and-errors were conducted during the trenching of panel 25. The typical range of bentonite unit weight is known approximately between 10.5 and 12 kN/m³.

Two analyses have been conducted using the lowest and the highest bentonite unit weight value for the simulation of panel 25’s excavation (first installed panel). Comparing to the measured soil settlement in the extensometers, the value of 12 kN/m³ produces closer surface settlement result to the measurement which can be observed in Figure 6.22 below. The main reason of this difference is the stress relief occurs in the deeper layer. Using bentonite unit weight of 10 kN/m³ will induce larger difference between the in-situ stress and the applied pressure in the Eem clay, and respectively, larger soil displacement occurs.
However, using 12 kN/m\(^3\) does not produce exactly similar results to the measured settlement. Applying larger value of bentonite unit weight certainly will lead to unrealistic value. Nevertheless, it can be argued that the accuracy of the simulation is reasonably well, considering that the extensometers might not be able to capture such a small settlement as indicated in the calculation results.

Based on this finding, the rest of the simulation for diaphragm wall installation will be modelled using 12 kN/m\(^3\) of unit weight.

![Surface settlement profile during trenching at panel 25](image)

**Figure 6.22 Surface settlement profile during trenching at panel 25**

### 6.5.4.3 Fresh Concrete Pressure Envelope

Clayton and Milititsky [1983] reported that during concreting, the wet concrete pressure can be approximated as in hydrostatic condition up to a specific shallow depth. Respectively in deeper layer, the pressure of wet concrete remains constant. The depth at which the maximum pressure acts is denoted as \(h_{\text{crit}}\). Therefore, the maximum pressure can be explained by \(P_{\text{max}} = \gamma_c h_{\text{crit}}\) and the distribution of the wet concrete pressure is illustrated below. This observation was made in the installation of bored pile.

Harrison [1983] explained that such pressure distribution is influenced by the hydration of the concrete. In freshly placed concrete, the initial pressure will be mainly produced by the pore pressure of the concrete. Because the aggregates are still suspended, the effective stress of the aggregates is basically zero. But, in deeper layer where the concrete has been placed first, hydration proceeds earlier, subsequently, the maximum hydrostatic pressure cannot be achieved as the pore pressure dissipates. In this condition, effective stress starts to develop, but it only takes practically a little portion of the total horizontal pressure. Figure 6.24 illustrates this concept and Harrison denoted \(\lambda\) to describe the portion of the effective stress.
Based on several observations, CIRIA report 108 [1985] proposed the following formulas below to be used as a design guideline. Similarly as suggested by the aforementioned researchers, bilinear profile is used. Equation 6.2 is used at the depth shallower than $h_{\text{crit}}$, while equation 6.3 is used for deeper depth. The vertical profile is presented in Figure 6.25.

$$\sigma_h = \gamma_{\text{concrete}} \cdot z$$  \hspace{1cm} (6.2)

$$\sigma_h = (\gamma_{\text{concrete}} - \gamma_{\text{bentonite}}) \cdot h_{\text{crit}} + \gamma_{\text{bentonite}} \cdot z$$  \hspace{1cm} (6.3)

However, the above pressure envelopes are developed based on the empirical data. Theoretically, the determination of pressure envelope during concreting depends on several aspects as indicated in some guidelines and researches, such as:

- Rate of pouring – the faster the rate, the closer the pressure to hydrostatic concrete pressure.
- Elapsed time – the pressure will be decreased gradually when hydration occurs and pore pressure is dissipated.
Therefore, the pressure envelope in the real condition is time dependent while, the suggested pressure envelope actually covers only the maximum pressure regardless the time when the pressure acts to the soil.

With such pressure envelope, it has been proven from several back analysis, for instance de Wit and Lengkeek [2002] and Schad et al [2007], that bilinear pressure envelope is feasible to model panels installation.

But then, it is required to determine the value of $h_{crit}$ since it is an indispensable factor. It has been understood from several studies that the determination of $h_{crit}$ is not trivial. Several observations and theoretical studies will be discussed to give a better understanding.

Before Noord-Zuid Lijn commenced, Wit and Lengkeek [2002] conducted some preliminary assessment about this specific case. Numerical analysis and field measurement were performed to reduce the uncertainties in the prediction. Based on the monitoring of fresh concrete pressure, the value of the critical height was situated at $h/5$ (Figure 6.26).

Schad et al [2007] shows several measured fresh concrete pressure envelope in Figure 6.27. The measurement was conducted at Cambridge, Oslo, and Seville. It was concluded from these 3 different cases that the value of $h_{crit}$ lies between $h/3$ to $h/5$ from the top of the panels, where $h$ is the height of the diaphragm wall panels. Schad et al also has shown another observation representing Dutch experience at Blijdorp, Rotterdam. It was found that the value of $h_{crit}$ is at $h/5$ as shown in Figure 6.28.

Concluding from several local experiences, especially at Mondriaan Tower, it is decided to use $h/5$ as the value of $h_{crit}$. The applied bentonite and bilinear concrete pressure is presented in Figure 6.29.
Figure 6.26 Fresh concrete pressure at Mondriaan Tower, Amsterdam (De Wit and Lengkeek, 2002)

Figure 6.27 Fresh concrete pressure envelope at Cambridge (left), Oslo (middle), and Seville (right) (Schad, 2007)
6.5.4.4 Comparison of Soil Displacement Results

Figure 6.30 shows the comparison between measured and calculated soil settlement in horizontal cross section at 13110E, as a result of consecutive installation of 9 panels. Figure 6.31 and Figure 6.32 indicate the distribution of the soil settlement with depth at 06150401 and 06150402. It should be noted that the displacements presented below shows the displacement induced only by the installation, not the total displacement from the period prior to sand fill placement.
Figure 6.30 Settlement profile at 13110E due to diaphragm wall installation

Figure 6.31 Soil settlement in vertical cross section at 06150401 due to diaphragm wall installation
From the pictures, it can be deducted that the surface settlements agree reasonably well. But, it is not applicable for the deeper layer. It is found that the simulation produce larger settlement in deeper layer. As a direct consequence, the settlements of the piles are also overestimated, since it is significantly influenced by the foundation layer settlement.

Another observation is performed on the lateral displacements of the soil which are shown in Figure 6.33 and Figure 6.34. Both measurement and simulation show that the soils move inwards to the soil body, which is induced by the concrete pressure. Largest compression occurs in Hollandveen and Oudezeeklei layers.
From these figures, certainly it can be simply concluded that the simulation has overestimated the lateral displacement of the soil. But, it is contradictive to the finding in the soil settlements, which shows that the simulation overestimates the soil settlement.

However, it seems that there are some errors in the lateral displacement pattern in the inclinometers. Based on the inclinometer at 06150401, it is evident that in the deeper layer below NAP -35m, the inclinometer matches with the simulation. But, the lateral displacement immediately increases in the layer between NAP -25m and NAP -35m. Similar odd displacement occurs at 06150402, but in shallower position.

An attempt to fit the measurement was performed by changing the bilinear concrete pressure profile. Instead of using $h_{crit} = 20\%$ of the panel height, $h_{crit} = 33\%$ of panel height is used. So, it is expected that the pressure against the soil is increased. The comparison is presented in Figure 6.35.

Even with the increase in the concrete pressure, the lateral soil displacement could not catch up with the measurement values. A speculative decision is taken by disregard the inclinometer measurement for diaphragm wall installation phase, considering the following points:

- Further changes in the soil stiffness (softening) might be possible to be applied. But considering the value of the lateral displacement in the soil, i.e Second Sand layer, it will require an extremely low stiffness value for 25mm of lateral displacement.
- Another change is possible by applying larger horizontal stress during concreting. But, in the end, considering the unloading stiffness of the layers deeper than the First Sand layer, it will require a large stress to produce 20-30mm of lateral displacement.
Concluding the above discussion, the following issues are present:

- The settlement profile with depth will be the basis of the verification in diaphragm wall installation phase.
- Based on the settlement profile at 2 different extensometer locations, the surface settlements from the simulation match reasonably well with the measurement. However, the settlements of the deeper layers are overestimated. This leads to the softer response of the pile. An improvement of the soil parameters will be required. The improvement will focus on the decrease of the stiffness of Holocene layer and increase of the stiffness of the deeper layer. The changes of the stiffness will be based on the data range given in APPENDIX B.
- The reliability of the inclinometers results in the soil cannot be concluded due to the unrealistic values. Therefore, it will not be used as the verification data.

6.5.5 Pumping Test
6.5.5.1 Model and Schedule

After the diaphragm wall has been installed, several pumping tests were conducted. The pumping tests were only conducted in the soils inside excavated area. The schedule of the pumping tests is summarised in the following table.

<table>
<thead>
<tr>
<th>Pumping test</th>
<th>Starting date</th>
<th>Finishing date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pumping test at first and second sand layer to NAP -3.1m; and</td>
<td>07-03-2006</td>
<td>10-03-2006</td>
</tr>
</tbody>
</table>
According to the schedule, the following finite element simulation stages are applied.

**Table 6.13 Finite element analysis stages for pumping test period**

<table>
<thead>
<tr>
<th>Stage Number</th>
<th>Activity</th>
<th>Type</th>
<th>Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>64</td>
<td>Change groundwater head of:</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>- First and second sand layer; and Allerød to NAP -3.1m; and Tussenzandlaag to NAP -7m.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>65</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>3 days</td>
</tr>
<tr>
<td>66</td>
<td>Normalisation, change groundwater head of:</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>- First and second sand layer; and Allerød back to NAP -3m; and Tussenzandlaag to NAP -3m.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>67</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>27 days</td>
</tr>
<tr>
<td>68</td>
<td>Change groundwater head of:</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>- First and second sand layer; and Allerød back to NAP -13m; and Tussenzandlaag to NAP -34m.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>69</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>7 days</td>
</tr>
<tr>
<td>70</td>
<td>Normalisation, change groundwater head of:</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>- First and second sand layer; and Allerød back to NAP -3m; and Tussenzandlaag to NAP -3m.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>71</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>75 days</td>
</tr>
</tbody>
</table>

The simulation of the pumping tests was performed by lowering the groundwater head of a specific layer subjected to the test uniformly, as a conservative approach. The following figure shows different groundwater head between the soils inside and outside the excavated area.
6.5.5.2 Comparison of Soil and Pile Settlement Results

The comparison between pile and soil settlement due to this specific period is presented in Figure 6.37 and Figure 6.38. The measured building displacement in this case cannot be presented since the extensometers show unrealistic value during this period and also the manual measurements are absent. From the figure, it can be deducted that the simulation overestimates the settlement of the soil, particularly at the location close to the diaphragm wall. This could be induced by the conservative approach taken to model the pumping test.

However, the calculation results show different pattern compared to the measurement. The calculation results suggest that the first sand layer settle down almost with the same rate with the surface. This means that the settlement of the deeper layer is the governing factor.

It is evident that according to the simulation results, the most influential layer is Drenthe clay or Warvan clay (NAP -37m to NAP -45m). This effect is caused by the fact that the influence of pumping tests performed inside the excavated area attenuates more significantly in those layers, which lie close to the tip of the diaphragm wall.
6.5.6 Main Excavation
6.5.6.1 Model and Schedule

The main excavation phase contains several stages of excavation. The following analysis stages are formulated based on the construction sequence in Table 3.4.
### Table 6.14 Finite element analysis stages in main excavation phase

<table>
<thead>
<tr>
<th>Stage Number</th>
<th>Activity</th>
<th>Type</th>
<th>Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>71</td>
<td>Excavation to -2m</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>72</td>
<td>Concreting the station roof</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>73</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>355 days</td>
</tr>
<tr>
<td>74</td>
<td>Excavation to NAP -6.2m and installation of 1&quot; level of strut</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>75</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>115 days</td>
</tr>
<tr>
<td>76</td>
<td>Excavation to NAP -10.3m</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>77</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>30 days</td>
</tr>
<tr>
<td>78</td>
<td>Installation of 2nd level of strut</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>79</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>40 days</td>
</tr>
<tr>
<td>80</td>
<td>Excavation to NAP -15.3m</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>81</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>22 days</td>
</tr>
<tr>
<td>82</td>
<td>Installation of 3rd level of strut</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>83</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>32 days</td>
</tr>
<tr>
<td>84</td>
<td>Excavation to NAP -19.6m, 1&quot; floor installation and release of 3rd strut</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>85</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>195 days</td>
</tr>
<tr>
<td>86</td>
<td>Excavation to NAP -25.6m and installation of 4th level of strut</td>
<td>Plastic</td>
<td>-</td>
</tr>
<tr>
<td>87</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>284 days</td>
</tr>
<tr>
<td>88</td>
<td>Excavation to -31.1 followed by Installation of foundation layer (sand)</td>
<td>Plastic</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Installation of 2nd floor</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Air pressure (65kPa) is applied</td>
<td></td>
<td></td>
</tr>
<tr>
<td>89</td>
<td>Consolidation</td>
<td>Consolidation</td>
<td>134 days</td>
</tr>
</tbody>
</table>

Figure 6.39 shows the 3-dimensional model of the excavation.

![3D Model of Excavation](image)

**Figure 6.39 3D model of the excavation**
The concrete floors and roof are modelled with concrete volume governed by linear elastic behaviour and assumed having similar stiffness with the diaphragm wall, 20.106kPa. Node to node anchor with elastoplastic model is used to model 4 levels of struts. The properties of the supporting structures are summarised in chapter 6.5.2.

In Figure 6.39, it can be observed that there is surface pressure applied in the lower part of the station box. It is intended to model the air pressure as indicated in stage 89. The air pressure is assumed as a uniform surface load applied to all direction.

6.5.6.2 Comparison of Soil and Pile Displacement Results

Using the hardening soil parameters in Table 6.5, the following soils and piles settlements in Figure 6.40 are obtained. This settlement is induced only due to excavation activities to NAP-25.6m (stages 87) only.

![Figure 6.40](image)

**Figure 6.40 Comparison between calculated and measured settlement of the soils and piles**

It can be observed that the simulation underestimate the surface settlement and First Sand layer settlement. It has been understood that the settlement of the soil during deep excavation could be influenced by wall deflection.

A comparison on the lateral wall displacement is also presented below. The calculated wall deflection is confirmed by referring to the analysis result of monitoring data from Korff [2012]. Korff analyses the measured diaphragm wall deflection from 4 September 2007 to 6 June 2009. 4 September 2007 refers to the time when excavation to NAP -10m was about to commence, and 6 June 2009 refers to the time when excavation to NAP -25.6m had been accomplished. The calculated wall deflection is presented in Figure 6.66 and the analysis of the measurement data by Korff is presented in Figure 6.42.
FIGURE 6.41 LATERAL WALL DEFLECTION FROM 4 SEPTEMBER 2007 TO 6 JUNE 2009

FIGURE 6.42 MEASURED DIAPHRAGM WALL DEFLECTION AND GROUND DISPLACEMENT AT DIFFERENT LEVEL FROM 4 SEPTEMBER 2007 TO 6 JUNE 2009 (KORFF, 2012)
It is clear that the wall deflection is underestimated. This could be caused by the stiffness of the concrete or the prestress force applied in the model. Considering the applied value of concrete stiffness, it is unlikely that lower value should be applied. Therefore, the prestress force of the struts should be inspected.

### 6.5.7 Conclusion on Preliminary Back Analysis

Based on the results above, it can be observed:

- **After 100 years of time dependent settlement:**
  100 years of time dependent settlement has been modelled. The settlements of the soils and the piles fit reasonably well with the measurement. Pile load distribution as a result of 100 years time dependent settlement has been presented. Pile with external load of 70kN has neutral plane situated at NAP -7m, while the neutral plane of pile with external load of 100 kN is at NAP -6m. It can be concluded that pile loaded with higher load and fully mobilised shaft friction will have higher neutral plane position. Pile load distribution in other construction stages will be discussed later after the adjustment of analysis’ parameter.

- **After sand fill placement:**
  The simulation of sand fill placement produces reasonable match of soil and pile settlement. However, the calculated pile settlement is overestimated.

- **After diaphragm wall installation:**
  Based on the analysis results in the diaphragm wall installation, the surface settlements from the simulation match reasonably well with the measurement. However, the settlements of the deeper layers are overestimated. This leads to the softer response of the pile. The reliability of the inclinometers results in the soil cannot be concluded due to the unrealistic values. Therefore, it will not be used as the verification data.

- **After pumping test:**
  The simulation overestimates the settlement, either the surface or the First Sand layer. The surface settles in the same value with the First Sand layer, hence the piles follow this value. It is found that the governing layers are located deeper down, which are Drenthe and Warvan clay. According to the measurement, only the surface experiences settlement, while the deeper layers have no reaction.

- **After main excavation:**
  After main excavation phases, it was found that the surface and First Sand layer settlements are underestimated. This is suspected to be influenced by the lateral wall deflection, which is underestimated as well.

Improvement in the parameters is definitely required. Considering the results in the diaphragm wall installation, it can be concluded that Holocene layers should act softer, while the deeper layer should act stiffer.
Another improvement should be applied in the main excavation phases as well. Theoretically, the wall deflection will govern the settlement of the soil. Therefore, the properties of the retaining structures are of importance.

Currently, the adopted concrete stiffness of the wall is 20.106 kPa. Using lower stiffness value, i.e. 10.106 kPa (lower bound value), will possibly lead to better agreement in the wall deflection. But however, this low value is actually for crushed concrete which is unlikely in this case.

Therefore, the attention will be brought to the pre-stress forces. Later in the passage below, it will be explained that the actual (monitored) force exists in the struts is actually lower than the applied pre-stress force. Definitely, this aspect will significantly influence wall deflection.

6.6 Refined Analysis

To observe the pile behaviour correctly, it is of importance to obtain a correct soil displacement. Due to the discrepancies between the calculation and the measurement results, some additional analyses were performed. The focus is dedicated to diaphragm wall installation phases and the main excavation.

Remarks on Diaphragm Wall Installation Phases

Based on the comparison between measurement and calculation results in Figure 6.30, it can be observed that the first sand layer and deeper layers behave softer than the reality. At the same time, it implies that the stiffness values of Holocene layers (NAP 0.4m to NAP -12m) should be reduced.

The adjusted parameters are presented in the table below. These parameters are obtained from the parameter ranges in the datasets. The Holocene layers adopt lower bound values while the deeper layers adopt the ultimate upper bound value.

### Table 6.15 Adjusted Soil Parameters

<table>
<thead>
<tr>
<th>Material</th>
<th>( E_{soil} ) [kN/m²]</th>
<th>( E_{fr} ) [kN/m²]</th>
<th>( E' ) [°]</th>
<th>( \epsilon' )</th>
<th>( E_{soil} ) [kN/m²]</th>
<th>( E_{fr} ) [kN/m²]</th>
<th>( k_{cr} ) [m/s]</th>
<th>( k_{cr} ) [m/s]</th>
<th>( k_{cr} ) [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aanvulling</td>
<td>15</td>
<td>15</td>
<td>0.1</td>
<td>23</td>
<td>8000</td>
<td>6000</td>
<td>25000</td>
<td>8.64 x 10⁻⁵</td>
<td>0.8</td>
</tr>
<tr>
<td>Hollandersteen</td>
<td>14</td>
<td>14</td>
<td>5</td>
<td>20</td>
<td>3000</td>
<td>900</td>
<td>7000</td>
<td>8.64 x 10⁻⁵</td>
<td>0.8</td>
</tr>
<tr>
<td>Oudezoutdijk</td>
<td>11</td>
<td>16.5</td>
<td>7</td>
<td>33</td>
<td>7500</td>
<td>2000</td>
<td>2000</td>
<td>1.29 x 10⁻⁵</td>
<td>0.8</td>
</tr>
<tr>
<td>Vlaanderfaring</td>
<td>13.3</td>
<td>17.9</td>
<td>2</td>
<td>35</td>
<td>10000</td>
<td>5000</td>
<td>25000</td>
<td>8.64 x 10⁻⁵</td>
<td>0.5</td>
</tr>
<tr>
<td>Haploheukak</td>
<td>15.2</td>
<td>15.2</td>
<td>8</td>
<td>24</td>
<td>7500</td>
<td>2200</td>
<td>15000</td>
<td>8.64 x 10⁻⁵</td>
<td>0.8</td>
</tr>
<tr>
<td>Polderheukak</td>
<td>11.7</td>
<td>11.7</td>
<td>6</td>
<td>21</td>
<td>2000</td>
<td>1000</td>
<td>7000</td>
<td>8.64 x 10⁻⁵</td>
<td>0.8</td>
</tr>
<tr>
<td>Floreroel</td>
<td>16.8</td>
<td>15.8</td>
<td>0.1</td>
<td>33</td>
<td>5000</td>
<td>5000</td>
<td>30000</td>
<td>1236</td>
<td>0.5</td>
</tr>
<tr>
<td>Allerduin</td>
<td>14.4</td>
<td>18.5</td>
<td>0.1</td>
<td>33</td>
<td>25000</td>
<td>25000</td>
<td>25000</td>
<td>2.982</td>
<td>0.5</td>
</tr>
<tr>
<td>Grondvolkenduine</td>
<td>17</td>
<td>17</td>
<td>1</td>
<td>35</td>
<td>6750</td>
<td>5000</td>
<td>39400</td>
<td>0.8</td>
<td>0.5</td>
</tr>
<tr>
<td>Rein Clay</td>
<td>13.1</td>
<td>17.9</td>
<td>20</td>
<td>32</td>
<td>18200</td>
<td>6650</td>
<td>40500</td>
<td>1.73 x 10⁻⁴</td>
<td>0.8</td>
</tr>
<tr>
<td>Polderheukak</td>
<td>16</td>
<td>19.6</td>
<td>0.1</td>
<td>35</td>
<td>30000</td>
<td>30000</td>
<td>30000</td>
<td>8.64 x 10⁻⁵</td>
<td>0.8</td>
</tr>
<tr>
<td>Glaciale Drentse Clay</td>
<td>15.8</td>
<td>19.7</td>
<td>15</td>
<td>34</td>
<td>23400</td>
<td>10400</td>
<td>117000</td>
<td>8.64 x 10⁻⁵</td>
<td>0.8</td>
</tr>
<tr>
<td>Glaciale Warnau Clay</td>
<td>15</td>
<td>18.5</td>
<td>5</td>
<td>37</td>
<td>73040</td>
<td>8750</td>
<td>91000</td>
<td>8.64 x 10⁻⁵</td>
<td>0.8</td>
</tr>
<tr>
<td>Third Sand layer</td>
<td>17</td>
<td>19.6</td>
<td>0.1</td>
<td>35</td>
<td>58500</td>
<td>58500</td>
<td>59000</td>
<td>8.64</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Remarks on The Main Excavation Phases
Theoretically, the settlement of the soil behind the diaphragm wall is governed by the volume displaced due to the deflection of the wall. To improve the results of the soil settlement profile, then, the focus in main excavation will be addressed to the properties of retaining structures, such as concrete stiffness of the wall and the pre-stress force of the struts.

As discussed above, the wall in the simulation behaves stiffer than the reality. It has been explained that the current adopted stiffness value for the concrete is reasonably good. Lower value will lead to a condition for crushed concrete, which is unlikely to occur.

Furthermore, it is found that the pre-stress force in the struts is the main cause. From a report on the monitoring of the struts’ force [Project bureau Noord/Zuid Lijn, [2008a], it is observed that the actual existing force in the struts after pre-stressing process is not as high as the applied value.

This statement does not imply that the contractor applied smaller pre-stress force than the design value. Rather, it is possible that the pre-stressing process causes the release of the force.

These struts were installed using hydraulic jack placed between the struts and diaphragm wall. Once the pre-stressing is accomplished, hydraulic jack is dismantled and the gap is filled by a grout volume. It is possible that this process allows the struts to release some of the forces. Thus, the actual force transferred to the soil is less than the applied pre-stress force.

Four sensors were installed at the circumference of each strut. Some of the results of the monitoring are summarised in the table below. These struts are situated close to cross section 13110E.

**Table 6.16 Applied and Measured Pre-stress Force in the Struts during its Installation**

<table>
<thead>
<tr>
<th>Level</th>
<th>Struts number</th>
<th>Sensor number</th>
<th>Applied Pre-stress Force [kN]</th>
<th>Monitored Force after Pre-stressing [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>NAP -5.55m</td>
<td>8</td>
<td>1</td>
<td>3000</td>
<td>3122</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td></td>
<td>3068</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td></td>
<td>2636</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td></td>
<td>2448</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>1</td>
<td>3000</td>
<td>3376</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td></td>
<td>2952</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td></td>
<td>2643</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td></td>
<td>3242</td>
</tr>
<tr>
<td>NAP -9.55m</td>
<td>8</td>
<td>1</td>
<td>5500</td>
<td>4817</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td></td>
<td>4340</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td></td>
<td>3365</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td></td>
<td>3136</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>1</td>
<td>5500</td>
<td>1000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td></td>
<td>3370</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td></td>
<td>3814</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td></td>
<td>2598</td>
</tr>
</tbody>
</table>
Based on the table above, the first struts at NAP -5.55m (first level) do not experience significant difference between the monitored and applied force. Therefore, the transferred force to the wall can be assumed equal to the applied pre-stress force. On the other hand, the struts at NAP -9.55m (second level) suffer considerable reduction, especially in strut 14 with an average force of 2900kN.

There is no available information about the struts at 3rd and 4th. However, based on the experiences above, it is decided to adjust the pre-stress force applied in the simulation. Reasonable agreement in the lateral displacement of the wall can be achieved by using pre-stress force of 3000kN for the 3rd and 4th level. The result will be presented in more detail in the subsequent passages.

6.6.1 Soil and Pile Response due to Sand Fill Placement

The comparison of the measured and calculated settlement after parameters refinement can be observed in Figure 6.43.

![Figure 6.43 Soil and pile settlement due to sand fill placement after parameters refinement](image)

The results of the current simulation shows that First Sand layer behaves stiffer compared to the previous results. A slight difference between the settlement of Holocene layers' from simulation and measurement can be found, however the discrepancy is not significant.

The figure also suggests that the settlement of pile 70s and 100s overestimate the actual building settlement. However, it is evident that the measured building settlement is closer to pile 70s with a maximum difference of 3mm. It implies that the load of the actual piles is approximately close to 70kN, or it could be less.

The overestimate of the piles’ settlement could also be caused by the embedded pile model itself. The current model has not taken into account the effect of installation effect, and therefore, the current interface stiffness of the pile tip behaves softer than reality. Some
refinement could be made by applying a local soil volume with higher stiffness value at the tip.

The other explanation is the difference of the reality and the model. It is assumed in the model that the building is flexible enough to follow pile settlement, therefore single piles are applied. In reality, it is possible that there is an influence of building stiffness and moreover, the location of the measurement points that are located at the wall of the building. The response of the wall might be partially transferred to the pile and to the soil due to the existence of *kesp* (cross beam). Thus, such discrepancy occurs.

The location of the neutral planes after the period of sand fill placement can be observed in Figure 6.44 and Figure 6.45. It can be observed that there is different behaviour in both pile groups. Pile 70s indicate reduced neutral plane while the neutral plane in pile 100s is steady. This difference is mainly caused by the existence of the transition zone in pile 70s, which is able to lead the pile to mobilize more positive skin friction. While in pile 100s, maximum positive skin friction has been reached. The only space the shaft friction can vary is around the existing neutral plane. Therefore, the changes in the neutral plane are not much.

For furthest piles, in which the neutral plane does not change, pile settles with a value close to the foundation layer. But, such conclusion is not clear since the difference in settlement value between the surface and the foundation layer is not much.

Figure 6.46 and Figure 6.47 represent the changes of pile 70-3 and pile 100-3 as the representative of the most affected piles. Figure 6.48 and Figure 6.49 show the additional settlement of both piles and the adjacent soil that triggers the changes in the load distribution of the pile.

Based on Figure 6.43, Figure 6.44, and Figure 6.45, it can be concluded that the settlement of the pile is determined by the position of the neutral plane of the pile, especially observing pile 70-3 and pile 100-3. By having higher location of neutral plane, pile will be subjected to larger settlement value in the condition when the surface settlement is larger than the foundation layer settlement.
FIGURE 6.44 Changes of neutral plane in pile 70s due to sand fill placement

FIGURE 6.45 Changes of neutral plane in pile 100s due to sand fill placement
Figure 6.46 Load distribution (above) and mobilized shear force (below) in pile 70-3
Figure 6.47 Load distribution (above) and mobilized skin shear force in pile 100-3 (below)
Table 6.17 and Table 6.18 provide another comparison about the influence of neutral plane position to the pile settlement. Table 6.19 tabulates the comparison of the measured building settlement to the measured $\Delta$Settlement. It should be noted that the term $\Delta$Settlement refers to the difference between the settlement of the surface and First Sand layer settlement, where surface settlement is larger than First Sand layer settlement.
TABLE 6.17 PERCENTAGE OF CALCULATED SETTLEMENT OF PILE 70s TO THE CALCULATED SUBSURFACE SETTLEMENT – DUE TO SAND FILL PLACEMENT

<table>
<thead>
<tr>
<th>Location</th>
<th>Surface Settlement [mm]</th>
<th>First Sand Layer Settlement [mm]</th>
<th>Pile 70s Settlement [mm]</th>
<th>Percentage of Pile Settlement to ΔSettlement</th>
<th>Neutral Plane</th>
</tr>
</thead>
<tbody>
<tr>
<td>06150401</td>
<td>11</td>
<td>0.9</td>
<td>4.7</td>
<td>37%</td>
<td>65%</td>
</tr>
<tr>
<td>06150402</td>
<td>3.6</td>
<td>0.8</td>
<td>2.5</td>
<td>60%</td>
<td>55%</td>
</tr>
<tr>
<td>06150403</td>
<td>1.7</td>
<td>0.5</td>
<td>1.3</td>
<td>67%</td>
<td>50%</td>
</tr>
<tr>
<td>06150404</td>
<td>0*</td>
<td>0.4</td>
<td>0.2</td>
<td>48%</td>
<td>51%</td>
</tr>
</tbody>
</table>

*heaving

TABLE 6.18 PERCENTAGE OF CALCULATED SETTLEMENT OF PILE 100s TO THE CALCULATED SUBSURFACE SETTLEMENT – DUE TO SAND FILL PLACEMENT

<table>
<thead>
<tr>
<th>Location</th>
<th>Surface Settlement [mm]</th>
<th>First Sand Layer Settlement [mm]</th>
<th>Pile 100s Settlement [mm]</th>
<th>Percentage of Pile Settlement to ΔSettlement</th>
<th>Neutral Plane</th>
</tr>
</thead>
<tbody>
<tr>
<td>06150401</td>
<td>11</td>
<td>0.9</td>
<td>7.0</td>
<td>60%</td>
<td>40%</td>
</tr>
<tr>
<td>06150402</td>
<td>3.6</td>
<td>0.8</td>
<td>3.2</td>
<td>85%</td>
<td>42%</td>
</tr>
<tr>
<td>06150403</td>
<td>1.7</td>
<td>0.5</td>
<td>1.5</td>
<td>83%</td>
<td>41%</td>
</tr>
<tr>
<td>06150404</td>
<td>0*</td>
<td>0.4</td>
<td>0.3</td>
<td>28%</td>
<td>45%</td>
</tr>
</tbody>
</table>

*heaving

Attention is drawn to 06150401 since it provides the clearest picture about the correlation of the position of the neutral plane and the percentage of the pile settlement to the differential settlement of the subsurface. In pile 70-3, it is evident that the neutral plane is located at 65% of pile length. On the other hand, the pile settlement is 35% from the total differential settlement of the subsurface. Thus, it is evident that neutral plane and additional pile settlement is closely related.

By comparing the percentage of pile settlement to ΔSettlement in Table 6.17 and Table 6.19, it is also clear that the pile load of the buildings is close to 70kN.

Such assessment is only valid for pile with significant changes in neutral plane. Pile with no changes in neutral plane tends to follow the settlement of the foundation layer plus the settlement at the neutral plane, as discussed in chapter 5.3. Changes of neutral plane of the piles at 06150402 and 06150403 have been observed.

However, the comparison of the neutral position and the percentage of pile settlement to ΔSettlement in further location, i.e 06150404 is hindered by the small discrepancy of the settlement values. Therefore, the value might be different.
### Table 6.19 Percentage of Measured Settlement of the Building and the Soils - Due to Sand Fill Placement

<table>
<thead>
<tr>
<th>Location</th>
<th>Surface Settlement [mm]</th>
<th>First Sand Layer Settlement [mm]</th>
<th>Corresponding Buildings Settlement [mm]</th>
<th>Percentage of Pile Settlement to $\Delta$Settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>06150401</td>
<td>10</td>
<td>0*</td>
<td>2</td>
<td>22%</td>
</tr>
<tr>
<td>06150402</td>
<td>2</td>
<td>0.2</td>
<td>0.5</td>
<td>28%</td>
</tr>
<tr>
<td>06150403</td>
<td>1</td>
<td>0.4</td>
<td>0.2</td>
<td>34%</td>
</tr>
<tr>
<td>06150404</td>
<td>1</td>
<td>0*</td>
<td>0.7</td>
<td>64%</td>
</tr>
</tbody>
</table>

*heaving

Ultimately, it can be concluded that the settlement of the piles due to the sand fill placement is influenced by the settlement of foundation layer and the position of the neutral plane. The higher the position of the neutral plane, the larger the value of the settlement is.

Moreover, the load of the pile is also influential to the settlement. Higher load will induce higher neutral plane and therefore, it will lead to larger settlement value as shown by pile 70-3 and pile 100-3.

#### 6.6.2 Soil and Pile Response due to Diaphragm Wall Installation

Better fit can be obtained in the simulation of diaphragm wall installation compared to the previous analysis. The comparison is presented below. A vertical profile for the soil settlement is also presented to give a clearer picture about the settlement distribution.

The settlement pattern of building, surface, and First Sand layer from 06150402 to 06150404 produced from the simulation agrees with the measurement. However, at 06150401, the simulation shows a sagging shape. This could be triggered by the bilinear pressure of the fresh concrete. It has been understood that the bilinear pressure is the approximation of the maximum fresh concrete pressure during concreting. However, in reality, such pressure does not exist in the same time. Rather, it is a time dependent.

Another remark is addressed to the difference of the real building settlement and the simulated pile settlement. The maximum difference is 5mm at 06150402. Similar to the previous explanation, it is possible that the interface stiffness of the embedded pile at the tip is too low due to the absence of pile installation effect. Refinement could be made by applying stiffer soil locally at the tip.

The other influencing factor is the difference of the model and the reality. The model simplifies the real building with a series of piles, assuming that the building is flexible enough to follow pile head settlement. While in reality, the measurement points are located at the wall of the buildings. The load of the building might be redistributed not only to the pile, but also to the soil, considering the existence of the $k_{ep}$.
During the installation phases, a series of unloading-reloading occurs in the soil. Figure 6.52 is the plot of the changes in total horizontal stress experienced by the soils. It can be observed that the soils at NAP 0.4m to NAP -25m are mainly subjected to additional compressive loading while soils from NAP -25m to NAP -45m experience substantial unloading condition.
Figure 6.53 and Figure 6.54 show the changes in total horizontal stress at different level, NAP -12m (First Sand layer) and NAP -30m (Eem clay layer) during installation of panel 25.

**Figure 6.52 Total horizontal stress changes during panel installation**

**Figure 6.53 Total horizontal stress changes in first sand layer (NAP -12m) at 0m distance from diaphragm wall edge during installation of panel 25**
A very fine meshing is not applied in the model, nevertheless, arching effect can still be observed. At NAP -12m, the horizontal stress increases at the corner of the panel while a reduction occurs at mid-position during trenching. Due to the concreting, the total horizontal stress increases. At NAP -30m, which is in Eem clay layer, arching effect also exists. But the total horizontal stress does not recover to the original stress level (after sand fill).

The unloading-reloading series, together with pile-soil relative displacement, definitely influence the load distribution of the pile. Figure 6.55 and Figure 6.56 are presented to illustrate the changes in normal stress and also the pile-soil relative displacement due to trenching and concreting and the example is taken based on installation at panel 25 and pile 70-3 condition.

As a result of these changes, the load distribution of pile 70-3 during the installation of panel 25 can be observed in Figure 6.57. It is evident that during trenching, pile load distribution is slightly released as an effect of reduction in normal stress and the pile-soil relative displacement. Subsequently, caused by the increase of normal stress and the pile-soil relative displacement during concreting, the pile load distribution is increased.
FIGURE 6.55 PLOT OF CHANGES IN NORMAL STRESS AND PILE-SOIL RELATIVE DISPLACEMENT DUE TO TRENCHING AT PANEL 25
Figure 6.56 Plot of changes in normal stress and pile-soil relative displacement due to concreting at panel 25.
Figure 6.57 Evolution of load distribution in pile 70-3 after trenching and concreting at panel 25

Figure 6.58 and Figure 6.59 present the final load distribution of pile 70-3 and pile 100-3 after 9 panels have been installed. In the final condition, it appears that the final load in the piles is reduced compared to the condition after sand fill placement.
FIGURE 6.58 LOAD DISTRIBUTION (ABOVE) AND MOBILIZED SHEAR FORCE IN PILE 70-3 AFTER DIAPHRAGM WALL INSTALLATION
The final position of the neutral planes of the piles are demonstrated in Figure 6.60 and Figure 6.61. The neutral planes are shifted up in the end of diaphragm wall installation, compared to the previous neutral plane after sand fill placement.
To conclude this chapter, the percentages of the soil and building settlement due to diaphragm wall installation are summarised below. It should be noted that the term \( \Delta \text{Settlement} \) below refers to the difference between surface and First Sand layer settlement, where large settlement is located at the surface.

Similar to the observation in the sand fill placement, the neutral plane of the pile is closely related to the percentage of the pile settlement to the differential subsurface settlement, especially referring to pile 70-3. For pile 100-3, the neutral plane is located at the Hollandveen in which the maximum shaft friction is 0, thus it is difficult to judge and decide the correct position of the neutral plane.

Based on the observation on the percentage of Pile Settlement to \( \Delta \text{Settlement} \), it is likely that the load of the real pile is close to 70kN.

**Table 6.20 Percentage of Calculated Settlement of Pile 70s to the Calculated Subsurface Settlement – Due to Diaphragm Wall Installation**

<table>
<thead>
<tr>
<th>Location</th>
<th>Surface Settlement [mm]</th>
<th>First Sand Layer Settlement [mm]</th>
<th>Pile 70s Settlement [mm]</th>
<th>Percentage of Pile Settlement to ( \Delta \text{Settlement} )</th>
<th>Neutral Plane</th>
</tr>
</thead>
<tbody>
<tr>
<td>06150401</td>
<td>11</td>
<td>3.2</td>
<td>7.1</td>
<td>50%</td>
<td>51%</td>
</tr>
<tr>
<td>06150402</td>
<td>13.1</td>
<td>4</td>
<td>8.8</td>
<td>53%</td>
<td>60%</td>
</tr>
<tr>
<td>06150403</td>
<td>10.9</td>
<td>4</td>
<td>8.2</td>
<td>61%</td>
<td>50%</td>
</tr>
<tr>
<td>06150404</td>
<td>3.8</td>
<td>2.3</td>
<td>3.4</td>
<td>73%</td>
<td>51%</td>
</tr>
</tbody>
</table>
### Table 6.21 Percentage of calculated settlement of pile 100s to the calculated subsurface settlement – due to diaphragm wall installation

<table>
<thead>
<tr>
<th>Location</th>
<th>Surface Settlement [mm]</th>
<th>First Sand Layer Settlement [mm]</th>
<th>Pile 100s Settlement [mm]</th>
<th>Percentage of Pile Settlement to ΔSettlement</th>
<th>Neutral Plane</th>
</tr>
</thead>
<tbody>
<tr>
<td>06150401</td>
<td>11</td>
<td>3.2</td>
<td>7.6</td>
<td>56%</td>
<td>32%</td>
</tr>
<tr>
<td>06150402</td>
<td>13.1</td>
<td>4</td>
<td>10.5</td>
<td>71%</td>
<td>26%</td>
</tr>
<tr>
<td>06150403</td>
<td>10.9</td>
<td>4</td>
<td>9</td>
<td>72%</td>
<td>40%</td>
</tr>
<tr>
<td>06150404</td>
<td>3.8</td>
<td>2.3</td>
<td>3.5</td>
<td>80%</td>
<td>46%</td>
</tr>
</tbody>
</table>

### Table 6.22 Percentage of measured settlement of the building and the – due to diaphragm wall installation

<table>
<thead>
<tr>
<th>Location</th>
<th>Surface Settlement [mm]</th>
<th>First Sand Layer Settlement [mm]</th>
<th>Corresponding Buildings Settlement [mm]</th>
<th>Percentage of Pile Settlement to ΔSettlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>06150401</td>
<td>10.7</td>
<td>2.2</td>
<td>6</td>
<td>45%</td>
</tr>
<tr>
<td>06150402</td>
<td>11.1</td>
<td>3</td>
<td>4.5</td>
<td>19%</td>
</tr>
<tr>
<td>06150403</td>
<td>8</td>
<td>2.2</td>
<td>4.5</td>
<td>40%</td>
</tr>
</tbody>
</table>

### 6.6.3 Soil and Pile Response due to Pumping Test

The settlement induced by pumping test period has similar pattern with the previous back-analysis result, that the settlement is governed by the deeper layer. However, since the stiffness of the deeper layer has been increased, the settlement is also reduced. The comparison between the measured and calculated settlement is shown in the figure below.

Figure 6.63 shows the observed excess pore pressure induced by the pumping tests. Excess pore pressure, either positive or negative, is developed at NAP -25m to greater depth. The location of the developed excess pore pressure surely corresponds to the settlement pattern, in which deeper layer governs the settlement as shown by Figure 6.64.
FIGURE 6.62  SOIL AND PILE SETTLEMENT DUE TO PUMPING TEST AFTER PARAMETER REFINEMENT

FIGURE 6.63  EXCESS PORE PRESSURE DURING SECOND PUMPING TEST AT 06150401 (CALCULATION PHASE 68)
The maximum difference in settlement between the measurement and the simulation is 4mm. This discrepancy is possibly explained by 2 reasons. Firstly, it might be the case that the measurement points in the extensometer do not capture small settlement in deeper layers very well.

Theoretically, the deeper layer should settle since the source of the groundwater difference is located in the deep location. In this case, surface settles more than the deeper layer. Secondly, the conservative approach taken in this simulation might produce overestimated settlement.

Since there are no differential settlement between the surface and the foundation layer, the piles do not experience any changes in the load distribution. Due to such condition, the piles settle following the foundation layer. The neutral planes also remain constant.

### 6.6.4 Soil and Pile Response due to Main Excavation

The result based on the adjusted soil parameters and pre-stress force, is presented below. It should be noted that the comparison of the soil and pile settlement is only for the condition of excavation until NAP -25.6m.

It can be observed that the simulation still underestimates the surface and First Sand layer settlement. However, focusing on the First Sand layer settlement, the value is better than the previous result. Another check is performed to the lateral displacement of the wall. Comparing Figure 6.34 and Figure 6.66, wall deflection profile from current analysis is closer to the measurement, although there is a discrepancy of 4mm to the reality.

Beside the parameter changes, the properties of the retaining structures such as pre-stress force of the struts, are also adjusted as explained in the introduction of chapter 6.6. Based on these findings, therefore it can be concluded that the actual properties of the retaining
structures are significantly influential. Further improvement could be made by adjusting 2 variables, the struts’ pre-stress forces and concrete stiffness.

**Figure 6.65 Settlement profile due to main excavation using refined parameters**

**Figure 6.66 Calculated diaphragm wall deflection after parameters refinement, from 4 September 2007 to 6 June 2009**

Considering that the First Sand layer settlement and wall deflection from the simulation are correct, the discrepancy in the settlement of Holocene layers is still questionable. There is a possibility that another effect influences the performance of Holocene layers.

Several reasonable aspects influencing the settlement are:

- Creep of the Holocene layers
Attention is drawn to the difference of the settlement at 06150404 (28m from diaphragm wall). The difference between simulation and measurement results at this particular location is 15mm, which could be an indication of volume changes in Holocene layers.

It is understood that during the main excavation there are no primary loadings applied to the Holocene layers, therefore, no volume changes should have occurred. Rather, the deformation of the soil during the excavation should have been governed by the wall deflection.

From the site characterisation, Amsterdam subsurface is still experiencing time dependent settlement. With an annual settlement rate of 2.2-3.5mm/year (Table 4.1), the difference at 06150404 should be compensated. It should be noted that roughly the main excavation took 3 years. Therefore, the total additional settlement in this period is approximately 10mm.

Considering this fact, another analysis is performed by using a similar approach during the initial condition, using parameters in Table 6.4. The result of the analysis is presented below.

**Figure 6.67 Settlement profile due to main excavation using SSC model for Holocene layers**

It is obvious that incorporating creep in the simulation can produce reasonable fit at 06150404. The settlement at 06150404 has approach the measured value. However, significant discrepancies still occur for soils closer to the diaphragm wall.

- Activities induced loading at the surface
  Assuming creep plays important role in the settlement of Holocene layers, it can be observed that the simulation still underestimates the surface settlement. Another possible explanation for the surface settlement is activities at the surface inducing additional load. It could be the case that the activities are not recorded and therefore it cannot be taken into account in the simulation.
The excavation has influenced the load distribution of the piles. From the previous construction phases earlier, it can be deducted that the behaviour of the actual building is close to pile 70s, therefore, as an example, the evolution of the load distribution of pile 70-3 is presented here. The presented results are taken from the simulation using Hardening Soil model (Table 6.5).

**Figure 6.68 Evolution of pile 70-3's load distribution during main excavation phases**

Based on the graph, it can be observed that as the excavation progresses especially until NAP-10m, the pile experiences a reduction in the load distribution, especially triggered by the reduction of shaft friction. While the tip resistance is stable. This event is caused by the unloading condition in the soil, thus the normal stress is reduced.

But once the excavation progresses to deeper position than the depth of foundation layer, a significant tip resistance can be observed. Thus the pile is required to mobilise a new pile resistance, resulting in a longer part of the pile experiencing positive shaft friction. In the end, the neutral plane is shifted up close to the surface. A small part at the pile tip experiences negative skin friction as a consequence of the settlement of the First Sand layer.

Table 6.23 presents the comparison between additional settlement of the surface, First Sand layer, and the adjacent building at 06150401. It appears that the most influential phase to the building settlement is during excavation to NAP-15m. During this period, a significant changes occur in the load distribution, when the pile mobilise a new resistance to compensate the lost of tip resistance.

It can be observed that subsequently, after the excavation to NAP-15m, basically no additional settlement occurs. This is caused by the uplift pressure at the station box. The station box is pushed up and it drags the adjacent soil up. Therefore, additional settlement of the soil is actually compensated.
The observation of the changes of the neutral plane refers to the Figure 6.69 and Figure 6.70. The neutral planes are obtained from the simulation using Hardening Soil model parameter (Table 6.5). The neutral plane of the piles obtained from the simulation incorporating creep is located slightly deeper. An example is presented in Figure 6.71. However, further discussion will focus on the simulation results from the simulation using Hardening Soil model.

**Figure 6.69 Changes of neutral plane in pile 70s due to main excavation**
It could be concluded that neutral plane position play less significant role in the main excavation stages. The mobilization of new pile resistance plays greater role. This is certainly different to the diaphragm wall case, when the tip resistance is not heavily influenced. Since the series of unloading and reloading maintains the tip resistance reasonably stable.

Moreover, as observed above, stress relief has also its influence. Compared to the diaphragm wall installation phases, more significant stress relief occurs in the pile.
The comparison between soil settlement and building settlement is summarised in the following tables. The percentage of the measured building settlement is not presented since based on the measurement, buildings settle less than the foundation layer, which is unlikely to occur.

One possible explanation is that the extensometers actually show inaccurate results at the deeper level. It has been understood that the measurement of the extensometer at deeper level cannot be verified by manual measurement. Therefore, the inaccuracy is highly possible to occur, especially after the extensometer experiences long and large displacement.

**Table 6.24 Percentage of calculated settlement of pile 70s to the calculated subsurface settlement – due to main excavation**

<table>
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<tr>
<th>Location</th>
<th>Surface Settlement [mm]</th>
<th>First Sand Layer Settlement [mm]</th>
<th>Pile 70s Settlement [mm]</th>
<th>Percentage of Pile Settlement to the Differential Subsurface Settlement</th>
<th>Neutral Plane</th>
</tr>
</thead>
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<td>10.9</td>
<td>5</td>
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<td>06150402</td>
<td>11.5</td>
<td>6.1</td>
<td>6.9</td>
<td>15%</td>
<td>30%</td>
</tr>
<tr>
<td>06150403</td>
<td>11.3</td>
<td>3</td>
<td>5.2</td>
<td>27%</td>
<td>52%</td>
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<td>1</td>
<td>2.4</td>
<td>15%</td>
<td>52%</td>
</tr>
</tbody>
</table>

**Table 6.25 Percentage of calculated settlement of pile 100s to the calculated subsurface settlement – due to main excavation**

<table>
<thead>
<tr>
<th>Location</th>
<th>Surface Settlement [mm]</th>
<th>First Sand Layer Settlement [mm]</th>
<th>Pile 100s Settlement [mm]</th>
<th>Percentage of Pile Settlement to the Differential Subsurface Settlement</th>
<th>Neutral Plane</th>
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<tr>
<td>06150403</td>
<td>11.3</td>
<td>3</td>
<td>5</td>
<td>24%</td>
<td>40%</td>
</tr>
<tr>
<td>06150404</td>
<td>10.4</td>
<td>1</td>
<td>3.5</td>
<td>27%</td>
<td>40%</td>
</tr>
</tbody>
</table>

**Table 6.26 Percentage of measured settlement of the building and the – due to main excavation**

<table>
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<tr>
<th>Location</th>
<th>Surface Settlement [mm]</th>
<th>First Sand Layer Settlement [mm]</th>
<th>Corresponding Buildings Settlement [mm]</th>
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<td>26.6</td>
<td>9.43</td>
<td>3.9</td>
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</table>
### 6.6.5 Conclusion on Refined Analysis

Based on the results of the refined analysis above, several conclusions can be deducted:

- **After sand fill placement:**
  After the adjustment of the parameters, more accurate comparison can be obtained. The settlements of the First Sand layer and the piles are close to the measured data. It can be deducted that the actual behaviour of the building is close to the behaviour of piles loaded with 70kN of external load.
  Due to the sand fill placement activity, the neutral planes of the piles are shifted. In pile 70s, the neutral planes are shifted down, while pile 100s has a slight increase of neutral plane position. This is concluded based on the calculation results. The difference between the responses is due to the mobilised resistance of the pile. In pile 70s, a transition zone exists, so there is a space for positive skin friction to be mobilised. Once the pile is pushed down by the soil, definitely, the neutral plane will move down. While in pile 100s, the resistance of the pile has achieved its maximum value and the changes only occurs at the location close to the neutral plane.
  It has been understood that the neutral plane position is closely related to the settlement experienced by the pile. The higher the position of the neutral plane, the larger the settlement will be.

- **After diaphragm wall installation:**
  Better fit for the settlement of the soils and the piles is also obtained.
  Based on the analysis, a series of unloading reloading occurs during trenching and concreting. It has been found that Eem clay suffers severely from the effect of trenching. The increase of total horizontal pressure in the soil during concreting does not recover to the original position.
  The settlement of the buildings seems to be close to the settlement of the pile 70s. The neutral planes of the piles, both 70s and 100s, move up as a result of unloading – reloading series during the diaphragm wall installation.
  In the end of the diaphragm wall installation phase, the load distribution of the piles is slightly lower compared to the previous condition after sand fill placement.

- **After pumping test:**
  The simulation still overestimates the settlement, either the surface or the First Sand layer. As found in the previous back analysis, deeper layer governs the settlement of the soil, which is not found in the measurement. No clear conclusion can be found about the comparison.
  It is possible that the extensometers do not capture a small settlement correctly. Another reason is the conservative approach to model the pumping test could induce excessive response of the soil.
Based on the calculation, the load distribution and the neutral plane of the pile remains the same since the pile settles together with the foundation layer and the surface.

- **After main excavation:**
  After the changes in the pre-stress force of the struts, better fit can be obtained for the wall deflection and the First Sand layer’s settlement, despite a few millimetres of discrepancy. Further improvement could be made by adjusting the stiffness of the concrete and the pre-stress force of the struts. Based on this finding, it can be concluded that the properties of the retaining structures are significantly important in governing the soil settlement.

  However, the settlement of the surface is still underestimated. There are couple of possible explanations, including the influence of creep and additional activities inducing additional loading which are not recorded. Using a model incorporating creep, reasonable match could be obtained for furthest measurement point, which is considered experiences less influence from the wall deflection. This means that creep behaviour of Amsterdam subsurface needs to be taken into account in the analysis.

  The main excavation changes pile load distribution significantly. Stress relief during the excavation triggers reduction of pile load distribution. The pile could lose tip resistance significantly. The most significant phase is the excavation close to the depth of foundation layer.
7. Closure

Data analysis and simulation of deep excavation of Ceintuurbaan station in North/South Line project have been performed. The analysis and simulation focus on cross section 13110E. Based on the findings and comparisons, some conclusions will be drawn to provide answers to the research objectives. As the final closure, some ideas will be recommended to be included in further research about soil-structure interaction in deep excavation.

7.1 Conclusions

7.1.1 Significance of Particular Activities Involved in The Deep Excavation Project

Based on the analysis of the monitoring data at Ceintuurbaan station cross section 13110E, it is discovered that preliminary works contribute in large percentage in total soil and building settlement. Preliminary works involved in the project include:

- Additional sand fill placed at the top of the proposed station box as working platform
- Diaphragm wall installation
- Jet grout strut installation
- Pumping test.

Based on the results above, several conclusions can be made. It is evident that preliminary works are indispensable factor to be included in the analysis and design of deep excavation, especially in relation to the soil-structure interaction. As observed, building movement has been induced since the period of preliminary works, therefore, any damages could be triggered not only by the main excavation phases, but also there is a contribution of preliminary works.

Surface settlement induced by preliminary works is between 31%-51% of total settlement, while the contribution of preliminary works in the settlement of foundation layer (NAP - 12m) is 20% maximum. Based on the prism monitoring, building settlement induced during preliminary works period is between 33%-59%.

The most significant activity among preliminary works is the installation of diaphragm wall. In the surface, this activity contributes between 15%-22%, while at the foundation layer, the contribution is 13% maximum. The influence of diaphragm wall installation is more significant to the building settlement, with a range between 32%-41%.

7.1.2 Modelling The Excavation Stages

It has been shown that modelling the complete activities of deep excavation project is possible. Comparison between the simulation and measurement results has been presented. It can be concluded that the simulation produces reasonably good comparison. However, some remarks are presented here.
• Initial condition and sand fill placement period
  Initial condition of the soil is important, especially the in-situ stress. Having the correct in situ stress will enable us to achieve more accurate result. Moreover, modelling piles subjected to long term settlement, such as in Amsterdam, also requires awareness of the initial condition of the pile. In this case, negative skin friction should be taken into account.

• Diaphragm wall installation
  Bentonite and fresh concrete pressure are of interest in the modelling of diaphragm wall installation. Bentonite pressure can be modelled using hydrostatic pressure with a specific unit weight. On the other hand, the use of bilinear pressure profile as suggested by several researchers has been proven as a reliable approach. Refinement could be made by modelling time-dependent fresh concrete pressure. It is believed that the current bilinear pressure has overestimated the real-time pressure. However, fresh concrete pressure is dependent on the pouring rate, temperature, and other factors. Therefore, in the detail modelling of diaphragm wall installation, those factors are required to be taken into account to give more accurate approach.

• Jet grouting and pumping test
  In this case, jet grout strut is modelled using wished-in-place volumetric material. In reality, the installation involves fracturing and also reduces the stiffness of the adjacent soil. Conservative approach is taken by reducing the adjacent sand layers’ stiffness (65% of total stiffness). From the analysis of measurement data, it is evident that the installation did not affect the soil outside the excavated area. It is believed that the diaphragm wall has assisted to reduce the fracturing effect. It might be more significant in other conditions.
  Conservative modelling of pumping test is taken by lowering the groundwater head of the soil in the excavated area uniformly. In more accurate modelling, the position of the well and the fluctuation of the actual groundwater head are required for more accurate modelling.

• Main Excavation
  To model the real behaviour of the soil and building during main excavation phase, the actual condition should be applied, for instance, struts force transferred to the soil, the actual concrete stiffness, groundwater head and pumping test scheme. However, there is still a discrepancy between calculated and measured soil settlement. It is possible that the interface between the diaphragm wall and the soil behaves much smoother. Moreover, including creep in the simulation seems a reasonable approach, especially for deep excavation in very soft soil condition.

In overall, considering the comparison of the simulation results and the measurement, it can be concluded that it is possible to model the entire construction process. The comparison shows that the simulation results agree reasonably well with the measurement results, except the soil settlement during the main excavation. However, as learned in the back analysis, a rigorous assessment on model parameters and the local site recognition are required.
7.1.3 Pile Behaviour and Settlement

7.1.3.1 Triggering Factors of Pile Settlement

As observed in the aforementioned passages, pile behaviour and settlement are dependent on several factors. In the case of deep excavation, the following factors are significant:

- Initial pile condition.
- Load of the piles
- Position of neutral plane.
- Stress relief in the soil
- Foundation layer displacement.

During sand fill placement, the most governing factor is the position of the neutral plane. It has been shown that the settlement of the pile induced by a specific activity corresponds to the neutral plane of the piles. This occurs since there are no changes in the pile tip resistance or stress relief effect.

In the diaphragm wall phases, additional pile settlement is induced not only by the changes in the neutral plane, but also the effect of normal stress changes which affect the load distribution of the pile. A series of unloading and reloading trigger this event. As observed, the neutral planes are shifted up in the end of the diaphragm wall installation.

Unlike to aforementioned observation, the most significant influence comes from the stress relief at the pile shaft and pile tip. It is evident that the largest additional settlement occurs during the excavation to NAP -15m, which is close to the foundation layer level.

The magnitude of the working load applied to the piles has significant influence during sand fill placement and diaphragm wall installation period. On the other hand, both pile groups indicate similar behaviour and settlement during main excavation period.

7.1.3.2 Factors Influencing Difference in Calculated Pile Settlement and Measured Building Settlement

The main remark on the current model is the lack of stiffer soil at the tip of the embedded pile. This will be beneficial to model the installation effect exists at the tip of the pile. As a result of 'soft' behaviour at the tip, the simulation overestimates the settlement.

Moreover, in this current model the stiffness of the building is overlooked. The model for the buildings is required to be improved. Despite the influence of the interface stiffness of embedded pile, it is believed by incorporating the actual structure, a better simulation results can be obtained.

The influence of lateral pile displacement should be investigated further. It is suspected that during main excavation stages, this factor influence pile settlement significantly. By having lateral displacement, pile settlement could be hindered by the adjacent soil and also the flexibility of the pile.
7.1.3.3 Bearing Capacity Problem

Zeevaert [1973] suggested that negative skin friction could induce geotechnical capacity failure. There are some engineers adopt this concept by introducing drag load as additional load in calculating factor of safety of pile capacity.

From the simulation results show above, it is evident that even the pile resistance has reached the maximum value (during sand fill placement period), pile does not plunge through the soil. The mechanism of pile settlement subjected to negative skin friction is governed by the changes of neutral plane and remobilisation of shaft friction. As suggested by Poulos [2008], this should be handled as a serviceability problem of the pile.

Moreover, taking into account the stress relief effect, it is evident that the pile could lose its resistance as shown during the main excavation phases, especially the resistance of the tip. Again, serviceability problem is more appropriate to be addressed in the analysis of pile settlement in deep excavation area.

7.1.3.4 Correlation Between Pile Head Settlement and Soil Settlement

Figure 7.1 is presented to summarise the ratio of additional pile settlement to additional surface settlement in each construction phase. It can be observed that the pattern found in the simulation corresponds to the analysis result on the monitoring data by Korff [2012] (Figure 5.4)

![Figure 7.1 Comparison of additional pile settlement to additional surface settlement in each construction phase](image)

To compare it with the position of the neutral planes of the piles, the following figures are presented. Evidently, the neutral plane position from sand fill placement period to diaphragm wall installation period corresponds to the additional settlement of the pile, since there is a differential settlement in the soil. During pumping test period, the neutral plane...
remains at the same level, while the settlement of the Holocene layers is identical. As explained earlier, during main excavation phases, stress relief and changes in tip resistance have more domination. Therefore, the neutral plane position and the ratio do not coincide.

**Figure 7.2 Neutral plane position of pile 70s during each construction stage**

**Figure 7.3 Neutral plane position of pile 100s during each construction phase**

7.2 Future Research Recommendations

It has been understood that there are some drawbacks in the discussed model. Therefore, the following recommendations are listed to be considered in the future research.
• In some applications, it has been found that diaphragm wall installation phase is neglected. Wished-in-place wall is usually directly applied. However, it has been shown that during diaphragm wall installation phases, there is a complex response in the soil affecting the piles. Hence, modelling a diaphragm wall installation phase is necessary to capture the real behaviour of the soil and its interaction with the adjacent structures.

• The application of the fresh concrete pressure in the model adopts a conservative approach. More accurate pressure envelope should be applied with the emphasis on the time-dependent development of the envelope and the position of critical height.

• To model a complete interaction between buildings and deep excavation, the influence of building stiffness is of importance as well. It would be recommended in the future that the stiffness of the wall and other structures are included. Consequently, the actual behaviour can be captured.

• The use of embedded pile to simulate driven pile is better accompanied by a local stiffened soil to incorporate the real behaviour. From the comparison above, it is one possibility that the interface at the pile tip behaves too 'soft'.

• During the main excavation stages, it is believed that lateral displacement of the pile influences the amount of settlement. Therefore, it is suggested to develop further understanding of the coupled behaviour between lateral and vertical pile displacement.
# REFERENCES


APPENDIX A. Soil Layers and CPT Results
APPENDIX B.  Adopted Parameters for Finite Element Modelling
De samendrukkingsparameters (Cc, etc) zijn op basis van log10. Calpha geldig tussen korrel_huidig en Pgrens

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De dwarscontractiecoëfficient (C) is afgeleid uit literatuuronderzoek.

1) De karakteristiek gemiddelde onder- en bovengrond zijn de grenzen waarbinnen het gemiddelde van de proefresultaten met een kans van 90% ligt.
2) Resultaten gebaseerd op minder dan 3 testen of op basis van "engineering judgement", zijn aangemerkt met rood.
3) Dillantienhoek is gedefinieerd als: \( \theta = \gamma' - 30^\circ \)
4) De laag "aanvulling" is door de verschillende soorten materiaal (hout, puin, grind, klei, veen, zand) zeer divers van samenstelling.
5) De hoek van inwendige wrijving (\(\gamma'\)) is gebaseerd op een rek niveau van 0,5% (zand) of 1,0% (klei).
6) "Delligging" is gedefinieerd van het Holocene is grotendeels gebaseerd op archief gegevens van Omgemag.
7) De dwarscontractiecoëfficient (\(\gamma'\)) is afgeleid uit literatuuronderzoek.
8) De K0 waarde is gebaseerd op de hoek van inwendige wrijving bij bezwijken voor zand en op de Ip waarde voor klei.
9) De E'50 is afgeleid voor een gemiddelde in-situ horizontale korrelspanning.
10) De standaarddeviatie van de doolradialiteit is 500% van het gemiddelde.
11) De samendrukkingsparameters (Cc, etc) zijn op basis van log10. Calpha geldig tussen korrel_huidig en Pgrens.
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<tr>
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<th>k&lt;sub&gt;1&lt;/sub&gt;</th>
<th>c&lt;sub&gt;1&lt;/sub&gt;</th>
<th>v</th>
<th>K&lt;sub&gt;O&lt;/sub&gt;</th>
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De Dilitantiehoek is gedefinieerd als: Dilitantiehoek = \(\phi_i\) - 30°.
## Tracé Noord tot en met Europaplein

**Parameters ten behoeve van Eindig Elementen Model berekeningen**

<table>
<thead>
<tr>
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<th>Omschrijving</th>
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<th>Doorlatenheid</th>
<th>Additioneel</th>
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1. De onder- en bovengrens van de vervormingsberekeningen mht het Hardening soil model, zijn afgeleid uit verschillende typen onderzoek.
2. Dillantietoeknoop is gedefinieerd als: $\psi = \varepsilon_c - 30^\circ$
3. Laag 01, aanvulling is door de verschillende soorten materiaal (hout, puin, grind, klei, veen, zand) zeventeen deel van samenstelling.
4. De hoeveelheid van inwendige vrijheid (q) is gebaseerd op een reik van 15%, dat wil zeggen na bezwijken.
5. De "engineering judgement" van het holocen is grotendeels gebaseerd op archief gegevens van Oogom.
6. De dwarscontractiecoëfficiënt ($\psi$) is afgeleid uit literatuuronderzoek.
7. De K0 waarde is gebaseerd op de hoeveelheid van inwendige vrijheid bij bezwijken voor zand en op de Ip waarde voor klei.
8. De verticale doorlatenheid is gelijk aan de horizontale doorlatenheid, behalve voor laag 04, 08 en 12. Voor deze lagen is de verticale doorlatenheid 2x zo groot.
<table>
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### Noord tot en met Europaplein

#### Parameters ten behoeve van Eindig Elementen Model berekeningen

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<td>22</td>
<td>ZAND, zeer fijn, zwak zilzig</td>
<td>1.0</td>
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<td>23</td>
<td>ZAND, zeer fijn, zwak zilzig</td>
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<td>ZAND, zeer fijn, zwak zilzig</td>
<td>1.0</td>
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<td>0.2 (\eta)</td>
<td>0.7</td>
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</table>

#### Opmerkingen

1. De onder- en bovengrens van de vormingsberekeningen mbt het Hardening soil model, zijn afgeleid uit verschillende typen onderzoek.
2. Dilitatiehoek is gedefinieerd als: \(\psi = \varphi - 30^\circ\)
3. Laag 01, aanvulling is door de verschillende soorten materiaal (hou, puin, grind, klei, veen, zand) zeer divers van samenstelling.
4. De hoek van inwendige vrijrijking (\(v\)) is gebaseerd op een rek niveau van 15%, dat wil zeggen na bezwijken.
5. De "engineering judgement" van het holocene is grotendeels gebaseerd op archiefgegevens van Omegem.
6. De dwarscontractiecoëfficiënt (\(v\)) is afgeleid uit literatuuronderzoek.
7. De K0 waarde is gebaseerd op de hoek van inwendige vrijrijking bij bezwijken voor zand en op de \(\varphi\) waarde voor klei.
8. De verticale doorlatendheid is gelijk aan de horizontale doorlatendheid, behalve voor laag 04, 08 en 12. Voor deze lagen is de horizontale doorlatendheid 2x zo groot.
APPENDIX C. Main Excavation Construction Sequence
APPENDIX D.  Diaphragm Wall Layout and Construction Sequence
APPENDIX E. Jetgrout Installation Layout