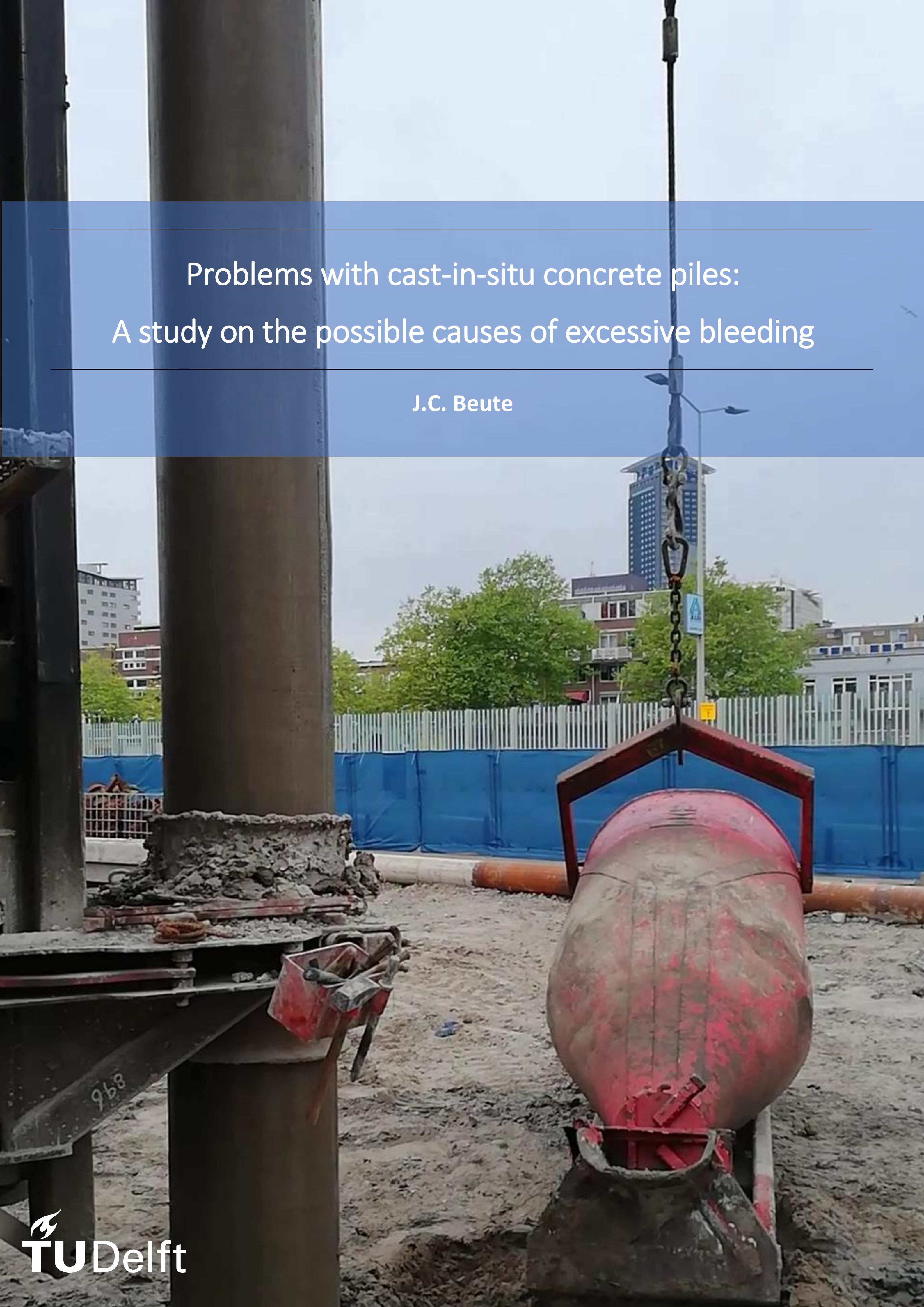


Problems with cast-in-situ concrete piles:

A study on the possible causes of excessive bleeding

J.C. Beute

TU Delft

Problems with cast-in-situ concrete piles: A study on the possible causes of excessive bleeding

By

J.C. Beute

to obtain the degree of Master of Science at the Delft University of Technology,
to be defended publicly on January 13, 2020.

Student number:	4236637	
Committee:	Dr. ir. M. Korff,	TU Delft, Section Geo-Engineering
	Ir. J. H. van Dalen,	TU Delft, Section Geo-Engineering
	Prof. dr. ir. S. Grünewald,	TU Delft, Section Concrete Structures
	Dr. ir. W. Broere,	TU Delft, Section Geo-Engineering
	Ir. F.J.M. Hoefsloot,	Fugro NL Land B.V.

An electronic version of this thesis is available at https://repository.tudelft.nl/.

Preface

Before you lies the result of my final work for the master Geo-Engineering at the TU Delft. The subject is closely linked to the practice which had it positives and negatives and brought me in contact with a lot of people resulting in interesting conversations.

I would like to express my gratitude to my supervisors Mandy Korff, Jan van Dalen, Steffen Grünewald and Flip Hoefsloot for their guidance and feedback throughout the project. I would also like to thank Wout Broere for joining my committee at the final stage of my research. Special thanks to Flip Hoefsloot who was always willing to make time for questions and discussions at Fugro.

Then I would like to thank all the colleagues at Fugro for the provided information and data regarding this topic, for answering my question, for the very pleasant working environment enabling me to focus on my research and for the distraction during the coffee and lunch breaks.

Furthermore, I express my gratitude to friends and family. I cannot thank you enough for all the support you have given me during my studies in Delft. Special thanks to Johan for the very helpful discussions and answering my ‘not always very intelligent’ questions during the period of my thesis. Finally, I want to thank Evelien for your positive attitude, support and patience during the full period of my studies and even more during my thesis.

Rijswijk, January 2020
Jeroen Beute

Abstract

First of all, an overview was created of different problems that can occur by the construction of cast-in-situ concrete piles, discussing the mechanisms that cause these problems. From the overview it was concluded that little information is present about the exact cause of excessive bleeding. Two possible causes were identified, excess pore water pressure in the surrounding soil caused by the pile installation and concrete stability, these are further researched.

Comprehensive measurement data of the hydraulic head in an intermediate confined sand layer, during the installation of displacement piles, showed a consistent trend. During the descending phase a large increase in the hydraulic head was measured which can be described by volume displacement in the sand layer. In addition, a cumulative effect in the hydraulic head was found at the start of withdrawal of the temporary casing as the excess pore water pressure, resulting from the volume displacement, was not fully dissipated. During withdrawal, two additional pore water pressure increments can be distinguished. The direct increase starting from the onset of withdrawal has been interpreted as being caused by the reaction force of the machine on the ground resulting from the pull up force on the casing necessary to get the casing in motion. Unfortunately, no conclusive cause was found for the second increase in hydraulic head regarding the processes that take place during withdrawal. This calls for further research as the hydraulic head after withdrawal will influence the interaction between the fresh concrete and the surrounding soil.

Then a literature study on (excessive) bleeding in cement based materials was performed to assess how concrete stability influences the initiation of bleeding. The literature study showed that in order to capture the full bleeding phenomenon three processes can be distinguished. Firstly, if the governing process is sedimentation, which is characterized by negligible effective stress in the cement based material, segregation and channelled bleeding can arise. After settling or directly, depending on the stability of the mixture, the governing process in the cement based material is consolidation which means that the excess pore water pressure inside the concrete wants to dissipate, either horizontal (filtration) or vertical (bleeding). The existence of flow paths herein is very important. Finally, cement hydration should be incorporated in the analysis as the bleeding process is also governed by the chemical processes in the cement paste. Literature, however, confirms that sedimentation or consolidation are the initial mechanisms which are ceased by cement hydration.

No literature was found that succeeds to fully capture the intrusion of external ground water into the fresh concrete. However it is likely that when a high external ground water pressure is present it will follow the path of least resistance and add up to the water flow through the fresh concrete pile if a connection is found between a drainage path in the fresh concrete and the excess pore water pressure in the surrounding soil.

Contents

1. Introduction

For the construction of buildings different types of foundation can be used. Foundation on piles is very common in the Netherlands because of the soft soils in the underground. Requirements are set to nuisance and vibration for the installation of the piles depending on the distance to neighboring buildings, houses or other structures. For each pile type, the corresponding pile installation method will have different effects on the soil resulting from vibrations, soil displacement or soil removal. As a result of the built-up areas and the heterogeneous soil conditions in the Netherlands a typical category of piles is ‘cast-in-situ concrete piles’, because these can be constructed to different depths within a project and can be installed with limited vibrations and nuisance. Installation of cast-in-situ concrete piles can be distinguished in two different systems, Soil displacement pile systems (“B 4400 In de grond gevormde paalsystemen, met volledige of gedeeltelijke grondverdringing”, Handboek Funderingen Deel B) and Non-displacement pile systems, with auger (“B 4500 In de grond gevormde paalsystemen, met avegaar”, Handboek Funderingen Deel B) from which examples are shown in Figure 1.1 and Figure 1.2, respectively.

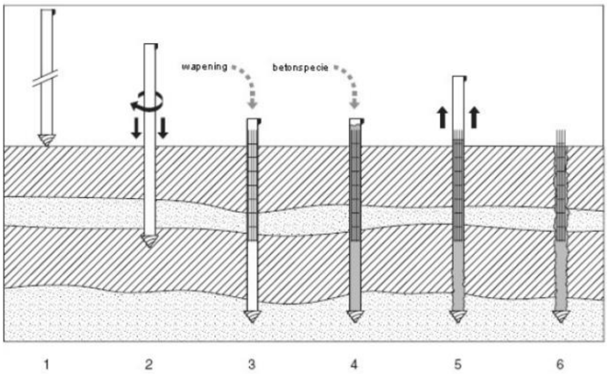


Figure 1.1: Example soil displacement pile system (“B4430 Schroefpaal met verloren punt”)

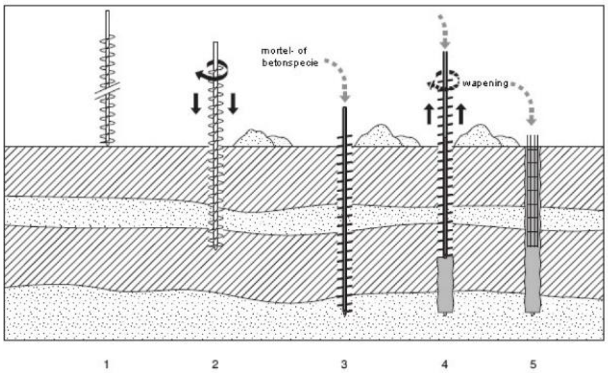


Figure 1.2: Example non-displacement pile system, with auger (“B4510 Schroefpaal”)

The installation method of the piles is different but both systems have in common that the concrete is placed in-situ. After withdrawal of the casing or auger the freshly poured concrete shaft will be in contact with the surrounding soil, that will act as a natural casing for the fresh concrete. This means that the quality of the final ‘product’ is not known beforehand and will depend on the hardening process and the soil conditions. The installation method, execution and soil stratigraphy will influence these soil conditions and this will affect the interaction between the soil (meaning the soil skeleton and ground water) and the fresh concrete. In practice, this has resulted in unpredicted problems influencing the pile integrity.

1.1 Problem Statement

During the installation of cast-in-situ concrete piles several problems still occur in practice that influence the integrity of the piles. The possible problems and their causes are often not taken into account in the design stage and the contractor also omits, or misses the expertise, to do a comprehensive study to predict certain problems. In general, the contractor accepts the work based on experience obtained at other projects in the vicinity of the new project location. This implies that the current practice of project design is not focused on the practicability of foundation work resulting in the possibility of anomalies during or after pile installation. In The Netherlands a recommendation is published to give some guidance to the construction of piled foundation and is called CUR Recommendation 114 “Toezicht op de realisatie van paalfunderingen” (CUR Bouw & Infra, 2009) in which some problems are addressed but it is not detailed and no qualitative measures are presented.

One of the problems that can arise during installation of cast-in-situ concrete piles is water that percolates from the pile head along the reinforcement bars, through the center or at the perimeter of the pile. This problem is called ‘Bleeding’. The water that percolates from the pile head must either come from the concrete itself or from the surrounding ground water. When this results in preferential water flow pathways, or bleed channels, the pile integrity can be affected. In several construction projects in The Netherlands this problem has occurred, without expecting it, leading to project delay and additional costs. Although it is known that cast-in-situ displacement piles, either driven or screwed, might lead to excess pore water pressures in the soil, the mechanism that leads to a flow of water through the pile is not fully understand and whether this is a result of the generated excess pore water pressures is to be investigated.

1.2 Research Objective

The objective of this master thesis is to learn why these problems arise and bring attention to certain soil situations or design aspects which might influence the final quality of the pile. As a result an overview will be reviewed with problems and possible causes, and if possible mitigating measures will be presented.

In addition, this research will focus on the mechanisms causing (excessive) bleeding found in cast-in-situ full displacement piles. When these mechanisms are fully understood models will be used to describe the generation of excess pore pressure and excessive bleeding. The models are being validated with project data.

1.2.1 Research Scope

As above mentioned, this research will concentrate at a soil displacement pile system. Different installation techniques exists for this pile system all having different influence on the soil, therefore a choice is made to focus on full displacement screwed piles with lost tip, also referred to as Fundex piles, only. To support penetration through hard soil layers grout injection can be applied.

Further research will focus on the problem of (excessive) bleeding. Bleeding is associated with a mechanism of concrete stability and excessive bleeding is referred to when considering a large amount of water flowing through the pile for a longer time period than normally expected resulting in a channel when the pile is hardened.

1.2.2 Research Questions

The main research question is:'

"What are the possible causes of excessive bleeding after installation of cast-in-situ concrete piles?"

For which the following sub-questions were defined:

1. What are the main problems during installation of cast-in-situ concrete piles and what are the mechanisms that can cause these problems?
2. How is the pore water pressure in subsequent soil strata influenced by the installation of full displacement piles and how can this best be modelled?
3. How does the water flow develop in the fresh concrete pile and is this only a result of the excess pore water pressures?
4. How does the water flow influence the integrity of the cast-in-situ concrete pile?
5. Can this mechanism be predicted prior to pile installation and what are the possibilities to reduce the risk of occurrence?

1.3 Approach

The research starts with a literature study on the problems that might occur during the installation of cast-in-situ concrete piles and their causes, to be able to understand the mechanisms that explain these problems. Because little has been written in research papers or conferences about the problems that have occurred at piling construction, this part of the literature study focuses on project data acquired at Fugro. An overview will be reviewed that creates understanding in the possible causes of the problems that arise with the installation of cast-in-situ concrete piles. This overview will form a start to see which mechanisms play a role and what is known and unknown about these mechanisms. From this overview a problem will be chosen to do more extensive research on. In addition, the installation of the hollow steel casing results in a volume displacement, this can either be described by means of a cavity expansion method or a geohydrological model. Subsequently, these models will be compared to see which one is able to describe the process most accurately. Then, it will be necessary to understand the mechanisms of concrete stability, how the mixing water that is used for the production of concrete behaves and how this is related to bleeding, which is a part of concrete stability itself.

Then, project data will be used to focus on the possible cause of excessive bleeding, namely excess pore water pressures. The extensive hydraulic head measurement data will be analyzed to find the effects of the pile installation on the surrounding soil and to see whether this can explain the bleeding phenomenon. Following from the measurements, the water pressure on the fresh concrete column can be compared to see if a disturbed equilibrium can cause the water to enter the fresh concrete.

If no conclusive result follow from the project data further research will be done to the other possible source of water, the fresh concrete itself. This will be done by means of a literature study to find out what is known about concrete stability in relation to excessive bleeding that is found in practice. In this part references are made to bleeding tests that are used nowadays to see if models are created that describes this process.

Furthermore it would be beneficial to find possible solution to this problem, either in the subsoil or in the concrete mix itself. When looking to the possibilities in concrete mix design current guidelines will be taken into account together with practical considerations like money and practicability.

1.4 Outline of the report

In Chapter 2 a literature review will be given on the different pile systems, their installation techniques and the similarities and differences between the pile systems based on the "SBR Handboek Fundering versie B" (Smienk & De Quelerij ,2010). Currently also new techniques are on the market and the book is a bit outdated but the main techniques are still similar. The influence of the pile installation on the subsequent soil strata will also be discussed in which two models will be reviewed that focus on the generation of excess pore water pressures. Thereafter, the first basics of concrete stability will be presented because this explains the behaviour of the fresh concrete. Additionally, bleeding is one aspect of concrete stability and it is important to describe the difference between normal concrete bleeding and excessive concrete bleeding found in cast-in-situ concrete piles.

In the following chapter, Chapter 3, the main problems that can occur during and after installation of concrete cast-in-situ piles will be presented. By means of an overview of the problems and their potential causes this can be related to specific situation in which the risk of a deficiency that influences the pile integrity is present. The causes can be divided into soil conditions, pile installation and concrete stability. Than a choice is made to further focus on one of the problems.

Firstly, the generation of excess pore pressures due to the installation of the piles will be discussed in Chapter 4. This is done based on extensive hydraulic head measurements following from the project data. The different causes of the increases in hydraulic head will be explained. This chapter will relate the external water pressure with the fresh concrete pressure by means of a comparison. And ends with a description of the influencing factors and mitigating measures related to the external pore water pressures.

In addition to the external pressures, Chapter 5 continues with the fresh concrete itself. By means of a literature study the state of the art information is gathered that focused on bleeding, channelling and other concrete related research that tried to describe the occurrence of bleeding in fresh concrete. This chapter also ends with influencing factors and mitigating measures but now relating to the water inside the fresh concrete.

Finally, both possible source of water have been discussed and the conclusion following from these analyses will be discussed and recommendation will be given for further research on the phenomenon of excessive bleeding in cast-in-situ concrete piles.

2. Literature Study

Firstly, the different pile systems of cast-in-situ concrete piles will be reviewed. This includes the installation technique addressing their differences and similarities. Secondly, the influence of a soil displacement pile system on the subsequent soil strata will be discussed because of the objective of this research focuses on excessive bleeding, potentially caused by excess pore water pressures in the surrounding soil. Two models that can describe the influence of volume displacement are discussed. Finally, the principles of concrete stability will be explained to have a basic understanding of the properties and specifications of the concrete itself.

2.1 Pile Systems

As stated in the introduction of this chapter only the pile systems of the category 'cast-in-situ concrete piles' will be reviewed. There are other pile categories like wooden piles, precast concrete piles or open ended steel pipe piles but these do not fall in the scope of this research. The cast-in-situ concrete pile systems can be divided into two different installation techniques. The first pile system is installed with full or partial soil displacement and can be subdivided into driven piles or screwed piles. The second pile system is installed with soil removal which is described by bored piles due to the installation with an auger system.
In general, the main advantages of screwed cast-in-situ pile systems are the variation in length of the piles within a project and construction with minimum vibration and noise. The main disadvantage is the unknown result of the concrete pile due to its in-situ construction below ground level. Other specific advantages or disadvantages of the different systems will be discussed in the following paragraphs.

2.1.1 Soil displacement pile systems

As the name of this system already reveals, the pile systems have full or partial soil displacement depending on the installation technique. Due to the lateral soil displacement this system has beneficial pile factors in the bearing capacity calculation compared to a non-displacement pile system, which is accounted for in the NEN 9997-1(2017). In addition, this system is preferable when the site contains contaminated soil and therefore no soil removal is accepted.

Driven pile systems

The pile types that are categorized in the driven pile systems are assumed to be driven by a drop hammer or by means of vibrations. The 'Vibropaal' as well as the 'Franki' pile are driven with an external drop hammer, although the 'Franki' pile can also be installed with an internal drop hammer since the casing is sealed off with a concrete plug instead of an end plate or shoe. The 'Voton HSP' (High Speed Piles) and the 'Vibration Fluidization pile' are both installed with a vibratory hammer plus a vertical force but the 'Vibration Fluidization pile' also uses water nozzles to aid the penetration. These pile systems can be described as full-displacement piles because during installation the soil around the casing is pushed aside. Additionally, these systems exert vibrations on the soil. So due to the volume displacement and the vibrations the disturbance on the surrounding soil is large.

The main disadvantage of these systems are the created vibrations during installation which can be harmful to previously installed piles or nearby structures. In addition, nuisance will be present due to the drops of the hammer or the vibrations. Both effects are disadvantageous when constructing in a build environment.

Screw pile systems

The pile systems in this category are screwed to the desired depth; this is a combination of torque and pull-down to get the closed-ended steel casing down. The benefit over the driven pile system is that during installation no vibrations are produced, reducing the risk of damaging neighboring buildings. Although considerable depths can be reached with these systems, hard soil layers like dense sands might be hard to penetrate and therefore some systems can be expanded with grout injection.

A lot of different installation techniques exist that can be categorized as screw pile system. The systems distinct in the way the soil is displaced by either the pile tip or a displacement body. Most of the pile systems can be categorized as full-displacement piles, but differ in the way they disturb the soil. The 'Fundex' pile system does only displace the soil during installation. Other pile systems that are installed in the same way have a larger displacement body at the pile tip compared to the diameter of the casing and therefore have disturbance during installation as well as during withdrawal, typical pile systems are 'Atlas' pile and 'Olivier' pile, see Figure 2.1. Due to the displacement body of the latter two pile systems the pile shaft can be constructed with a helical (screw) shaft when the casing is withdrawn counter-clockwise or with a cylindrical shaft by either rotating in the same direction on extraction as for installation or by straight withdrawal of the casing. The benefit of a helical shaft is the advantageous pile-soil interaction.
Finally, some pile systems can be classified as partial displacement piles like the 'Omega' pile system and the 'DPA' pile system (Drilling Push Aside). These systems combine an auger system with a displacement body. The auger system transports the soil vertically to the displacement body where the soil is laterally displaced, but at the final 0.5m to 1.0m the soil is negligible displaced. During withdrawal the soil will be displaced by the displacement body again, leading to additional soil disturbance.
It might be concluded that all of the systems disturb the surrounding soil in either way, which will influence the interaction between the soil and the fresh concrete after the pile is finished.

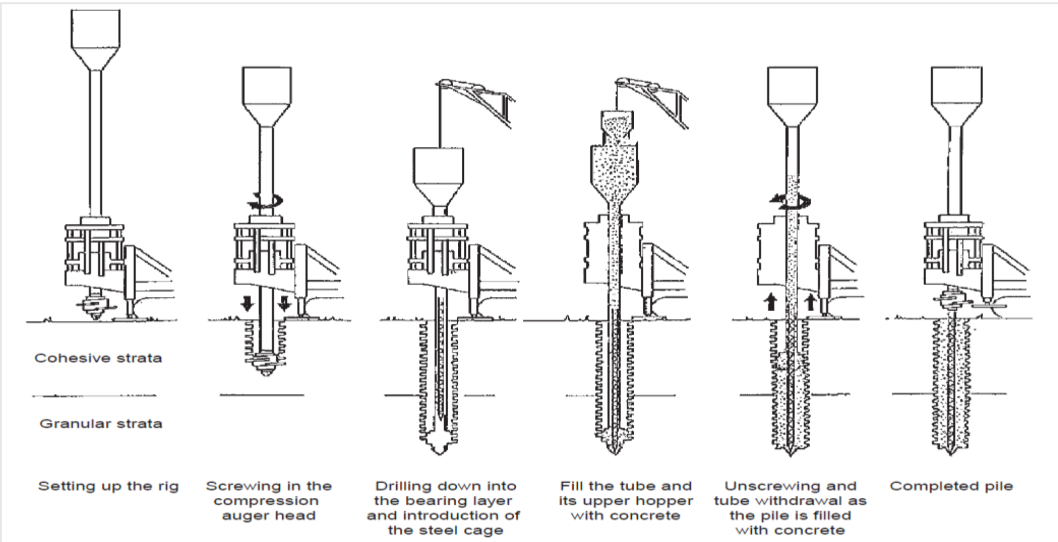


Figure 2.1: Atlas pile system

2.1.2 Non-displacement pile systems, with auger

This category can be distinguished by pile installation with soil removal. Non-displacement pile systems are constructed by removing a cylinder of soil from the ground and replacing the void with concrete, thereafter the reinforcement is placed. In some systems the reinforcement can be placed in the hollow stem, but the diameter of the reinforcement will be limited. The concrete is pumped through the hollow stem and placed under pressure, this is different compared to the previously mentioned systems in which the concrete is poured from large heights. The auger stem is rotated in the ground so no vibration and nuisance will be produced, which is a large advantage over the driven pile systems. The main disadvantage of these systems are the removal of soil, which is often prohibited if a site has contamination. In addition, the removal of the soil can negatively influence the in-situ soil stresses subsequently a lower bearing capacity coefficient for the shaft friction need to be applied for this type of pile systems.

Bored pile systems

In this research the bored pile systems are referred to when piles are installed by means of an auger system that has the characteristic of soil removal instead of lateral soil displacement, see Figure 2.2. The continuous flight auger (CFA) is a typical example of a bored pile system. Another system is the 'Buisschroefpaal' which has a larger diameter auger stem. Both systems can be installed with an additional outer casing that protects the surrounding soil and prevents over excavation. These systems can also be used to build a secant pile wall.

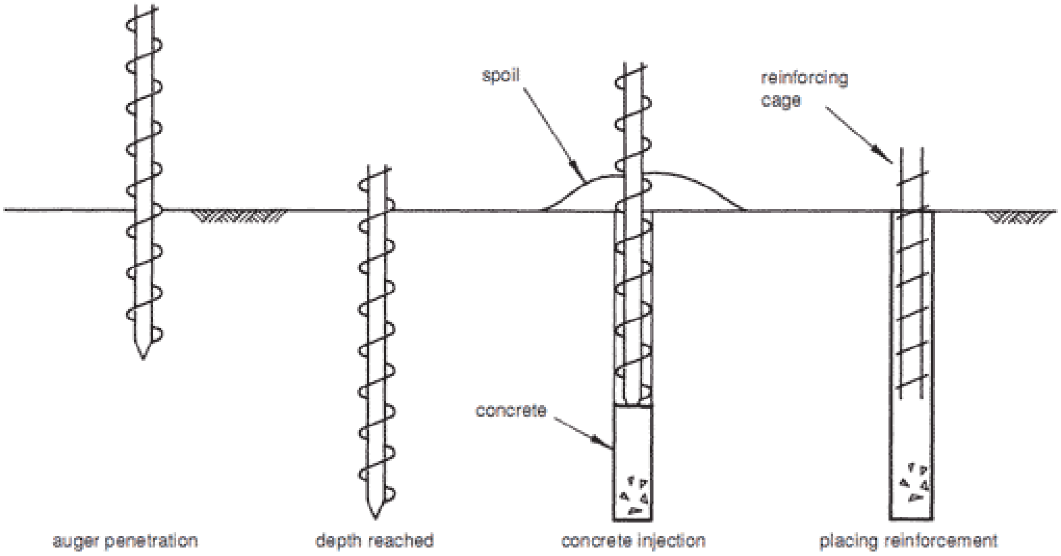


Figure 2.2: Installation sequence of a bored pile system

2.2 Influence of Volume Displacement on Surrounding Soil

The installation of the different pile systems will in either way disturb the surrounding soil. As quickly addressed in the previous paragraph the disturbance will depend on the pile systems and the corresponding installation technique. Additionally, the degree of disturbance will also depend on the soil that will be encountered during installation. Some characteristics that have an influence on the degree of disturbance are the type of soil and its relative density.
The relative density for loose to medium dense cohesionless soil is increased (compaction) close to the pile due to lateral displacement, vibrations and/or shear exerted on the surrounding soil by the installation sequence of a displacement pile system. Under drained conditions the water has time to dissipate resulting in increase of the effective stress only. If these soils are saturated and contain silts or fine sands, decreasing its permeability, positive excess pore pressures might arise due to the quick installation process relative to the permeability of the soil. This will reduce the soil shear strength and hence the resistance to the installation of the pile which enhances the penetration. If the cohesionless soils are dense or very dense the relative density may actually decrease in small zones (Dijkstra, 2009) since dense soils tent to dilate during shearing. Negative excess pore pressures arise which will temporary increase the resistance, but in time these negative pore pressure will dissipate.

In contrast to cohesionless soils, cohesive soils have in general very low permeability. Therefore, restricting the quick dissipation of the water leading to undrained soil behaviour. Undrained soil behaviour means that no drainage is possible from a soil, since the load is applied so quickly and the permeability is so small that there is no time for outflow of water and no consolidation can occur (Verruijt, 2001). Another feature of undrained behaviour is that the soil volume remains constant. During pile installation the volume of the pile will be displaced, so this will lead to heave and lateral displacement of the soil (Hagerty & Peck, 1971). Randolph (2003) states in his research that 'the excess pore pressures that are generated by pile installation arise from two sources: changes in mean effective stress during shearing and partial remolding of the soil (which will give rise to positive excess pore pressures for lightly overconsolidated clay, and negative pore pressures for potentially dilatant, heavily overconsolidated clay), and increases in mean total stress due to outward 'expansion' of the soil to accommodate the pile volume.' The excess pore pressures in cohesive soils will dissipate over time, allowing the soil around the pile to consolidate. During consolidation, the soil increases in strength and so does the bearing capacity of the soil (soil set-up). The prediction of the resulting shaft capacity has been point of interest in literature for some time but the focus of this research is on the pile-soil interaction in the short period of time after installation. Therefore focusing on the initial influence that causes the generation of excess pore pressures and/or soil stresses in the surrounding soil. Two models that can describe these generations are (1) the Cavity Expansion Method (CEM) and (2) a Geohydrological model. These will be further elaborated on in the following two paragraphs.

2.2.1 Cavity Expansion Method

The cavity expansion theory can be applied for several geotechnical problems like the penetration of a CPT, the interpretation of a pressuremeter test or the installation of displacement piles. The latter will be of interest in this study but to be able to apply the cavity expansion theory to this problem some justification should be made because this problem concerns the creation of a cavity starting with an initial radius of zero instead of the expansion of an initial cavity.

Theory of cavity expansion

The cavity expansion theory is based on the expansion of an initial cavity a_0 to a larger cavity by an increase of the pressure inside this cavity. The increase of the radius of the cavity will lead to variations in the stresses and strains in the soil surrounding the cavity. The cavity expansion theory can be described in a cylindrical coordinate system or a spherical coordinate system. To apply the cavity expansion method for the process of pile installation it is assumed that the effects over a large part of the shaft are similar to the expansion of a cylindrical cavity under plane strain conditions (Randolph, Carter and Wroth, 1979), see Figure 2.3. Due to the assumption of plane strain conditions in the vertical direction, all the movements will take place in the horizontal plane. Radial and circumferential directions are then directions of principal stresses and strains.

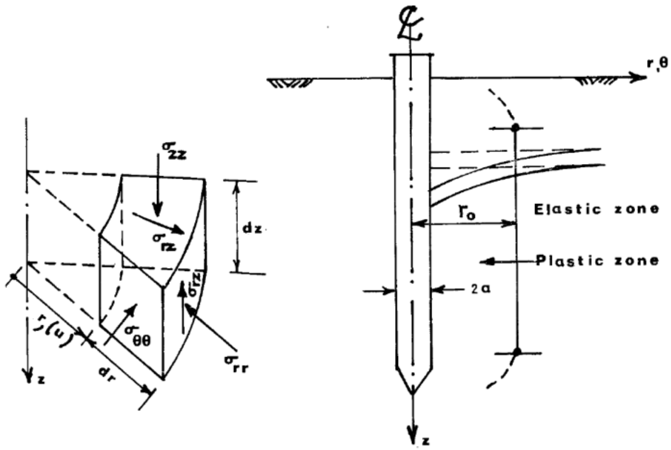


Figure 2.3: Cylindrical coordinate system for the idealized system (From Butterfield and Bannerjee, 1970)

When a displacement pile is installed, the volume of that pile will be displaced resulting in stresses and strains in the surrounding soil. Consequently, the soil fails when the deviatoric stress reaches a particular value that depends on the soil model (failure criterion) that describes the soil. If this value is reached the soil will enter into the plastic phase, this will start first at the wall of the cavity and will propagate into the mass, see Figure 2.4. The boundary between the elastic and plastic strains is denoted by *R* and depends on the created cavity (or pile radius).

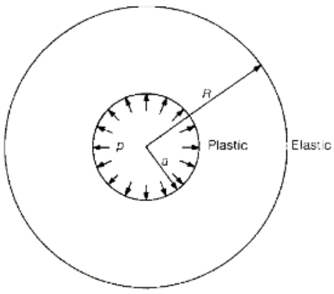


Figure 2.4: Cavity expansion problem (From Carter, Booker & Yeung, 1986)

Solutions to the cavity expansion problem for different geotechnical problems can be found in literature. In this section a general overview will be given for these solutions addressing different types of soil and soil models with the general content relating to pile installation. A distinction can be made between closed form analytical solutions and numerical models.
Baguelin, Jézéquel and Shields (1978) gave expressions for the stress field around a pile for two different elastic plastic soils, a purely cohesive soil and a soil with friction and cohesion, assuming a cavity of initial zero radius has been expanded to a finite radius. Randolph and Wroth (1979) present an analytical solution for the radial consolidation of clay around a driven pile, assuming that the soil skeleton deforms elastically. The consolidation analysis was based on an initial excess pore pressure distribution due to the expansion of a cylindrical cavity in an ideal elastic, perfectly plastic material. Carter, Randolph and Wroth (1979) made a numerical model and found very good agreement with the result of their analytical solution for an ideal elastoplastic soil model. They found that the consolidation which follows an undrained cavity expansion was

essentially the same for (1) cavity creation with a final radius r_0 and (2) cavity expansion from an initial radius a_0 to $2a_0$, where $a_0 = \frac{1}{\sqrt{3}} r_0$ if both types of deformation occurred at constant volume. Carter et al., (1979) also gave preliminary results for a case of a soil modelled as a work hardening elasto-plastic material, based on a modified Cam-clay model and concluded that the dissipation of pore pressures was relatively unaffected by the choice of soil model while the predicted stress changes were. Randolph, Carter and Wroth (1979) extended the preliminary results to include an investigation of the effect of the past stress history of the soil on the stress changes due to pile installation with their numerical analysis. Carter, Booker and Yeung (1986) found a closed form solutions for the expansion of a cylindrical or spherical cavity in an ideal, cohesive frictional soil. But Yu and Houlsby (1991) stated that there are certain unjustified approximations made in the analysis of Carter et al., (1986) and presented another analytical solution for the stress and displacement fields in the soil surrounding the expansion of a cavity. They modelled the soil as linear elastic-perfectly plastic, using a non-associated Mohr-Coulomb yield criterion and added dilation in their analysis. Collins, Pender and Yan (1992) produced semi-analytical solutions for cavities expanding under drained conditions from zero initial radius in sands using a state parameter model. Finally, Collins and Yu (1996) expand the cavity expansion theory for pile installation with an analytical study of the undrained expansion of cylindrical and spherical cavities in soil with various critical state soil models that is applicable to large strain cavity expansion in any isotropically hardening material. And found that their model is more applicable for cohesive soils with a high over-consolidation ratio than the numerical model that Randolph et al., (1979) used.

The abovementioned models can predict the resulting soil stresses for the case of drained soils and the generation of excess pore pressures in the case of undrained soils depending on different soil models all incorporating the formation of a cavity equal to the displaced soil volume. The cavity expansion theory has mainly been used for the substantiation of the increase in bearing capacity of driven piles after consolidation (soil set-up), but might also be of interest when looking at the stress equilibrium during the earlier stage when the fresh concrete gets in contact with the surrounding soil.

2.2.2 Geohydrological Model

In general, geohydrological models are used for the hydraulics of pumping and recharging wells and to determine the hydraulic properties of an aquifer during a pumping test. The hydraulic properties are calculated based on the measurements of the drawdown in piezometers at radial distances (r). But if the hydraulic properties of the aquifers at a site are known, or estimated with a good accuracy, such models can be used to calculate the water level as a function of radial distance and time. Hoefsloot (2001) addressed the possibilities of such a model for the prediction of excess pore water pressures in front of a tunnel boring machine (TBM). The cause of the generation of the excess pore pressures in front of the TBM was explained by the volume displacement of the pore water. The process of the installation of a displacement pile can be assumed to be similar to the boring of a TBM regarding the displacement of the pore water. In addition, it is known that during installation of displacement piles excess pore water pressures are generated. Thus it is assumed that a similar model can be used to calculate the excess pore water pressure during the installation process of displacement piles.
Logically, there will be no extraction of water but a discharge into the system. This might be addressed as a recharging well but an additional function is needed to convert the displaced volume of the pile towards a discharge.

The application of a geohydrological model depends mainly on two things, (1) the aquifer system and (2) the type of flow. There are three main types of aquifers: confined, unconfined and leaky. A confined aquifer is bounded above and below by an aquiclude, an impermeable geological unit that does not transmit water at all. An unconfined aquifer is only bounded below by an impermeable layer and hence the water table is free to fall or rise. And the leaky aquifer, also known as a semi-confined aquifer, is an aquifer whose upper and

lower boundaries are aquitards or one boundary is an aquitard and the other is an aquiclude. Aquitards are geological units that restrict the flow of groundwater, but are permeable enough to transmit water in significant quantities when viewed over large areas and long periods (clay is a typical example of an aquitard). Besides the different aquifer systems there are two types of flow that influence the equations that can be used in a geohydrological model. The first type is steady-state flow which is independent of time. This is often assumed to be applicable when the changes in the water level have become so small with time that they can be neglected. The second type of flow is unsteady-state flow which occurs from the moment pumping starts until steady-state flow is reached and is considered when the changes in water level are measurable. When considering the relatively quick process of pile installation and subsequent reaction of the groundwater in the soil it can be assumed that the aquifer system will not reach steady-state conditions and the equations that are applicable must be described by unsteady-state flow.
Considering the different aquifer systems for the generation of excess pore pressures during the installation of displacement piles the unconfined aquifer system is not applicable while the water table is free to rise to the ground surface as stated before. When looking at typical Dutch soil conditions, that is in the scope of this research, this can best be described by a multilayered system of more permeable sandy soils and less permeable soft soils. Depending on the permeability of the soft soil layers the aquifer system must be regarded as a confined or leaky aquifer, therefore both system will be discussed in more detail below.

When considering both aquifer systems the dimensionless parameter 'Aquifer Storativity' should be introduced first. Aquifer storativity, *S*, can be defined as the volume of water released from storage (or added to it) per unit horizontal area of aquifer and per unit decline (or rise) of piezometric head, ϕ. This stems from the compaction of the aquifer due to increasing effective stress and the expansion of the water due to decreasing pressure during pumping, and vice versa during recharging.
Theis (1935) was the first to develop an equation for unsteady-state flow in a confined aquifer. The equation takes the storativity of the confined aquifer and a time factor into account, see (4).

$$s(r,t) = \frac{Q_0}{4\pi kD}\int_u^\infty \frac{e^{-u}}{u}du \tag{1}$$

The dimensionless variable, *u*, appearing in (1) is defined as:

$$u = \frac{r^2 S}{4kDt} \tag{2}$$

s (r,t)	Drawdown at a distance r from the pumping well at time t [m]
Q_0	Steady well discharge, starting at t=0 [m^3/day]
kD	Transmissivity of the aquifer [m^2/day]
r	Distance from the well [m]
S	Storativity of the confined aquifer [-]
t	Time since pumping started [days]

The exponential integral in (1) is known as the well function for confined aquifers or Theis well function and is generally denoted by W(u). Cooper and Jacob (1946) suggested a simple but approximate method to compute Theis well function by replacing W(u) by an infinite series. With their method the outcome of the Theis well function can be approximate with small errors when 'u' becomes small, say u ≤ 0.01, (i.e., for a large time at a given distance).

In nature, leaky aquifers occur far more frequently than the perfectly confined aquifers since confining layers overlying or underlying an aquifer are seldom completely impermeable; instead, most of them leak to some extent (Kruseman and de Ridder, 1991). Kruseman and de Ridder(1991) present several methods to determine the drawdown as function of radial distance and time for unsteady-state flow in a leaky aquifer, but the only method in which no piezometer measurement data is necessary is the Walton's method. The Walton's method uses the formula derived in Hantush and Jacob (1955) and assumes that all the water flowing to the pumping well comes from elastic storage in the confined aquifer and leakage across the confining layer (aquitard), so the storativity of the aquitard is neglected. The Hantush-Jacob equation looks similar to the Theis equation, only the well function becomes the Hantush well function presented in (3) and (4).

$$s(r,t) = \frac{Q_0}{4\pi kD} W(u, \frac{r}{\lambda}) \tag{3}$$

$$W\left(u, \frac{r}{\lambda}\right) = W\left(\frac{r^2 S}{4kDt}, \frac{r}{\lambda}\right) = \int_{\frac{r^2 S}{4kDt}}^{\infty} \frac{1}{y} \exp\left(-y - \frac{\left(\frac{r}{\lambda}\right)^2}{4y}\right) \tag{4}$$

where

λ Leakage factor, $\lambda = \sqrt{kDc}$ [m]

c Hydraulic resistance of aquitard [days]

So, with use of the Hantush well function the drawdown in a leaky aquifer system for unsteady-state flow can be calculated. If it is assumed that the volume of water displacement is analogous to pumping or recharging in a well than this method can be used to calculate the excess pore water pressure that is generated during the installation of a displacement pile system passing a leaky aquifer.

2.3 Concrete Characteristics

First of all, concrete is a mixture of aggregates, water, cement and admixtures. These constituents will determine the rheology of the concrete that in turn will influence its properties and characteristics. The rheology of concrete is described by the yield stress and (plastic) viscosity hence influencing its behaviour during casting. The EFFC/DFI (2018) listed the following key rheological characteristics for fresh concrete:

- Workability (the general term defining the ability of the concrete to fill the excavation, self-levelling and self-compacting under gravity);
- Workability retention (defining how long the specified fresh properties will be retained);
- Stability (resistance to segregation, bleeding and filtration);

and presented the dependencies between composition, rheology, related characteristics, and overall requirements in a clear scheme which is illustrated in Figure 2.5.

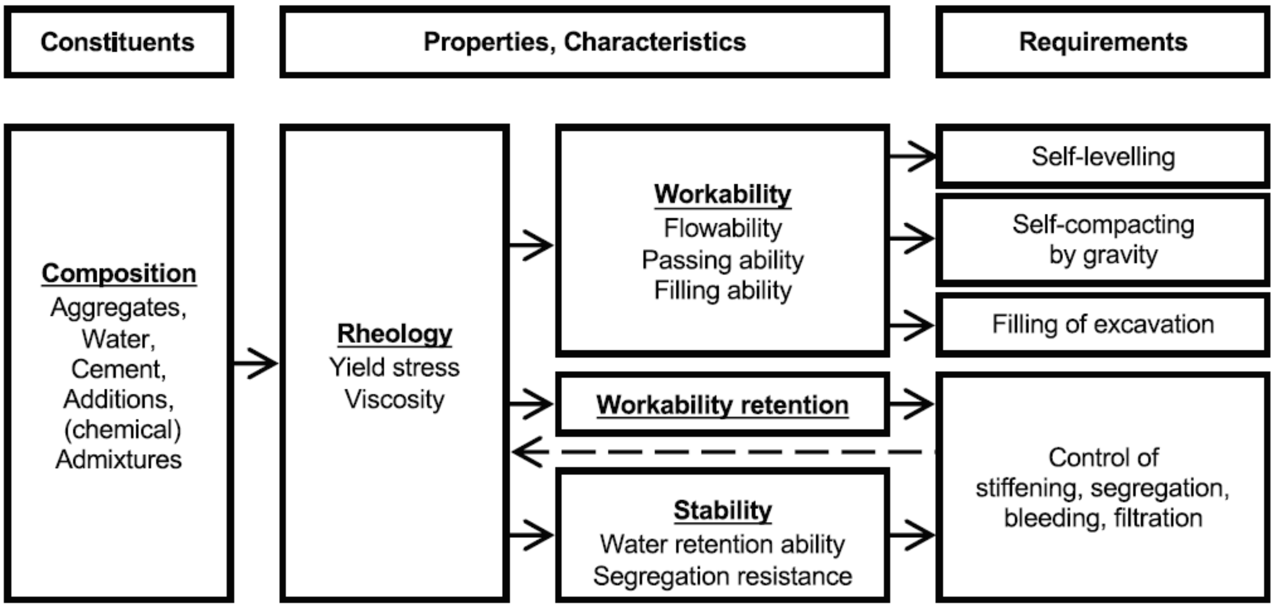


Figure 2.5: Dependencies between composition, rheology, characteristics and requirements (From EFFC/DFI, 2018)

To be able to successfully place a cast-in-situ concrete pile all of the key characteristics should be taken into account. Starting with the time needed for transportation from the concrete plant to the project site, casting the pile until the setting and hardening of the concrete all of these phases should be considered. Minimizing the pour durations and transportation time reduces the need for extended workability retention and any subsequent risk of increased concrete mix sensitivity. The workability should be such that the concrete is easily placed, can fill the created pile volume either under pressure or under its own weight and suitable to flow around the reinforcement necessary for providing sufficient concrete cover.
Another important characteristic regarding the final quality and integrity of the concrete pile is the stability of the concrete. Concrete stability is defined as its ability to retain water (filtration and bleed) and resistance to static segregation. Although in practice some water loss from fresh concrete will always occur, which is visible by a thin layer of water at floor slabs or small water flow arriving at the pile head. However, to prevent that this turns into preferential water flow pathways concrete stability is important.

There are two mechanisms for water loss from fresh concrete. These can be described by ‘filtration’ and ‘bleeding’. Another aspect of concrete stability is the resistance to segregation. These three mechanisms will briefly be discussed.

Filtration
Fresh concrete in deep foundations is subject to high head pressures which in turn lead to high pore-water pressures in the fresh concrete, increasing with depth. These concrete pore-water pressures can be much higher than the water pressures in the surrounding ground, theoretically as high as the total concrete stress. A hydraulic gradient develops and this leads to water flow out of the concrete, if the surrounding soil is permeable. The effect of this water loss is to stiffen the concrete (EFFC/DFI, 2018).

Bleeding
Bleeding of fresh concrete is a special form of segregation that occurs once the concrete has come to rest and can best be descripted as some cement water flowing out of the pile head in general along the reinforcement bars. Differences in specific gravity of the concrete constituents result in high water pressures in the fresh concrete which exceed the hydrostatic water pressures. This leads to a vertical hydraulic gradient which tends to make the water in the cement paste flow vertically towards the concrete surface. In general this bleeding doesn’t cause real damage to the pile shaft.

Whilst bleeding is a fundamental concrete characteristic, it is bleeding under very high concrete pressure heads that is of most relevance to tremie concretes. This results in large water pressures in the concrete, which are significantly greater than the hydrostatic water pressure. Preferential water flow pathways can also develop in concrete, often varying in size and frequency, depending on various parameters. When bleed tests are considered necessary as part of the suitability testing both bleed and filtration (under pressure) should be tested (EFFC/DFI, 2018).

Segregation
Relatively dense and large aggregate particles will tend to sink through the lighter cement paste due to gravitational differences. This leads to a gradation of materials depending on the yield strength of the concrete. Therefore a sufficient high yield stress is necessary to overcome the tendency of segregation.

3. Problem Overview

After introducing the different pile categories in Chapter 2, this chapter will continue with the problems that might occur during or after the installation of the piles. An overview, which is the result of a data analysis of problems that have occurred in construction projects and the possible causes that might have resulted in the problems, will be reviewed to clearly show the possible mechanisms and influencing factors. Every problem starts with a short description of the type of problem and how this is specified. Then the mechanism with their potential causes will be discussed. And finally a short overview of important aspect is given.
There are several conditions that influence the possibility of a problem, these can be divided in three categories (1) Soil conditions, (2) Execution and (3) Concrete stability. It is not always evident to assign a problem to one of these categories and therefore all of the possible mechanisms and underlying causes are discussed taking into account these categories.
It must be noted that the given causes and mechanisms are possibilities which have been found from project documents and literature, whether a problem might occur will be project depended and will be influenced by several factors, as is seen in current practice. However, the objective is to find relations between problems and their causes and to give advice about certain situations which might trigger the occurrence of problems.

3.1 Necking

First of all a short problem description will be given. Seven project cases were collected in which necking was found. Then the different mechanisms that were assigned to cause this problem are discussed and it is concluded with some indications or troublesome soil profiles to be aware of when considering necking.

Problem Description

Necking is the problem of a reduced pile diameter. It can occur in varying degrees of severity, from a minimal difference in pile diameter up to exposed reinforcement bars or a concrete front even behind the rebars. The degree of severity will influence the structural integrity of the final pile. Whether necking will lead to severe remediation or even rejection of the pile will depend on the size of the final diameter, the depth of occurrence beneath the final pile head, the concrete cover and specifically the exposure of the reinforcement bars. Deviations in the pile diameter can be checked with pile integrity tests. A commonly applied test in the Netherlands is the low strain integrity test (ASTM D5882-16), but with this test small and gradual deviation of the pile diameter are hard to detect.

Possible Causes of Necking

The problem of necking can be caused by different mechanisms. These will be discussed separately taken into account the different influencing factors.

Mechanism 1: Sagging

The first mechanism is due to a disturbed equilibrium between the fresh concrete and surrounding soft soils. When the lateral pressure of the concrete exceeds the passive resistance of the soil, the pile diameter increases which is called bulging. Very soft soils exhibiting undrained shear strengths less than 15 or 20 kPa are susceptible to this mechanism (NEN-EN 12699, 2015 and Fleming, Weltman, Randolph, & Elson, 2008). Reference value following from cone penetration tests (CPT) below 0.5 MPa for soft soils susceptible to this mechanisms was suggested by W. Nohl (personal communication, April 18, 2019) based on general practice.

When a permeable fill overlays such a soft soil layer than a plug of hardened concrete can be formed above the soft soil layer, because the excessive water in the concrete can dissipate more easily into the permeable fill. Additionally, the concrete underneath this plug can be in more fluid state because the soft soil layers are less permeable hence excessive water in the concrete cannot dissipate radially. Assuming that the soil layer is very soft a bulge might be formed by the lateral pressure of the concrete. Subsequently, if the hardened plug stay at its position, the fluid unset concrete will sag into the bulge and a neck will be formed between the plug and the bulge. This is illustrated in Figure 3.1.

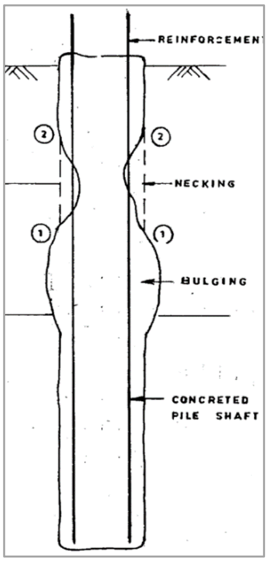


Figure 3.1: Example Necking and Bulging (From Karandikar, 2018)

This mechanism of necking might be triggered when the soil profile contains a very soft soil layer underneath a permeable fill (which is a regular soil profile in parts of the Netherlands, e.g. near Amsterdam). And it is probably restricted to shallow depths while the passive resistance and stiffness increases and the compressibility of the soil layers decreases with depth.

This mechanism of necking is also described by Hobbs (1957) where, additionally, the influence of artesian conditions on the setting of the freshly placed concrete was addressed. In CUR recommendation 114 (CUR Bouw & Infra, 2009) this problem and cause are also defined but adds the extreme situation in which the concrete keeps sagging after it has been placed. Obvious subsidence of the pile head and the need for excessive amounts of concrete are indications of extreme bulging either by displacing the soft soil or due to (unforeseen) cavities in the soil stratum. Although bulging is seen as a pile defect this might not negatively influence the pile ultimate capacity (El Wakil & Kassim, 2010).

Mechanism 2: Elastic Rebound of Soft Soils
The second soil related cause and the one that has been referred to most often in the project data is the elastic rebound or undrained response of soft soils due to the soil displacement following from pile installation.
The installation of a soil displacement pile system causes volumetric expansion and as a consequence the surrounding soil is compressed. In granular soils the soil grains will redistribute resulting in compaction or dilation, depending on its relative density. In cohesive soils, the soil response might be undrained and the volume tends to be constant, this means that the soil wants to return to its original position. In specific cases, these soils exhibit a high degree of elastic soil behaviour and after the casing is withdrawn this can result in necking of the fresh concreted pile. This has been noted in several projects where multiple piles were necked at specific depths where clay, silt or peat layers where present. CUR recommendation 114 (CUR Bouw & Infra, 2009) notes this mechanism as potential cause in specific situations where special clay- and silt layers are present. This is a bit arbitrary and more quantitative soil descriptions would better indicate which soils are critical. In the NEN-EN 12699 (2015) it is described that the distance between cast-in-situ piles that have not reached sufficient strength should be increased from six up to 10 times the pile diameter if a soil layer has an undrained shear strength of less than 50 kPa as the influence of volume displacement might also harm previously installed piles.
This mechanism can be triggered at sites that are closed off by retaining walls, when a high amount of displacement piles need to be installed or when a top layer of sand is present that affects the possibility of heave. On the other hand, the confinement might be reduced by pre-boring with an auger. It is notable from the provided data that this mechanism occurred more frequently in project where displacement pile systems with enlarged pile tips (e.g. Olivier, Atlas or DPA) were installed. The enlarged pile tips have a larger diameter than the casing, hence the soil is displaced during screwing in as well as during withdrawal. After the casing is withdrawn the concrete will be in contact with the surrounding soil, so the additional displacement during withdrawal might amplify the potential of necking.

Mechanism 3: Underdeveloped Flanges

Another type of necking, which has been noted for pile systems that create flanges, is the underdevelopment of these flanges. This can be seen as a form of pile diameter reduction while these flanges might be taken into account in the bearing capacity calculation. In one of the project cases, the flanges are not constructed in the upper four meters of soft soil (clay/peat). The underdevelopment of the flanges was related to a decreased concrete quality at the pile head due to some water flow that had been noted. The flanges did nicely develop in the sandy layer underneath. The opposite has been found in a test campaign in Limelette, Belgium. Within this large research program some piles have been extracted and underdevelopment of the flanges in a sand layer was noted for Atlas screw piles as well as for Olivier piles. The cause has been related to a much slower advancement per rotation than the theoretical pitch of the screw auger, thus disturbing the soil. In addition, Maertens & Huybrechts (2003) state that both systems exert a downward force on the disturbed soil underneath its blades during the withdrawal process and for the Olivier system the tooth at the end of the auger might additionally lead to a plough effect. The combination of these causes might have resulted in the underdevelopment of the flanges. These effects are illustrated in Figure 3.2 and Figure 3.3 for an Atlas pile and Olivier pile, respectively.

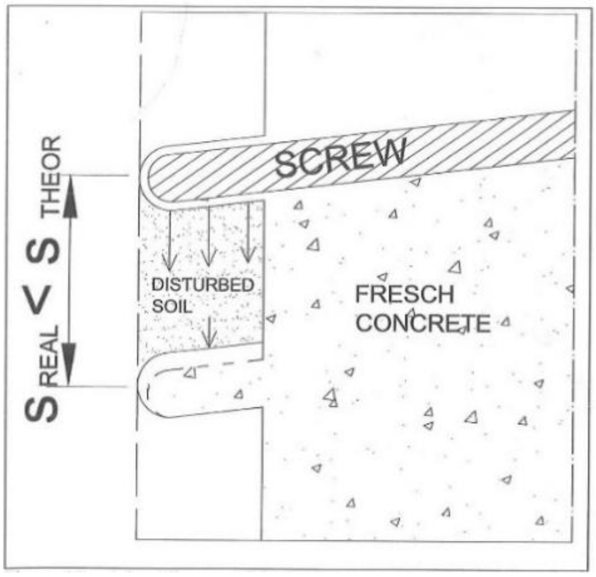


Figure 3.2: Atlas Pile: Downward force exerted on plastic concrete flange by screw blades (from Maertens & Huybrechts, 2003)

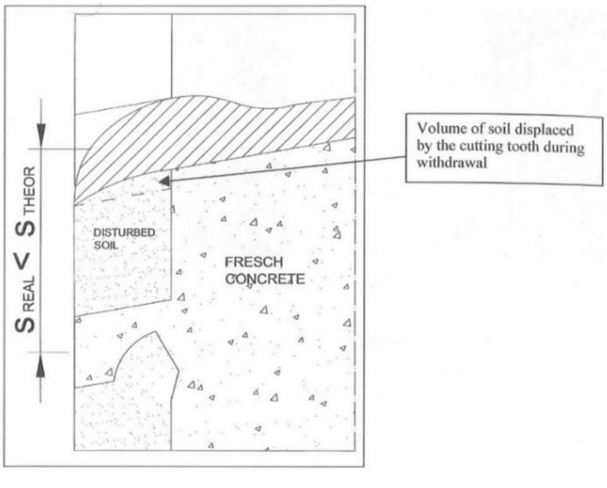


Figure 3.3: Olivier Pile: Possible principle of the occurrence of the thin connection of screw flanges to the pile body (from Maertens & Huybrechts, 2003)

Mechanism 4: Concrete Flow

This mechanism is related to the flow of concrete during withdrawal of the temporary casing or auger stem. Depending on the pile system the concrete will start to flow under its own pressure or it is put under additional pressure. Independently from the pile system, the rate or velocity at which the casing or auger stem is withdrawn should be in accordance with the flow of the concrete. If the casing is extracted too quickly or the auger is pulled too fast with respect to the ability of the pump to deliver volume, the soil will tend to collapse inward and form a neck (CUR Bouw & Infra, 2009). The flow of concrete is also dependent on its rheological properties and the amount of reinforcement bars. The potential for the occurrence of necking will increase at shallow depth during withdrawal since the concrete pressures are low. This will be influenced by the total amount of concrete and the additional height of the concrete head that is taken into account for soil displacement pile systems.

Another limiting factor is the reinforcement cage which restricts the concrete to flow out. With the current trend to construct higher buildings and accordingly, potentially higher reinforcement percentages, the design considerations that impact the concrete flow are very important. During structural design the reinforcement spacing and concrete cover are designed based on the ability of the concrete to flow through the horizontal and vertical reinforcement bars. Necking immediately influences the concrete cover therefore the ability of the concrete to fill the designed pile shaft, taking into account the reinforcement cage and rate of withdrawal, will be very important. This is also determined by the mixture of the concrete defined by its workability.

Mechanism 5: Incorrect Concrete Mixture

The workability of the concrete is the general term defining the ability of the concrete to fill the excavation, self-levelling and self-compacting under gravity (EFFC/DFI, 2018). It is one of the rheological characteristics and therefore depends on the rheology of the concrete. Successful placing of concrete requires a low viscosity as this affects its distribution within the excavation. Several test methods exist to control this behaviour (EFFC/DFI, 2018).

Other possible causes:

The abovementioned mechanisms were the main causes found in the project descriptions and literature. Other influencing factors have been found but these were never appointed to be the cause and therefore are listed below.

- Due to a wrong installation sequence, the equipment has to maneuver around the just installed piles and by its weight might harm the piles when concrete still is in the plastic stage.(CUR Bouw & Infra, 2009)
- Too little distance between consecutive piles.
- Too little use of concrete (can easily be checked by pumped or delivered volume of concrete).
- Casing getting hot due to shearing and consequently concrete getting stuck in the casing during withdrawal.
- Some remnants of concrete on the casing during withdrawal.
- Liquefaction of silt layer plus additional load of machinery.
- Obstructions like boulders, old foundation or unexploded ordnance (UXO).

Overview

To conclude it can be noted that if the soil stratigraphy shows (very) soft soil layers (peat, silt or clay) at shallow depths the risk of necking is present. This might be amplified by the presence of a permeable fill on top of the soft soil layers.

That necking will occur at shallow depths is mainly argued by two situations. Firstly, because necking is often found during excavation around a pile making the deviation in pile diameter visible and the excavation is often limited to a few meters. Secondly, the concrete pressure is low at shallow depths and thus easier to overcome by the soil. In general the volume of concrete should at least equal the pile volume but slightly more concrete volume can also be used to be sure to create the minimal diameter. It was also found that the outflow of concrete during withdrawal is very important and thus should be controlled by enough concrete pressure, a correct rate of withdrawal and a suitable concrete mix design incorporating pressure losses from reinforcement and other obstacles. The different mechanisms and causes showed that necking might arise due to a combination of soil conditions, installation procedure and concrete properties and that it is not always possible to predict a certain mechanism beforehand.

3.2 Erosion

The second problem which will be discussed is erosion. Although no projects were found that were referred to as erosion, some of the causes that were found for necking can better be assigned to erosion. Therefore, is it suggested to distinguish between these problems.

Problem Description

A problem, which has an equivalent effect as necking, is erosion or erosive flow ('uitspoeling'). Both problems influence the diameter or quality of the pile shaft but the main difference is the underlying mechanism, which is waterflow in the case of erosion. The following causes might initiate erosion of the piles.

Possible Causes of Erosion

The problem of erosion can be caused by different mechanisms. These will be discussed separately taken into account the different influencing factors.

Mechanism 1: High Groundwater Velocity

The first mechanism is a high groundwater velocity in coarse soil layers, that can occur in soils with large pores (high permeability) like gravel or very coarse sands. NEN-EN 12699 (2015) states that particular attention shall be paid to underground strata where the groundwater velocity is high. Soils susceptible to high groundwater velocities can be easily recognized with CPT and borehole data. High ground-water velocities are not very common in typical Dutch soils but in areas with large pressure head difference or large gradients a 'relatively' high groundwater velocity might be present.

Mechanism 2: Artesian Conditions

Another mechanism that can cause erosion is a hydraulic head difference between two soil layers (aquifers) that are separated by an impermeable layer (aquitard), this is called an artesian aquifer. In the original situation the impermeable layer will prevent the connection between two soil layers with different hydraulic heads but due to a pile penetrating through this layer a connection is bored and a water flow between the layer with the higher hydraulic head towards the soil layer with the lower hydraulic head can arise. If the impermeable layer is thick and/or the head difference is low, the flow might be very little and not harmful for the fresh concrete pile. But when the head difference over a thin impermeable layer is large, or gets large due to drainage, the water flow might erode the pile surface at that specific depth. As mentioned, drainage in a certain aquifer might increase the head difference and should therefore be taken into account when a design is made for dewatering of the project site. Project areas near the coast might suffer tidal influence if an aquifer is connected with a sea or lake. Due to tidal influences the head difference might fluctuate.

The problem of erosion might also arise during the installation of non-displacement piles with auger through layered soil. In the situation where a permeable soil is transported upwards by the auger and at the depth of an impermeable soil layer makes a connection for the ground water to flow between two aquifers (CUR Bouw & Infra, 2009). The concrete pressure should be large enough to fill the bored diameter and overcome the water flow. Otherwise the water flow might affect the concrete quality.

A situation that occurs quite often in construction projects is that pile installation commences from a ground level inside the building pit that is lower than the outer ground level. This is often associated with drainage and therefore increases the susceptibility to vertical ground water flow due to a higher head difference. If the excavated level inside the building pit is deep and the water level in an underlying aquifer is relatively close to the ground level outside the building pit, the head difference might become large and such that erosive flow can influence the pile quality. A combination of the two just mentioned influencing factors are schematized in Figure 3.4, but these can also occur separately.

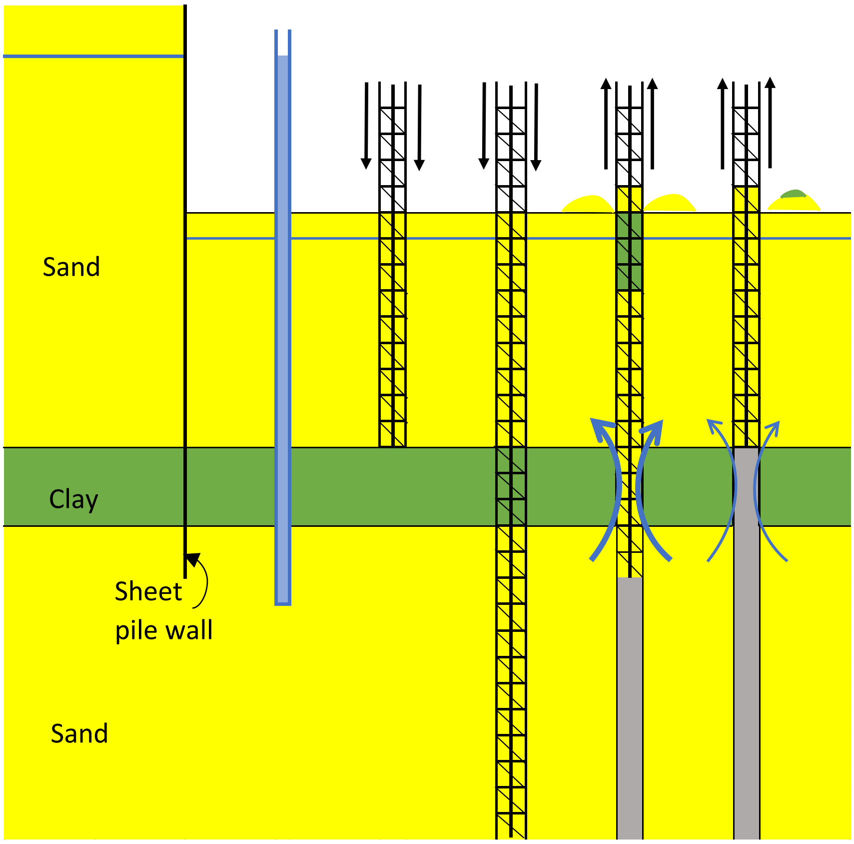


Figure 3.4: Schematization of CFA pile installed in a building pit creating an erosion connection

Especially when a pile inside the building pit is not installed up to the ground surface, to prevent the necessity of trimming the pile head afterwards, the concrete pressure is low. Hence the equilibrium between the concrete and water pressure in underlying aquifers might be critical. A practical solution to increase the concrete pressure is to fill the remaining height with water, but then it must be assured that the water cannot dissipate into the surrounding soil.

Overview

From the two mechanisms addressed, artesian conditions can be expected to be more pronounced in the Netherlands. Whether artesian conditions are present in the project area can be investigated with CPTs with pore water pressure measurements or piezometers. Additionally, influencing factors such as the tide, drainage and construction inside a building pit should be considered when installation of cast-in-situ concrete piles is chosen. Especially non-displacement pile systems with auger have an extra possibility to cause erosion as they loosen the excavated soil and transport it up through non-permeable layers.

3.3 Soil Relaxation

The third problem is mainly related to pile systems with an auger system, thus in the following either Omega- or DPA-systems (at the bottom part) or non-displacement pile systems, with auger will be meant. These systems remove the soil and if the hole that is created is not completely filled by the concrete or too much soil is removed, soil relaxation might occur. This problem was found in five projects.

Problem Description

Soil relaxation is the loosening of surrounding soil during pile installation. This can have adverse effects on the bearing capacity of the piles and therefore important to prevent.

Possible Causes of Soil Relaxation

The problem of soil relaxation can occur around the shaft or at the pile base and can be caused by different mechanisms. These will be discussed separately taken into account the different influencing factors.

Mechanism 1: Over-Flighting

Soil relaxation might be caused by over-flighting if the auger is over-rotated and excessive soil removed. During excavation the advance and rate of rotation of the auger shall be in accordance with the soil conditions so that soil removal is limited. If the auger penetrates slowly and gets insufficient base feed to keep the auger flights full, the auger feeds from the side with attendant decompression of the surrounding soil as illustrated in Figure 3.5. Consequently, the soil volume can increase resulting in less contact between the soil grains hence a lower stress state. This can eventually decrease the shaft or base capacity. Therefore, it is very important to choose a drilling machine with sufficient power that can provide the boring tool with the needed torque and pull-down force if hard stratum will be encountered.

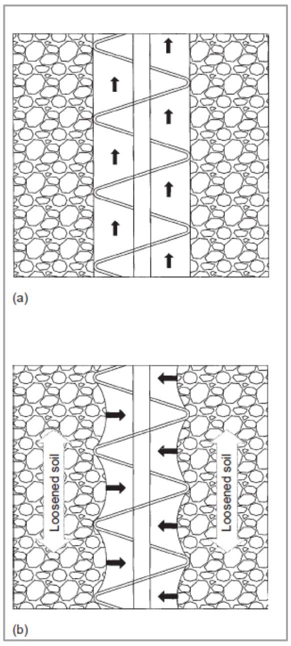


Figure 3.5: Diagrammatical representation of (a) normal CFA auger-piling and (b) the effect of 'over-flighting' (From Fleming et al., 2008)

Soil relaxation can occur around the shaft of the pile or at its base. These will be separately discussed while different causes exist that underlie this mechanism.

Firstly, soil relaxation around the shaft will be reviewed, this will only occur for non-displacement piles installed with an auger over the complete depth of installation like the Continuous Flight Auger (CFA). As already mentioned, the advance speed of rotation of the auger shall be in accordance with the soil conditions to prevent that the auger feeds from the side. However, the penetration rate will also depend on the soil that is encountered during installation. Especially in highly variable ground conditions the correct alteration might change over depth. For example, when a sand layer with a high cone resistance is overlain by soft soil or loose granular soil, the penetration will be slowed and the overburden soil might be flighted by side loading. Also, the degree in which old foundation piles (installed with soil displacement) have compacted the project location should be taken into account, while this can influence the advancement of the auger and therefore increase the potential of over-excavation. Especially when the new piles are installed in close proximity of old foundation piles some attention on soil relaxation is important. As already said the ability of the drilling machine to withstand the resistances is therefore very important.

Secondly, soil relaxation around the base might occur. Although some relaxation might be expected and is incorporated in the pile bearing calculation factors this is disadvantageous for the end bearing capacity of the pile and should therefore be prevented. Besides the previously mentioned CFA piles also the pile systems with partial displacement (Omega-or DPA pile) use an auger system at the bottom part, removing soil upwards. For these systems soil relaxation is only possible at the base of the pile, while the displacement

body above it will undo the loosening by displacing the soil radial outwards again. Therefore, special attention should be paid to the advancement and speed of rotation around the pile base level, while slow excavation relative to the number of rotation would transport excessive soil from the base to the displacement body leading to a decrease in the base resistance.
Most of the foundation piles in the Netherlands get their bearing capacity from a combination of end bearing capacity and shaft resistance. Thus the effect of soil relaxation at the base of the pile should also be limited where possible. This starts in the design of the pile plan where the influence distance of piles should be considered and piles located in close proximity should be designed to the same pile base level to limit the installation effects on a neighboring pile.

Mechanism 2: Dislodge pile cap
When the pile reaches the desired depth, before the concrete placing commences the cap at the pile head should be dislodged. This is done by lifting the auger a few centimeters, thus leaving room for the cap to be dislodged by the concrete pressure inside the hollow stem of the auger. Care must be exercised so that the auger is not lifted too rapidly in the initial stages and the pressure exerted pushes the concrete out to fill the shaft. Failure to do this may result in significant loss of end-bearing or possible segregation of the concrete in water-bearing ground (Fleming et al., 2008).

Overview
Over-excavation can be noticed during construction when the amount of soil that is removed is much larger than the total volume of the pile, although this is hard to see. It can also be noted by an increase in concrete volume, but this could also be the result of bulging or filling of soil cavities. On the other hand, at least the minimal required concrete would be beneficial to ensure that some of the over-flighting is counterbalanced and no additional relaxation occurs due to limited use of concrete. If the concrete pressure is constantly monitored, it can give information about the amount of concrete that is used and can give valuable information at the commence of concrete while a drop of pressure should be noticeable when the cap is dislodged. Specific soil conditions where an increased risk of over-flighting is present are (1) soft cohesive and cohesionless soil overlying form to hard strata and (2) soft clays, loose silts and sandy silts. Next to the soil conditions it may be clear that this problem will largely depend on the execution of the pile installation in combination with the right equipment.
Finally, when indications of over-excavation exist, CPTs can be performed after the pile is set. Preferably three CPTs around the pile will be performed to be able to determine the effect of the pile installation on the surrounding soil. If these are compared to CPT results before installation, soil relaxation will be verified by a reduction in the cone resistance. It should be noted that excavation of the building pit must be taken into account when these results are compared. With this information the diminished bearing capacity can be calculated.

3.4 Not Reaching Depth

The fourth problem that will be discussed is not reaching the designed depth. This problem was found three times in the collected problem cases.

Problem Description
The name of this problem is self-explanatory and does cope with longer installation times and a reduction in the bearing capacity of the pile if the bearing stratum is not reached.

Possible Causes of 'Not Reaching Depth'
The problem of not reaching the designed depth can be described by one mechanism but does depend on multiple influencing factors which will be discussed here.

Mechanism 1: Penetration Restrictions
Foundation piles are used to support the above structure and gain their bearing capacity from the surrounding soil. The piles will carry the load from the shaft resistance, the pile base resistance or a combination of both. In the Netherlands most of the piles are designed to reach the deeper sand layers to be able to resist the demanded pile load. If somehow the prescribed depth is not reached, the bearing capacity of the pile will be lower and thus a new configuration should be found. Depending on the achieved depth a redistribution of forces is sufficient otherwise additional piles are needed.

Non-displacement pile systems with auger will have little problems with reaching the required depth when a drilling machine with sufficient capacity is used. Because this system excavates the soil the resistance is limited compared to soil displacement pile systems where the surrounding soil is displaced and compacted resulting in high shear resistance subsequently influencing the penetrability.
The driving resistance of the soil depends on the soil stratigraphy. From geotechnical investigation at the site together with previous comparable experience (if present) a suitable hammer or boring tool should be chosen that allows penetration to the prescribed depth. Dense sand layers and large pile lengths have an adverse effect on the possibility to reach the required depth. Another aspect of the geotechnical investigation is to determine if a foundation of an old building, boulders or other obstructions are present. The presence of boulders or obstructions can cause difficulties during driving or screwing and therefore particular attention during geotechnical investigation shall be paid to their presence (NEN-EN 12699, 2015).
When projects are constructed in urban areas old foundations might be present. Most often, the old foundation does not meet the new requirements for the bearing capacity, hence new foundation piles should be placed in between the old foundation or the old foundation piles should be pulled out which is inconvenient. The effects of the old foundation should be taken into account when designing the new piling plan. Especially when the new foundation piles are designed relatively close to the old piles. When these were soil displacement piles, the surrounding soil might be compacted and a higher number of blows or larger torque and pressure must be expected. The effect of compaction can also be found in projects where a considerable number of piles will be installed in a small area sealed by sheet piles. The sheet piles limit the possibility of horizontal soil displacement which in turn leads to more soil compaction in the project area subsequently making the penetration more difficult .
When hard soil layers are present or large depths are required, grout injection can be used at 'Fundex' pile systems. The grout will loosen the soil at the pile base, transport the excavated soil upwards and decrease the shear resistance around the casing which enhances the pile penetration. But if the return flow of grout stops because of cavities, other flow paths or when the W/C factor of the grout becomes too low the energy necessary to penetrate the soil will increase again leading to a resistance in penetration and potentially not reaching the prescribed depth.

Overview
The above described causes will not always result in not reaching the designed depth but can also result in longer drilling times only. This might negatively affect the project planning and piling costs. When the necessary pile length is large or very dense sand layers are encountered it is important to choose a drilling machine with sufficient capacity. In these situations a non-displacement pile system will be beneficial compared to a soil displacement pile system considering this problem. On the other hand, a soil displacement pile system that can be executed with grout-injection can also reach considerable depths if the return flow is assured. Besides the previously mentioned soil conditions it is important to do comprehensive site investigation if obstacles can be expected. Especially, when new piles are installed in the vicinity of an old foundation some soil compaction can be expected and subsequently the advancement will be more difficult.

3.5 Cracking

Cracking in cast-in-situ piles will most certainly be caused by execution errors because the concrete can only crack after the concrete is cast, set and at least to some extent hardened. While fresh concrete in the early stages will not crack but deform. Although the cracks cannot visually be determined because of the construction underground, the cracks can be detected by low strain integrity testing. Cracking was only found in one of the project cases, the possible causes will be discussed further.

Problem Description

Cracking can occur in cast-in-situ concrete piles due to lateral or tensile loads before the concrete has reached a suitable strength. Cracks are suspected to be located under the reinforcement, especially when this is limited to the upper part of the pile.

Possible Causes of Cracking

The problem of cracking can occur at different depths and can be caused by different mechanisms. These will be discussed separately taken into account the different influencing factors.

Mechanism 1: Lateral Loading

Cast-in-situ piles can be damaged by driving or screwing adjacent piles too close or before the concrete has reached a suitable strength. To lessen the risk of cracking caused by soil movements, a minimum spacing is often employed. Fleming et al. (2008) states a minimum spacing of 5D center to center distance if the concrete is less than 7 days old, whereas CUR Recommendation 114 (CUR Bouw & Infra, 2009) states that a minimum distance of 4 times the diameter must be taken into account between consecutive piles installed within 15 hours. Other sources that can cause a lateral load on the pile are related to movements of heavy equipment resulting in one-sided loading or pile trimming exerting bending moments on the piles. Also excavating around a pile, when incorrectly executed, can cause lateral loads that eventually result in cracks.

Mechanism 2: Vertical Loading

Piles can also be damaged by tensile forces, as the ground heaves. This should be taken into account in clayey soils when piles will be excavated to a lower cut-off level. Installation of a soil displacement pile displaces the soil vertically and radially. The ensuing vertical soil movement can cause the uplift or heave of piles already installed in the vicinity, this might also cause cracks to form. Again, this effect will depend on the radial distance between the installed piles.

Overview

In general cracks can easily be detected by low strain integrity testing, but can still be disadvantageous for the bearing capacity of the pile. This will fully depend on the width of the crack as some minor cracks may be considered acceptable. The effect of the cracks on the bearing capacity can be determined from pile load tests, but this comes with high costs. Therefore, it is better to take the lateral and vertical loads during and after the construction of a pile into account in the design of the pile shafts and reinforcement. The loads from construction can come from excavation of the soil, the piling machine and trimming of the pile head. When soft soils are encountered lateral soil displacement and heave can be expected, both causes forces on the pile which eventually can lead to cracks. The risk of cracking is increased when the reinforcement length is limited and not designed to the full depth.

3.6 (Excessive) Bleeding

The last problem that will be discussed is ‘excessive bleeding’. Excessive is placed in brackets to be able to distinguish between the different mechanisms that cause the water percolation from the pile head. This problem occurred in eight different projects and therewith the problem encountered most often in this problem overview.

Problem Description

First of all, it is important to distinguish between normal concrete ‘bleeding’ and ‘excessive bleeding’. Bleeding is referred to when considering the stability of the concrete itself, as explained in the literature review, and can be characterized by little outflow of water plus some air bubbles which normally stops in a short amount of time (approximately within 30 minutes). When the intensity and the amount of the water percolating from the pile head is large, i.e. a flow of water can be recognized, and/or the duration is long, than excessive bleeding is meant. Additionally, the effect of bleeding on the pile should be minimal as it is only the outflow of excess water of the concrete itself while during excessive bleeding the water flow will take some fines with it resulting in a channel after the concrete is hardened. The latter can harm the integrity and durability of the concrete pile and is therefore seen as a problem.

The occurrence of excessive bleeding will only be visible when the water flow reaches the pile head or when a channel is encountered below the pile cut-off level. But even when an outflow of water at the pile head is observed, there is no recommendation on the information that should be gathered and hence the only information from the report of the supervisor is an indication that a pile observed ‘bleeding’. For example, the CUR recommendation 114 (CUR Bouw & Infra, 2009) only states that it is the task of the supervisor to note and report ‘bleeding’ such that measures can be taken. This is very indistinct but will probably refer to an unexpected large outflow of water, thus indicating excessive bleeding. The randomness of occurrence inside a project is one of the interesting things about this problem, but also makes it hard to find the root cause of it. The final assessment is sometimes difficult to obtain because the full range and severity of defects cannot be entirely assessed by low strain integrity testing or by using core drilling, as the core sections always represent a very small and limited cross-sectional area of the affected element.

Possible Causes of (Excessive) Bleeding

At least the following can be stated; the outflowing water must come from either the concrete itself or from ground water in the surrounding soil. Therefore, the causes of the problem are divided in two mechanisms which will be discussed separately.

Mechanism 1: Excess Pore Water Pressure in surrounding Soil

The cause that have been referred to most often is the generation of excess pore water pressures in confined sandy layers at relatively shallow depth. Especially in the west part of The Netherlands the Holocene layers are strongly variable with alternately sand and clay layers. These layers can be found from CPTs that are executed at a project site. The generation of excess pore pressures during installation can be explained by a cavity expansion theory or a geohydrological model as explained in the literature review. The excess pore water pressures that are generated in cohesive soils with low permeability will dissipate very slowly and are therefore not expected to support the water flow that occurs during excessive bleeding. But in non-cohesive soils which are confined by aquitards excess pore water pressures can also be generated and have the tendency to dissipate quicker. This will normally flow outwards in radial direction but if a connection is found through the plastic concrete it may add to the water flow, which is supported by some videos that show a very strong flow of water.

The excess pore pressures in the soil can be caused by sheari, volume displacement or compaction resulting from the displacements following from the pile installation. Additionally, piles driven with an impact hammer exert vibrations which can also enhance compaction. The amount of excess pore water pressure will depend on the installation method, pile diameter and soil conditions.

Mechanism 2: Concrete Stability

As introduced in the literature review bleeding as well as filtration induce water to flow out of the concrete. The hydraulic gradient developed outwards by the filtration mechanism can flow out in permeable soil strata. But sometimes and especially in cases where the pile extends over a large length in non-permeable layers, the excess water is not able to flow out and might reach the pile head and exit at that location. Note that this is not a necessity as it will depend on the stability of the concrete mix, otherwise it should always be observed at cast-in-situ piles with lost casing for example. Another aspect which might hinder the excess water to flow out of the concrete is the use of grout-injection to enhance the penetration, this will plaster the surrounding soil leading to a thin layer with a low permeability.

There are also external sources of water that are not necessary for curing of the concrete and therefore increasing the instability of the concrete if added. Possible sources are water at the pile base, which always needs to be checked, or water in the casting bucket before pouring of concrete. As concrete stability is one of the key rheological characteristics for fresh concrete it depends on the mix design and several test methods exists to test the water retention ability of the fresh concrete (EFFC/DFI, 2018).

Other possible causes:

The abovementioned mechanism can be seen as direct causes regarding the pile that is installed. In the project documents it was also found that during the installation of a pile water came out of a neighboring pile. This has been appointed to the following cause, but no literature was found that substantiates this mechanism:

- Fracturing occurring from grout-injection hence influencing a fresh neighboring pile.

Overview

A clear distinction was made in the source of the water between mechanism 1 and mechanism 2. Although all of the problems in the overview were assigned to mechanism 1, no conclusive description was found that explains how the pore water pressures enters the fresh concrete and why it looks to occur randomly inside a project. As was found in the literature, it can be stated that soil displacement pile systems do cause excess pore water pressures in the surrounding soil. And fine (silty) sand layers might be prone to generation of excess pore water pressures, especially when they are confined. Regarding mechanism 1, it was stated that the risk of bleeding is more pronounced in the upper part of the piles where the concrete pressure is lowest. In addition, it was concluded that cohesive soil could not trigger this mechanism as the dissipation of the excess pore water pressures will be too slow. But these layers might hinder the filtration tendency of the concrete itself as the excess water inside the concrete cannot flow outwards and therefore should find another way, up. To control the bleeding and filtration characteristics of the concrete, several test methods are available.

3.7 Problems: Causes and Mitigating Measures

In the problem overview, different mechanisms and underlying causes are addressed. In this paragraph this is tabulated and, where possible, the soil layers that are susceptible to a possible mechanism are given and mitigating measures are presented, see Table 1.

Table 1 Problem overview, susceptible layers and mitigating measures

Problem	Mechanism	Susceptible soil layer	Measures
Necking	1. Sagging	Very soft soil layers	Casing, concrete with higher yield stress, additional concrete head, overconsumption of volume of concrete
	2. Elastic Rebound	Soft soil layers, undrained behaviour	Higher concrete volumetric weight, additional concrete head, preferable pile system without enlarged boring head, pre-drilling, increase radial pile distances
	3. Underdeveloped Flanges		Drilling equipment with sufficient capacity, additional concrete pressure
	4. Concrete Flow		Decrease rate of withdrawal, additional pressure head, increase concrete workability
	5. Incorrect Concrete Mixture		Increase concrete workability
Erosion	1. High Groundwater Velocity	Permeable soil strata e.g. gravel , steep slopes	Casing, prefab or combi piles are preferable
	2. Artesian conditions	Layered stratigraphy	Casing, prefab or combi piles are preferable, additional concrete pressure
Soil Relaxation	1. Over-flighting	Layered stratigraphy, alternating strong and soft soil layers	Drilling equipment with sufficient capacity, soil displacement pile system, helical boring head, concrete over consumption, constant pile depth
	2. Dislodge pile cap		Careful execution
Not Reaching Depth	1. Penetration restrictions	Dense sands, stiff clays, obstructions	Soil removing pile systems are preferable, drilling equipment with sufficient capacity , grout-injection, increase W/C ratio of grout, pre-drilling, helical boring head
Cracking	1. Lateral loading		Full length reinforcement, increase time and radial pile distance, adjust machine routing, excavation
	2. Vertical loading	Soft soils susceptible to heave	Increase radial pile distance, pre-drilling,
(Excessive) Bleeding	1. Excess pore water pressures	Confined fine sandy layers	Install piles from a higher level, smaller pile diameter, vertical drains, pumping, increase radial distance and installation time between subsequent piles, pre-drilling
	2. Concrete stability		Decrease W/C ratio, increase yield stress, increase cement factor

3.8 Problem Choice: Excessive Bleeding

From the available data it was found that the amount of projects where excessive bleeding had occurred was the largest (note that this cannot be translated to an overall assessment on problems occurring in practice in the Netherlands as it is only based on project information collected at Fugro). In addition, there was one project case involving excessive bleeding where comprehensive water pressure measurements were obtained. These measurements could be used to further study the potential cause of excessive bleeding. Furthermore, as addressed in Paragraph 3.6, one of the possible causes is the generation of pore water pressures in the soil. This soil relating cause can be researched from a geotechnical point of view in which the knowledge gained during the master study can be used instead of focusing on problems where the causes relate to execution errors. Finally, the most important aspect why this problem is chosen is because the exact mechanism of excessive bleeding is unknown. And thus the specific soil conditions, installation effects or concrete characteristics that trigger this mechanism are undefined.

4. Excess Pore Water Pressure

In this chapter the generation of excess pore water pressures during and after the installation of a soil displacement pile will be reviewed. From the literature review it was found that soil displacement piles affect the pressure distribution in the surrounding soil and two analytical methods that can describe these changes are discussed. From Chapter 3 it was concluded that the cause of 'excessive bleeding' was either due to concrete instability or due to excess pore water pressure in an intermediate sand layer at shallow depth. This latter cause will be researched in this chapter with help of project data in which excessive bleeding was observed and high excess pore water pressures were measured. First the available data will be studied, following from data the possible causes of the generation of excess pore water pressures will be reviewed. Where possible some models are tested that substantiate the potential causes. Then, the water pressure after the casing is withdrawn will be compared to the concrete pressure. Finally the influencing factors relating to soil conditions and installation process will be discussed and the possibility to limit the excess pore water pressure are presented in terms of mitigating measures.

4.1 Data Analysis

First of all the available data will be reviewed. Starting with the water pressure measurements. These are presented in hydraulic head versus time. Analyzing the measurements focusses on the generation and dissipation of the excess pore water pressures but also on the possible mechanisms that cause these. Some focus on the measurement devices and the suitability of these are discussed. And finally the measurements are compared to the crane registrations to see if a causality can be found.

4.1.1 Measurement Data

Two test fields were set up to monitor the generation of the excess pore water pressures in the intermediate sand layer to be able to measure the influence of the current installation process and the effects of possible measures. In the first test field all processes were kept the same as previous installed piles to research the excess pore water pressure that might have caused the observed bleeding and washout of the concrete. In the second test field some measures were taken to see what their influence was on the generation of the excess pore water pressure and subsequent influence on the bleeding phenomenon, potentially decreasing the occurrence. The measures that were taken included an additional fill of half a meter and installation of eight vertical plastic drains around the piles.

The soil stratigraphy can be described as a general soil profile near Amsterdam starting with a manmade fill, than the Holocene layers consisting of soft clay with thin bands of sand ('wadzand') which is sealed off by a layer of peat ('basisveen') and followed by the Pleistocene sand layers in which these concrete pile were founded. A characteristic CPT profile from the project area can be seen in **Error! Reference source not found.**, this is also tabulated in Table 2 where the intermediate sand layer is assumed to form one layer of sand between NAP -7 to -8m. To support this assumptions, the measurement devices were installed in between this depth and thus the measurements of the hydraulic head will represent this layer. The ground level during construction can be assumed constant at NAP -4m and the design water level was 0.5m below ground level.

The foundation piles were screwed full-displacement piles with lost tip, referred to as Fundex piles, installed with grout-injection. The length of the piles was approximately 15.5m and the pile tip and casing diameter are 0.65m and 0.53m, respectively.

Table 2 Typical soil stratigraphy

Soil type	From [NAP +m]	To [NAP +m]	Thickness [m]
Sand	-4	-6	2
Clay	-6	-7	1
Sand ('Wadzand')	-7	-8	1
Clay	-8	-11	3
Peat	-11	-11.5	0.5
Sand	-11.5	-30 (final explored depth)	≥18.5

Figure 4.1 shows a general representation of the hydraulic head measured in the intermediate sand layer during the complete installation process of a pile in this project at different radial distances. It is clearly seen that quickly after the start of installation a large increase in the pore water pressure in the intermediate sand layer is generated. This peak quickly dissipates and keeps decreasing during the descending phase of the pile installation. During the placing of reinforcement and pouring concrete the water pressure kept decreasing until the moment the machine started to withdraw the temporary casing. Immediately an (small) increase in hydraulic head is visible near the pile. This increase adds up to the excess pore water pressure that did arise in the beginning of the installation process because these were not fully dissipated yet. After this sudden increase, which was noted around all the installed piles, there is no general reaction of the water pressures. The pore water pressures started to dissipate, stayed relatively constant or kept increasing during further withdrawal. But at a certain moment, approximately at two-third of the time needed for withdrawal of the temporary casing, again a sharp increase is noted at all piles reaching up to an additional 3 meters of water column. The causes of the increases of the pore water pressures during withdrawal will be further investigated because these were not expected and the cause of it is unknown. Furthermore, this is an important aspect because during withdrawal the concrete will flow out and be in contact with the surrounding soil, so this increase in pore water pressure might affect this interaction. The described trend of the pore water pressures were found at all the piles, therefore it can be assumed that the processes that govern these increases are only due to the pile installation itself. An overview of the installed piles and the distance to the measurement devices can be found in Appendix A.2 . The measurements of the hydraulic head from all the piles in the two test fields can be found in Appendix A.3 , the increases in hydraulic head during the descending and ascending phase are clearly seen.

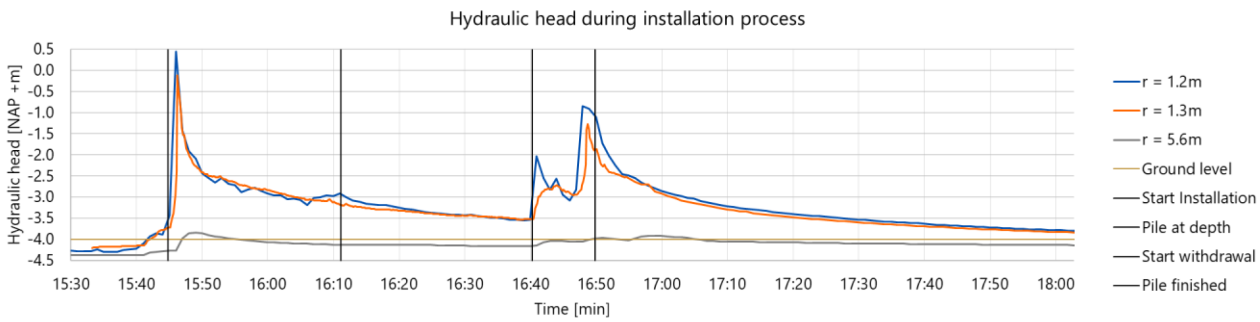


Figure 4.1: General trend of water level measurements at different radial distances during the installation process of a pile

4.1.2 Measurement Devices

The measurement devices consisted of monitoring wells (MW) ('peilbuizen') and pushed-in electric piezometers (EP) ('waterspanningsmeters') at different radial distances from the piles. Most of the measurement devices were placed in the intermediate sand layer between NAP -7 to -8m except for one electric piezometer that was installed in the deeper sand layer at NAP -12m at Test Field 2. The readings in the monitoring wells and electric piezometers were taken ones per minute. But an automatic water level logger that measured 6 times per minute was hung in the monitoring well most closely to the pile that was

installed at that moment, so this automatic water level logger was moved around the test field.
It must be noted that a monitoring well has some storativity itself and it can occur that these react slower than the electric piezometer although this was not clearly observed in the measurements. This can be explained by the fact that the radial distance between the piles and the measurement devices were mostly different, and the measurement frequency of the electric piezometers was lower than that of the monitoring wells close to the pile, this is illustrated in Figure 4.2.

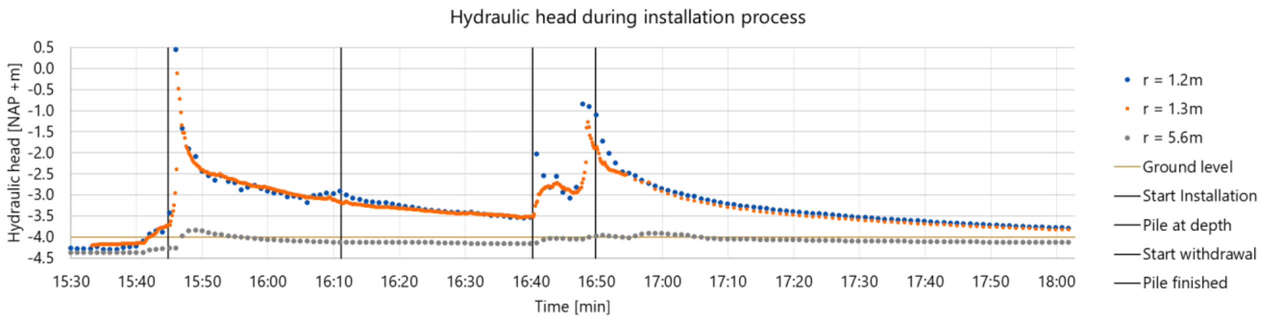


Figure 4.2: Hydraulic head over time showing the different frequency of the measurement devices

4.1.3 Crane Registrations

Furthermore, the crane registrations of the test piles were supplied by to the piling contractor. As these can help to create understanding in the installation processes over time. It can help to relate the moments that excess pore water pressures are generated to the pile depth. This comparison is illustrated in Figure 4.3. The crane registrations that were provided continuously measured the depth, torque, pull down, pull up, hydropressure, rate and RPM. All these parameters were compared to the measured hydraulic head and presented in Appendix A.4 .

The conclusions following from this comparison will be discussed further. First of all it was found that the penetration through the intermediate sand layer, i.e. the layer in which the measurement devices were installed, was in the range of 15 to 25 seconds for all the piles. This shows that the frequency of the electric piezometers and monitoring well measurements of ones per minute was not enough to be able to capture the different effects on the hydraulic head during the installation through the first couple of meters. Therefore, the analysis on the first peak of the hydraulic head is limited to the measurement data from the monitoring wells close to the piles.
Furthermore, Figure 4.3 shows that the hydraulic head starts to dissipate when the pile penetrates through the second clay layer and the rate of penetration decreases when reaching the deep sand layer. After reaching the final depth the reinforcement will be placed and the casing filled with concrete. Then the casing will be withdrawn, the start of this process can clearly be indicated by an increase in the pull up force of the machine, see Figure 4.4. As mentioned in Paragraph 4.1.1 no clear pattern can be found in the measurements of the hydraulic head in the following minutes. Although the hydraulic head seems to be influenced by the pull up force of the machine and the rate of withdrawal. Whether this combination of parameters can cause the second, larger, increase in hydraulic head is not clear from the crane registrations. For two piles, the increase of hydraulic head seems to correspond with the increased rate of withdrawal after some stagnation, but for the other piles this does not correspond to the start of the increase of the second peak. Therefore, the following paragraphs will focus on the individual increases in hydraulic head to find suitable explanations.

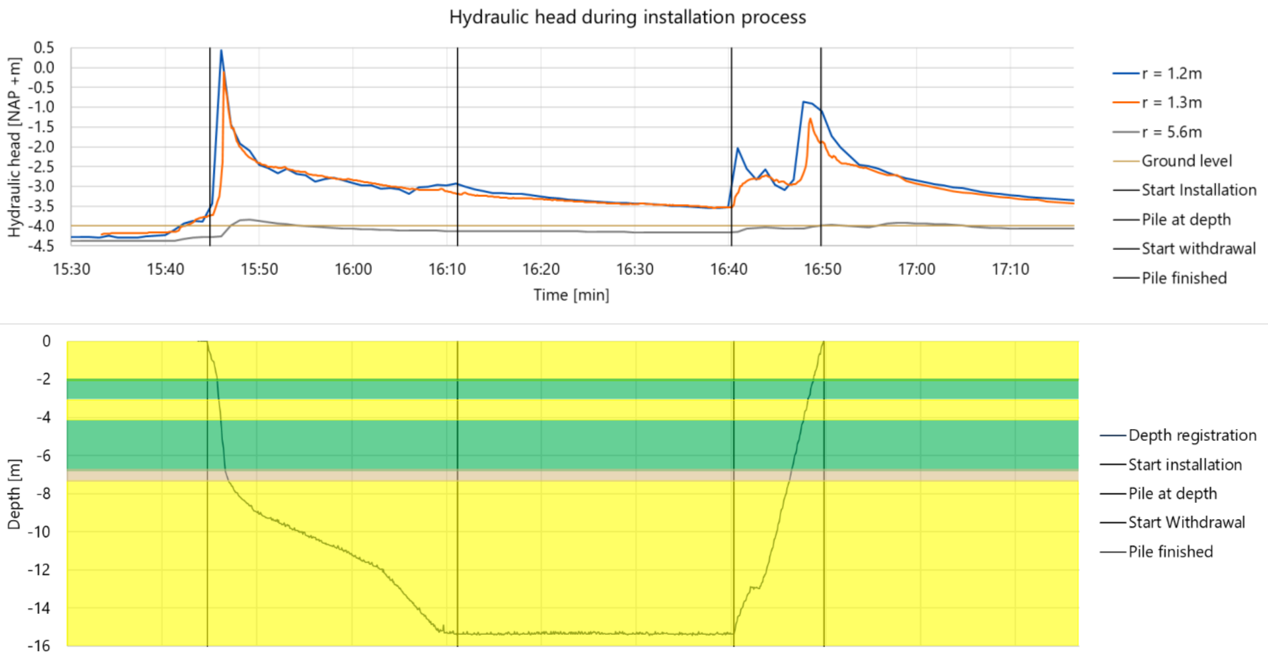


Figure 4.3: Hydraulic Head Related to Pile Depth

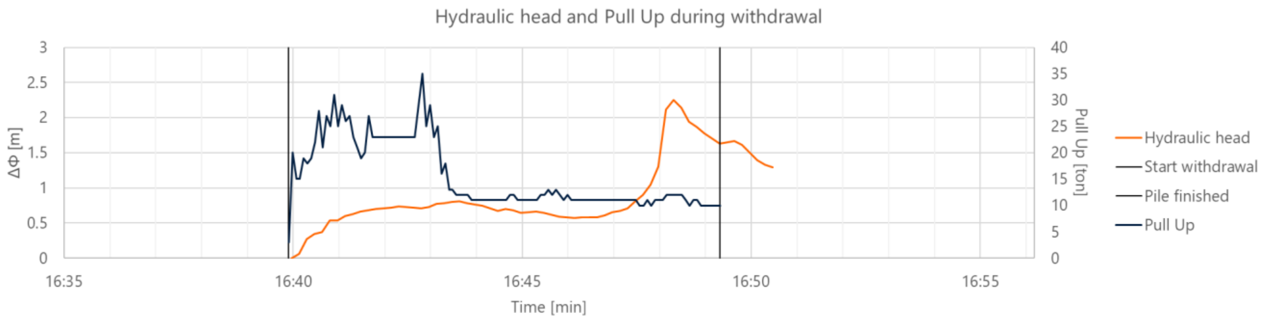


Figure 4.4: Hydraulic head and Pull Up Force during Casing Withdrawal

4.2 Pore Water Pressure Generation during Installation

As introduced in Paragraph 2.2 the installation of soil displacement piles is associated with the generation of excess pore water pressures in cohesive soils or non-cohesive soils with a high percentage of fines or confined by aquitards. In this section the generation of the excess pore water pressures following from the installation through the intermediate sand layer will be discussed based on the measurements.

4.2.1 Possible Causes

Two possible mechanisms that can cause an increasing in the hydraulic head can be suggested:

1. Shear induced pore water pressures
2. Volume displacement of water to occupy for the volume of the pile

As the piles are installed with grout-injection a layer of grout will form between the casing and the soil. This will restrict soil shearing. In addition, the measurements show a sharp increase in hydraulic head after which the excess pore water pressure quickly starts dissipating. As the sand layer would continuously be sheared as the pile progresses further, this should consequently be visible in the measurements. Instead, the excess pore water pressures dissipated while the pile penetrated to the designed depth. Therefore, the excess pore water pressure is assumed to be caused by volume displacement of the pile from the moment that the pile penetrates the intermediate sandy layer.

To be able to assign this cause it was first tested if the start of the increase of the hydraulic head in the measurements did correspond to the penetration of the sand layer. From the comparison between the depth registration of the crane and the water pressure measurements it was noticed that an increase in the hydraulic head was generated before the pile reaches the sand layer ($\Delta\Phi_1$). This can be caused by the force that the pile itself put on the ground and the grout pressure that will result in a disturbance zone under the pile. This zone of disturbance can be up to three times the pile diameter under the pile toe in soft clay (Massarsch and Wersäll, 2013). When the pile penetrates through the sand layer the hydraulic head shows a very large increase ($\Delta\Phi_2$). The different influences are tabulated in Table 3. The large increase after passing through the sand layer will be caused by volume displacement. This will be discussed further in Paragraph 4.2.2.

Table 3 Hydraulic head increase in sand layer during descending phase

Pile	r [m]	$\Delta\Phi_1$ [m] t_{start} -> t_{sand}	$\Delta\Phi_2$ [m] t_{sand} -> t_{max}	Φ_{total} [m] t_{start} -> t_{max}
319	1.3	0.77	2.85	3.62
421	1.4	0.48	4.67	5.15
252	1.1	0.54	4.46	5.00
57	1.0	0.36	5.15	5.51
246	1.1	0.26	2.92	3.18

4.2.2 Volume Displacement tested with Geohydrological Model

The CPTs illustrate that the intermediate sand layer is confined by clay layers above and beneath it, therefore restricting the water to dissipate vertically. In this paragraph it will be discussed if the generation of excess pore pressures resulting from the pile installation can be modelled by the geohydrological model explained in Paragraph 2.2.2.

First of all, the geohydrological model is based on the extraction or recharging of an aquifer. In this situation the installation of the pile can be assumed to result in a discharge into the aquifer. To convert the pile volume into a discharge the time to penetrate the sand layer is taken from the crane registrations.

The pile system used in this project is a screwed pile system with full soil displacement (Fundex) installed with grout-injection. It is assumed that the displacement resulting from the pile penetration is the average between the pile head and pile shaft diameter so, 590mm. This is summarized in Table 4.

Table 4 Pile specifications

Pile specifications		
Length [m]	Diameter [mm]	Average Diameter [mm]
15.5	650/530	590

Furthermore, the model is based on the soil properties of the aquifer represented by the permeability (*k*), layer thickness (*D*) and storativity (*S*). And the aquitard confining the aquifer by means of the hydraulic resistance (*c*). As no data of the soil properties of both the sand layer (aquifer) and clay layer (aquitard) was available from the project information, these were first estimated based on literature and then adjusted to fit the measurements. As the model only describes the excess pore pressures generated by the volume displacement in the aquifer, the measurement data was first converted to the difference in hydraulic head (ϕ) from the moment that the pile had reached the top of the sand layer, so $\Delta\Phi_2$ from Table 3.

A comparison between the measurements and the geohydrological model is illustrated in Figure 4.5 and Figure 4.6.

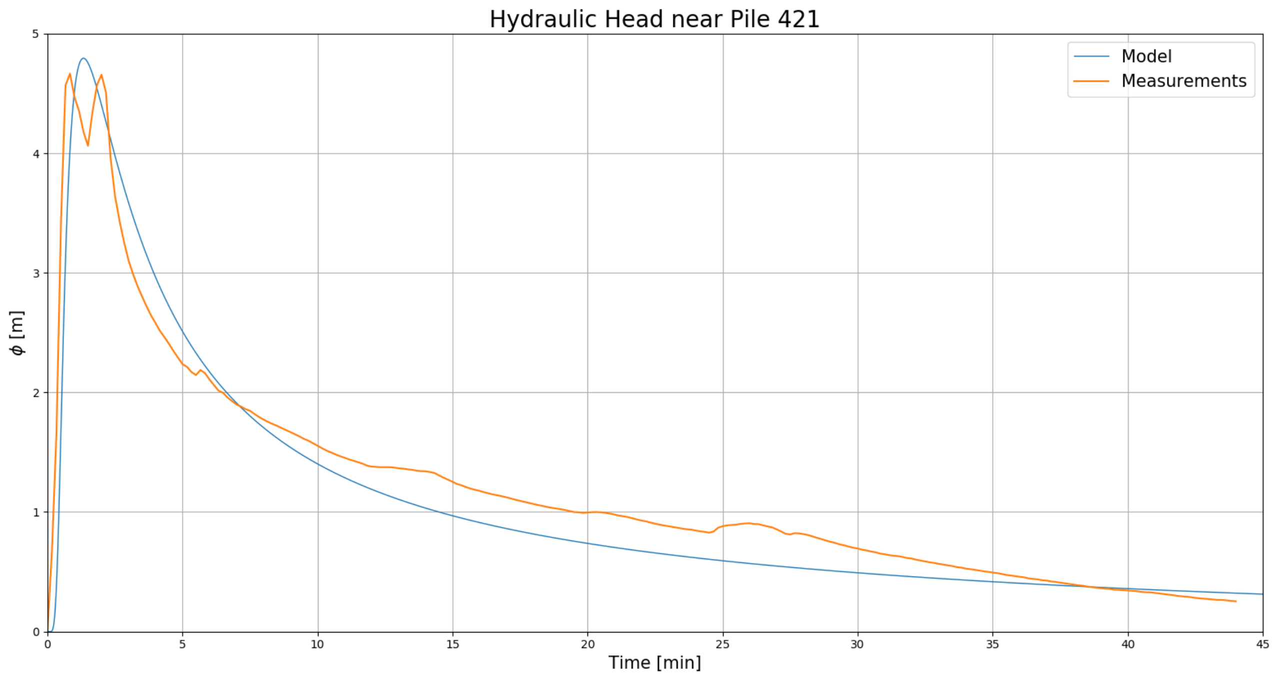


Figure 4.5: Hydraulic head (φ) increase [m] at r = 1.4m with k = 2 m/day and S = 0.0034

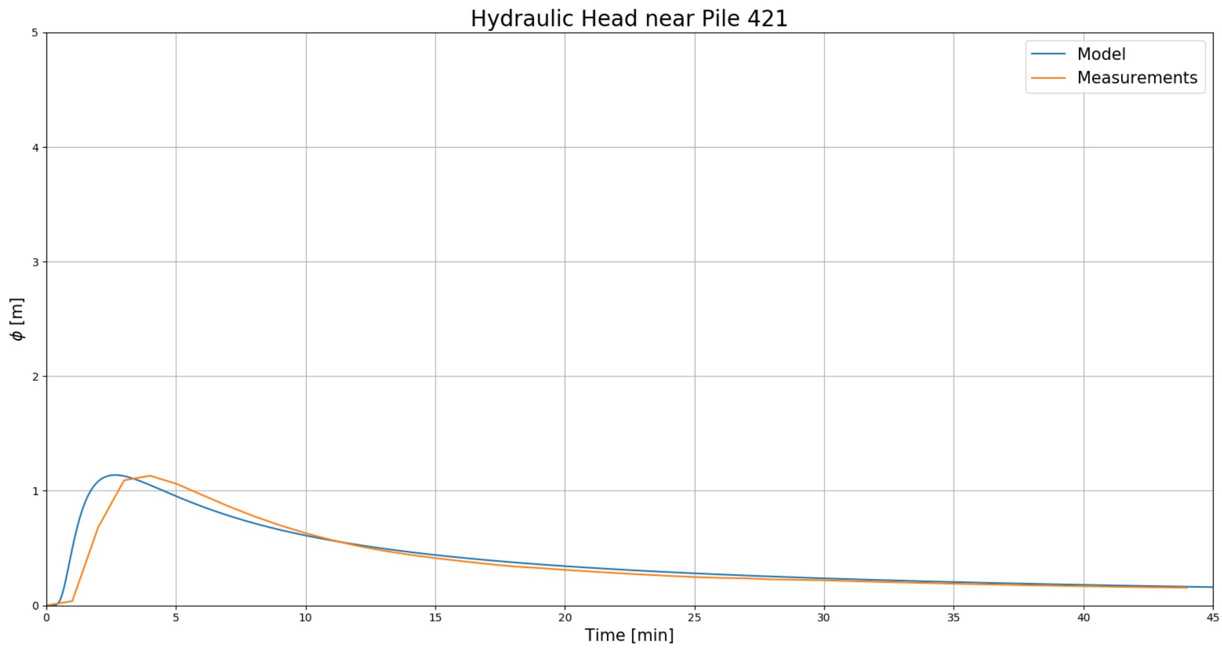


Figure 4.6: Hydraulic head (φ) increase [m] at r = 5.3m with k = 4 m/day and S = 0.001

The figures show good correlation between the geohydrological model and the measurements. Although, it should be noted that the aquifer parameters are fitted to the measurements. It was found that it was not possible to fit the measurements close to the pile and further away with one set of aquifer parameters. The measurement at a radial distance of 5.3m from the pile would be underpredicted if the parameters were chosen that correlate to the measurement at a radial distance of 1.4m. And vice versa, if the parameters are chosen such that they correlate with a larger radial distance, than the values close to the pile would be overpredicted.
If the aquifer parameters are compared for all the measurements close to the pile it can be found that the permeability, which describes the reaction of the aquifer system to the discharge and the subsequent consolidation is fairly equal. But the storativity shows a large variability in number, which is unfortunately also a parameter that is not well documented and only measurable by means of additional in-situ pumping tests. In Table 5 the results of the comparison between the measurements of the monitoring wells that measured six times per minute and the model is presented. The *r* is the radial distance from the pile to the monitoring well and t_i is the approximate time in seconds that the pile penetrates through the sand layer. Additionally, the corresponding graphs are presented in Appendix A.5 .

Table 5 Aquifer Parameters from Geohydrological Model

Pile	r [m]	k [m/day]	S [-]	t_i [s]
421	1.4	2	0.0034	15
252	1.1	2	0.006	20
57	1.0	1.5	0.006	20
246	1.1	2	0.009	25

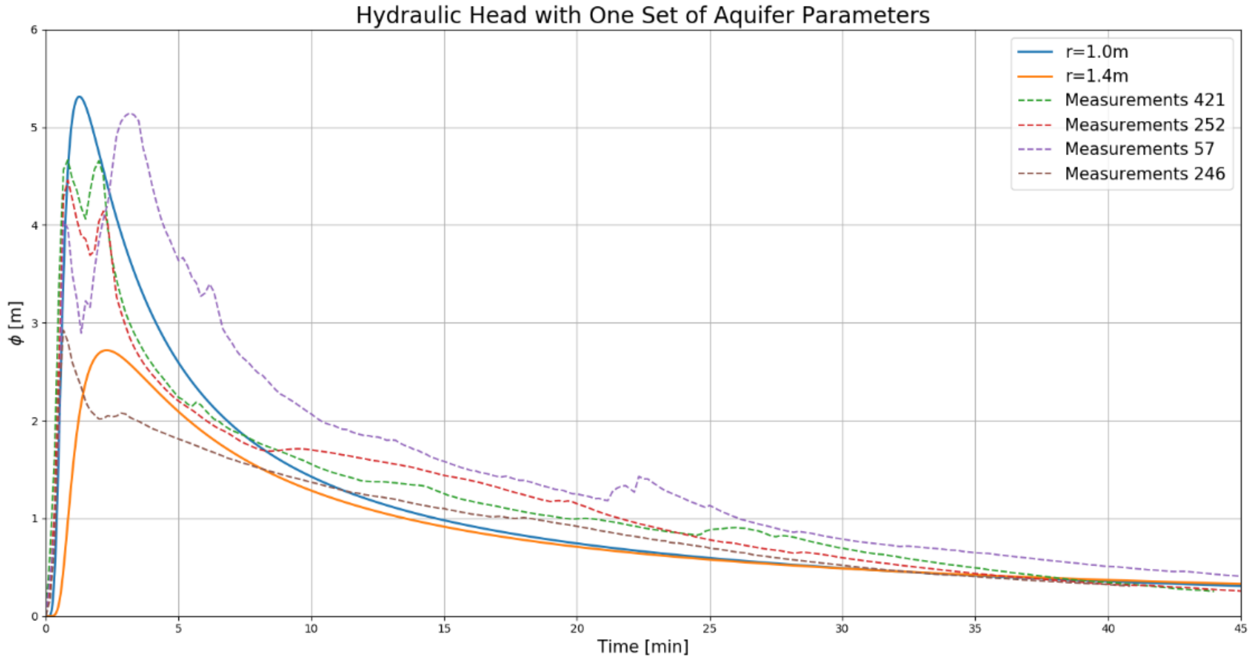


Figure 4.7: Comparison between measurements and 'best' set of aquifer parameters

Figure 4.7 shows the comparison between the measurements and the 'best' set of aquifer parameters. The best set of aquifer parameters were chosen as follows: k=2m/day, S=0.006, t=20s and the thickness of the sand layer D=1m. As could be expected the largest values is modelled closer to the pile (r=1.0m) and decreases with radial distance (r=1.4m). The measured increase in hydraulic head seems to fall within this upper and lower bound of the model. But if the measurements and modelled values are compared individually there are some clear deviations. The hydraulic head around pile 421 (at r=1.4m) is underestimated and the hydraulic head near pile 246 (at r=1.1m) is overestimated with this set of aquifer parameters. This does relate to the storativity which has the largest deviation between the individual fitted parameters (Table 5) and the best fit aquifer parameter set assumed here. It was also found that the hydraulic head at larger radial distance is underestimated with this set of aquifer parameters which is also mainly influenced by the storativity. But the comparison also shows that the dissipation at longer time intervals diverges and the hydraulic head at the start of withdrawal are in the same order of magnitude.

Overall, it can be concluded that the hydraulic head (or excess pore water pressure) resulting from volume displacement of water in a confined sand layer due to the installation of a screwed full displacement pile system can be described by a geohydrological model for unsteady-state flow in a leaky aquifer system. As can be expected the water level in the field is influenced by several processes (e.g. installation effects of the machine), which is visible in the fluctuation of the water level in the measurement data, but is not incorporated in the geohydrological model.

4.2.3 Limitations of the Geohydrological Model

First of all, the geohydrological model can only describe the excess pore water pressures generated due to the volume displacement of the pile in the aquifer, i.e. intermediate sand layer. Therefore, only the hydraulic head from the moment that the pile reaches the sand layer was considered.

The model is fundamentally restricted by the grout pressures next to the casing. This is not taken into account in the model and large increases in the hydraulic head are found very close to the pile. The model is also restricted by heave of the layers that confine the aquifer. Heave will occur if the water pressures exceed the volumetric weight of the soil above it, hence influencing the storativity of the aquifer system. From the measurement data it can be seen that the hydraulic head increases up to almost 6 meter, thus an additional water pressure of approximately 60 kPa which is very large at a depth of 3 to 4m beneath ground level. But as the peak of the hydraulic head also quickly dissipates, the radial dissipation seems to be modelled correctly. Additionally, the variation in the measurements of the radial influence of the installation process of the piles is presented in Figure 4.8. The two most important conclusions that can be drawn from the figure are (1) there is a large difference in hydraulic head close to the installed pile (<1.5m or <3*D*) and (2) a hydraulic head increase of 0.5 to 1.6m can be found at distances up to 10 times the pile diameter. Overall, this clearly indicates the large effect on the water pressure in the sand layer, which can be even larger near the pile perimeter, and subsequently the large radial extent of the pressure increase. The variation in the values of the hydraulic head can be restricted by the grout pressure around the casing.

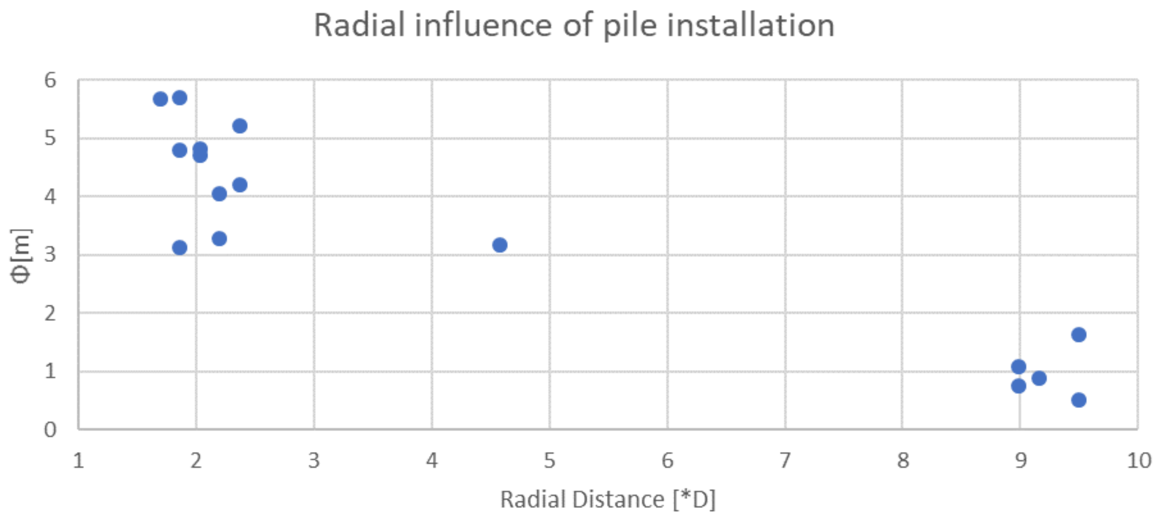


Figure 4.8 Maximum hydraulic head (Φ) versus radial center to center distance in the number of diameters

The radial flow in the aquifer is determined by its transmissivity and storativity and the hydraulic resistance of the aquitard. The transmissivity is the product of the permeability and the saturated thickness of the aquifer and is the rate of flow under a unit hydraulic gradient through a cross-section of unit width. The saturated thickness is determined from CPTs near the piles and is approximately one meter thick although slightly varying in the project location. By changing the permeability the reaction time of the aquifer system is influenced, this parameter determines the accumulation and dissipation time of the water pressure but for small variations its effect on the increase of hydraulic head is limited. From (4) it can be seen that the permeability also influences the leakage factor, a measure for the spatial distribution of the leakage through an aquitard into a leaky aquifer and vice versa. The leakage factor is also a function of the hydraulic resistance of the clay layer but variations in the practical range of clay has negligible effect on the output of the model, as was also found by Hoefsloot (2001). This can be explained because the radial influence of the pile installation is much smaller than the value of the leakage factor. Finally, the storativity has a large influence on the peak value and the radial extent of the hydraulic head. As it is defined as the volume of water that the aquifer can store per unit horizontal area and per unit rise of piezometric head, it is logical that a small value of S indicates that little water can be stored in the aquifer and thus a larger radial influence will be found. The result of the model is very sensitive to this parameter. This is the main disadvantage of the model since the storativity of the soil is often unknown. It can be obtained from pumping tests or other techniques such as the Hydraulic Profiling Tool (HPT) and Mini-Pumping Test(MPT).

The variation in the measurements in space and time can be explained by the following causes:

- Heterogeneous nature of the sand layer, which has a large variation in horizontal and vertical permeability
- The sand layer is actually a layered formation with small bands of clay which is neglected in the model
- The grout pressure around the casing
- Possible influence of compaction near the pile
- Other installation effects than the volume displacement in the aquifer

Table 6 General aquifer parameters found in literature

	Kruseman & de Ridder (1991)	*Bot (2016)*	*Verruijt (2001)*	*Hoefsloot (2001)*
Fine Sand				
k [m/day]	1-5	1-10 (k_h)	0.9-86 (Sand)	
S [-]	$5 \cdot 10^{-5} – 5 \cdot 10^{-3}$	$5.5 \cdot 10^{-5}$ (<10m depth)		0.001 – 0.01 (confined aquifer)
Clay / Average Holocene in Western Netherlands				
c [days]		1000 / 150-400		

The model is not able to react on these variations as it is based on the assumptions that the aquifer (1) has an infinite areal extent and (2) is homogeneous, isotropic, and of uniform thickness over the area influenced by the pile installation process which will most likely not be the case in real soil conditions (Kruseman & de Ridder, 1991). But the values obtained with the model fall well within typical values found in literature, see Table 6.

4.2.4 Conclusion

Volume displacement due to the installation of a full displacement pile has been modelled with a geohydrological model for unsteady state flow in a leaky aquifer system. From the comparison between the model and the measurements around the installed piles it can be concluded that the model is able to describe the generation and subsequent dissipation of water pressures reasonably well. Because the hydraulic parameters were unknown these were estimated and subsequently fitted to the measurements. It was found that the obtained parameters correspond with the limits described in literature.
The measurements showed that the excess pore pressures differ from magnitude but also from radial extent at every pile location, this is hard to model with the geohydrological model as it can only describe the volume displacement of water during the period that the pile penetrates through the sand layer and does not take other processes into account. The variation between the model and the measured values of the hydraulic head can be explained by the assumptions of the model, its limitations and the influence of other installation effects. The geohydrological model can give close approximation of the measured values if the parameters are chosen correctly. Some of the input parameters can be computed quite accurately with a limited range of deviation while others have to be assumed or detailed site investigation is required.

As already discussed the transmissivity and storativity of the aquifer together with the hydraulic resistance of the aquitard determine the reaction of the aquifer system. The latter has negligible influence on the model as the radial influence zone of the volume displacement is much smaller than the leakage factor. The transmissivity has a large influence on the reaction time of the system as a result of the volume displacement and mainly describes the dissipation over time. In addition, the value of the storativity has a large impact on the amount of hydraulic head increase and the radial influence, and in a less pronounced way on the reaction time. The storativity of an aquifer if often unknown and common tests are time consuming. Together with its large spatial variation and influence makes the model very sensitive for this parameter.

Finally, the measurements show that a large amount of the hydraulic head increase dissipated very quickly and kept dissipating until the withdrawal of the casing. During this time period a steel casing separates the fresh concrete from the soil and thus the water is not able to penetrate the fresh concrete. Therefore, it is concluded that the hydraulic head resulting from the descending phase cannot be the cause of excessive bleeding of the installed piles in this project. But the hydraulic head increase was not fully dissipated at the onset of withdrawal and therefore might act as a starting level for later effects. This is substantiated by the measurements where a cumulative effect of the hydraulic head increase was found by successive pile installation at 10D radial distance.

In addition, the measurements show a large radial influence, which means that a previously installed pile can be affected by the volume displacement of a new installed pile within the minimum recommended radial distance. The large radial influence as well as the cumulative effect substantiate the possible effect of water flow to a previously installed pile. This mechanism is enhanced by small radial distance, large amount of piles and restriction of water flow by sheet piles.

4.3 Pore Water Pressure Generation during Withdrawal

After a pile reached the bearing layer, the reinforcement cage was suspended and the concrete was placed. Subsequently, the casing was extracted thereby leaving the tip, concrete and reinforcement cage behind in place. Figure 4.1 illustrated that at the beginning and during the withdrawal procedure an increase in hydraulic head, or excess pore water pressure, was generated. This added up to the increased hydraulic head from the descending phase which was not yet fully dissipated. The increase in hydraulic head from the withdrawal procedure was not expected beforehand. During withdrawal the fresh concrete flows out and will get in contact with the surrounding soil and pore water, thus the increase in hydraulic head will have an influence on this interaction. Therefore, further research is done to the potential causes of the increases in hydraulic head during the withdrawal process.

4.3.1 Possible Causes

Figure 4.9 and Figure 4.10 show that during withdrawal two separate increments in the hydraulic head can be distinguished. The first increment can be noticed directly at the start of withdrawal, the second increment at a later moment during withdrawal, the increase of the latter is substantial larger than the first. The possible causes of the two increments in hydraulic head will be discussed separately in this paragraph.

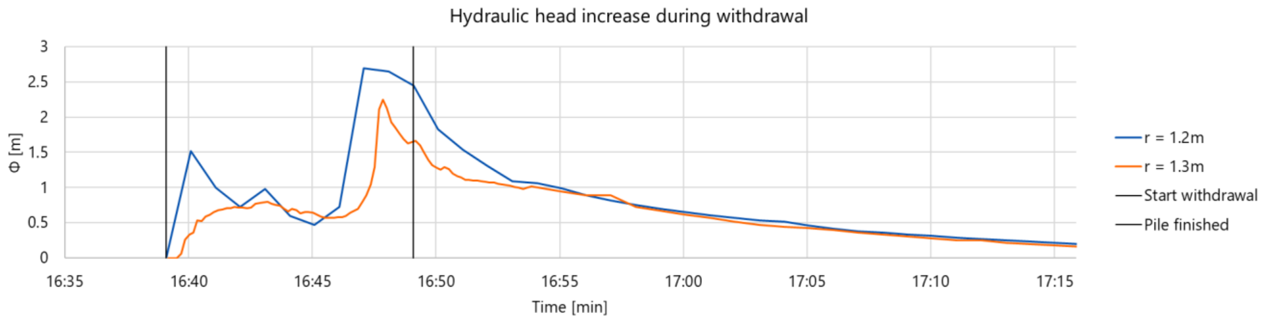


Figure 4.9: Hydraulic head increase during withdrawal – Example Test Field 1

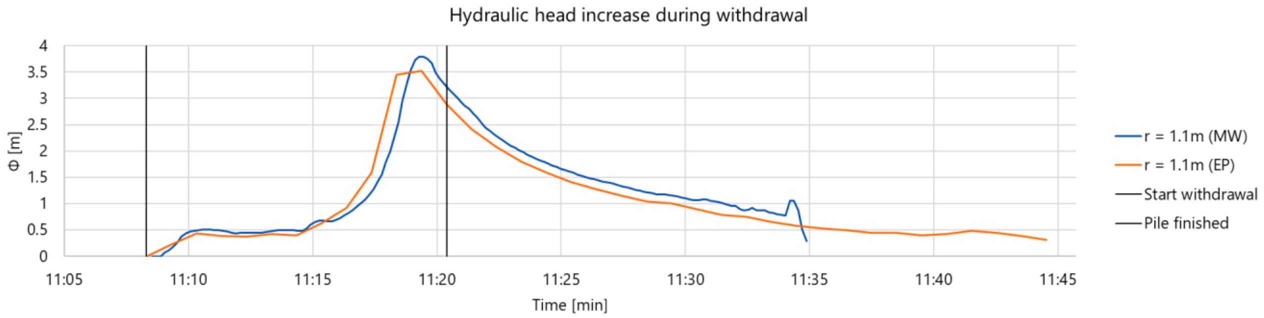


Figure 4.10: Hydraulic head increase during withdrawal – Example Test Field 2

4.3.1.1 Direct increase of the hydraulic head

From the measurements it is clearly seen that immediately after the start of withdrawing the temporary casing, the hydraulic head increased. Several causes are listed that might have influenced the hydraulic head in the sand layer.

- Shearing of the sand layer
- Pull Up force of the machine
- Weight of the casing

Whether these causes can have resulted in the measured increase of the hydraulic head will be evaluated. Firstly, the piles are installed with grout injection leaving a grout cake in place between the soil and the casing. This will limit the shear friction between the steel casing and the soil during withdrawal. Therefore, shear between the casing and the soil is assumed to have no influence on the hydraulic head in the sand layer. Additionally, in some of the measurements the hydraulic head kept constant or decreased during further withdrawal, which substantiates that shearing in this case has limited influence on the hydraulic head. Secondly, the crane registrations showed that the pull up force of the machine started to increase simultaneously with the start of withdrawal, which is logical. This pull up force, necessary to withdraw the casing, will result in an additional force on the soil as the machine will distribute this over the soil or crane mats. This mechanism will be tested in Paragraph 4.3.2.
Finally, from the moment the pile reached the design depth the casing was supported by the deep sand layer at which it was founded. From the start of the withdrawal the weight of the casing must be supported by the machine which can add up to the total weight of the machine. As an indication, it was calculated that a steel tube of 15mm thickness and a length of 25 meters would approximately weigh 5 tons. This might add up to the total force of the machine on the soil. The crane registrations show values around 20 to 35 tons for the pull up force causing the effect of the weight of the casing to be limited. Therefore, it is concluded that the effect of the weight of the casing itself will not cause the increase that is measured, but it can add up to the total force of the machine on the soil.

4.3.1.2 Second increase of the hydraulic head

At some time after the start of withdrawal, the hydraulic head started to increase again. From the measurements it can be concluded that the increase is substantial and adds up to the first increase after the start of withdrawal. The possible causes that can be thought of that might trigger the increase at this moment are listed as follows:

- Pull Up force of the machine
- Rate of withdrawal
- Outflow of concrete
- Shear of the sand layer

The shear force on the sand layer is again neglected on the same grounds as discussed before. After the start of withdrawal the crane registrations show the deviations in the depth, torque, pull up, hydropressure, rate and RPM (revolutions per minute), see Appendix A.4 . When considering the whole withdrawal process, the crane registrations do not show unique link that can substantiate the second increase in the hydraulic head. But it can be noted that, for all the piles, the withdrawal stopped after approximately 3.5 meters from the pile tip level and the hydraulic head started to dissipate from the moment that the casing was withdrawn out of the intermediate sand layer. For two of the piles, the start of the second increase of the hydraulic head did coincide with the re-start of withdrawal. This suggest a relation between the rate of the withdrawal process and the increase in hydraulic head, nonetheless this relation is not found at the other piles where the increase in hydraulic head started later. Finally, another possible cause of the increase is the volume displacement resulting from the concrete pressure from the moment that the fresh concrete flows out of the casing. First of all, this mechanism was thought to be an influencing factor and is further elaborated in Paragraph 4.3.3. But after receiving the crane registrations it was found that for all the piles the casing was still in the deep sand layer at the moment that the hydraulic head started to increase again. This seems to disprove that the outflow of concrete and subsequent concrete pressure has a large effect on the increase of the hydraulic head in the intermediate sand layer.

4.3.2 Direct increase of the hydraulic head

From Figure 4.9 and Figure 4.10 it is seen that the water level in the intermediate sand layer increases immediately after the machine started with withdrawal of the temporary casing. This is found in all the measurements around the piles. As stated in the introduction of this paragraph the grout body around the pile shaft will reduce the shear stress in the soil and is therefore assumed to be negligible. Consequently, the increase in pore water pressure in the intermediate sand layer should result from the machine above it. This can be substantiated by the large pull up force that is recorded at the start of withdrawal.

4.3.2.1 Analytical Model

"These stresses decrease with depth, of course. In engineering practice, it is sometimes assumed, as a first approximation, that at a certain depth the stresses are spread over an area that can be found by drawing a line from the load under an angle of about 45°." (Verruijt, 2001).This approach is used to control whether the measured values of the hydraulic head can be caused by the pull up force exerted by the machine, see Figure 4.11. The specification of the machine are used to assume the amount of crane mats that probably will be used to transfer the force. This area is increased to the mid-height of the sand layer which is 3.5 and 4 meters below ground surface for Test Field 1 and Test Field 2, respectively. Figure 4.9 shows a relatively large increase in water pressure around one of the piles in Test Field 1. Compared to all the other piles the increase is approximately three times larger and cannot be caused by the pull up force only. No reason has been found that can substantiate this reaction. On average, the increase was around 5 kPa which in the abovementioned calculation method would suggest pull up forces around 30 to 50 tons. From the crane registrations of the machine pull up forces of 10 to 35 tons were measured so this indicates that the cause of the excess pore water pressure from the onset of withdrawal can be caused by the pull up force of the machine.

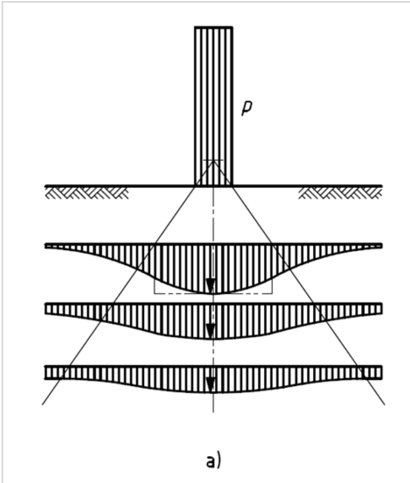


Figure 4.11: Load Distribution over Depth

4.3.2.2 Plaxis Model

The pull up force from the machine was also tested with a simple Plaxis model. Plaxis is a Finite Element program for geotechnical applications. This model incorporates the stiffness of the soil layers and is therefore more advanced than the simple analytical model from Paragraph 4.3.2.1.

Error! Reference source not found. illustrates a schematization of the situation in the field, with the location of the crane, the pile and the measurement devices. This will be used to verify the measurement data with the axisymmetric Plaxis model. Figure 4.13 shows the axisymmetric model in which the force of the machine was converted to a distributed load. The area of the distributed load equals the area influenced by the machine via the crane mats (A_{circle}=A_{crane}). From the crane dimensions it was assumed that the machine would occupy an area of 30m^2 (5 crane mats of 1x6 meters). This can be converted to a circle with radius 3.1m. And a pull up force of 30 ton would suggest a line load of 10 kN/m/m. The value of the pull up force does correspond to the measured values following from the crane registrations.

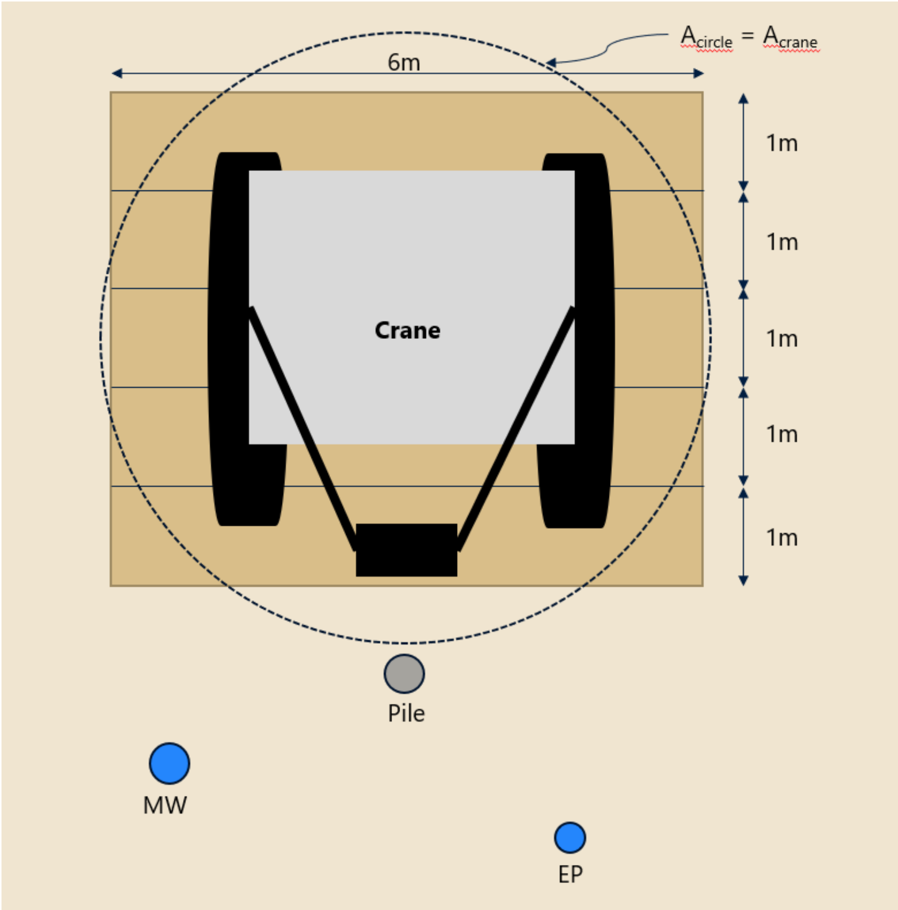


Figure 4.12: A schematized top view of the crane, pile and measurement positions

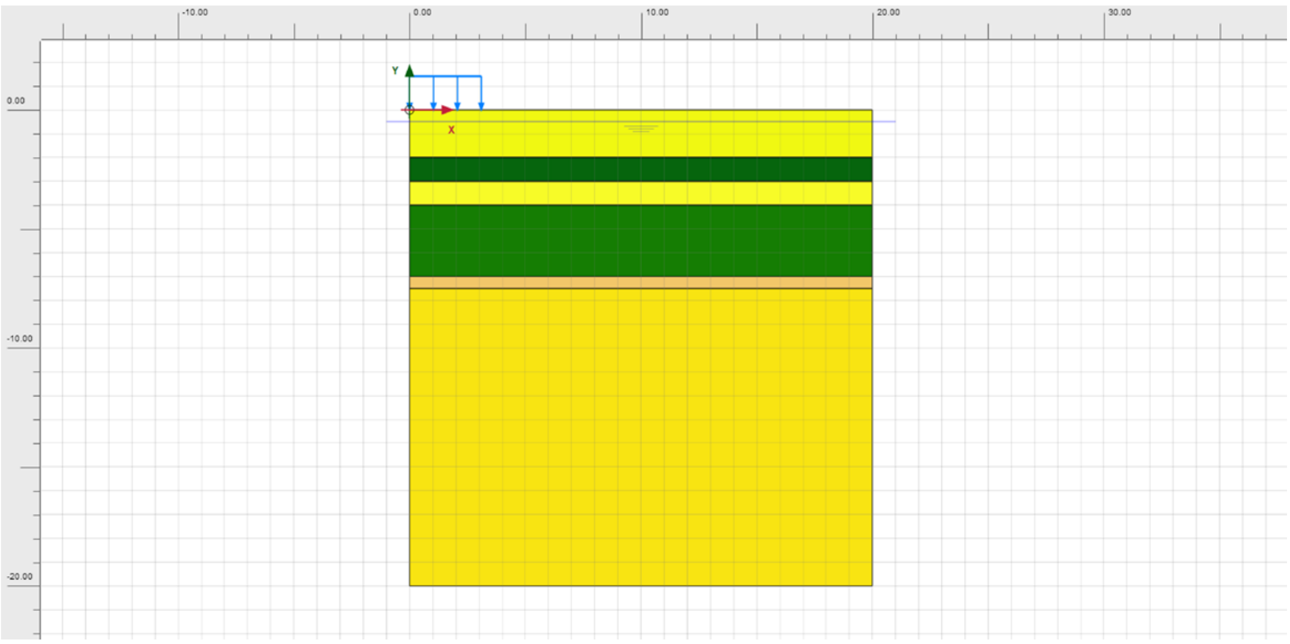


Figure 4.13: Plaxis model simulating pull up force

The reaction force from the machine due to the necessary pull up force will compress the soil underneath it. As illustrated in Figure 4.11 this will distribute under an angle over depth which will depend on the stiffness of the soil layers. Table 7 shows the assumed soil parameters. The excess pore water pressure is modelled based on the assumption that the intermediate sand layer is undrained hence the volumetric compression from the machine will result in excess pore water pressure. The modelled excess pore water pressure at mid depth of the intermediate sand layer is presented in Figure 4.14. In the figure the excess pore water pressure is plotted over the radial distance from the center of the machine, the highlighted area is the most likely distance between the center of the machine and the measurement devices (i.e. equal to the radial distance from the pile to the measurement devices). This example shows that a pull up force of 30 ton can lead to excess pore water pressures of around 3 kPa (or 0.3m water column) at these distances. If this is compared to the measured increase in hydraulic head after the start of withdrawal, presented in Figure 4.15, it can be seen that the order of magnitude of the modelled excess pore water pressure is similar.

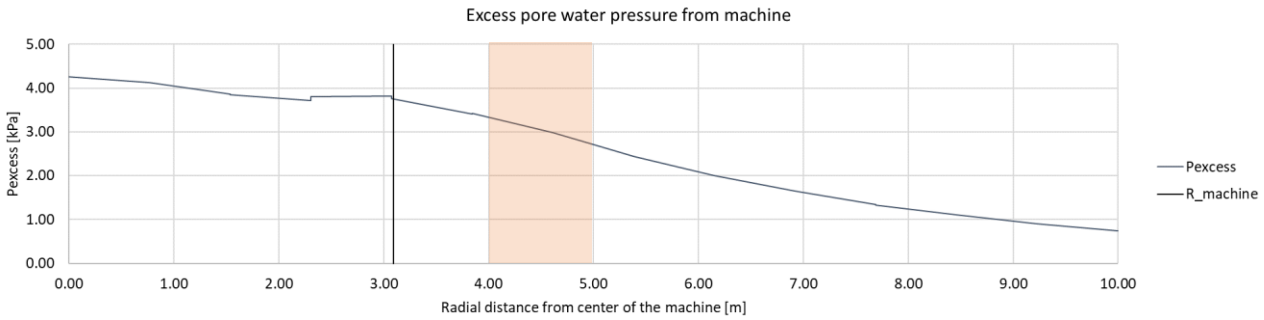


Figure 4.14: Excess pore water pressure modelled in intermediate sand layer from reaction force of the machine

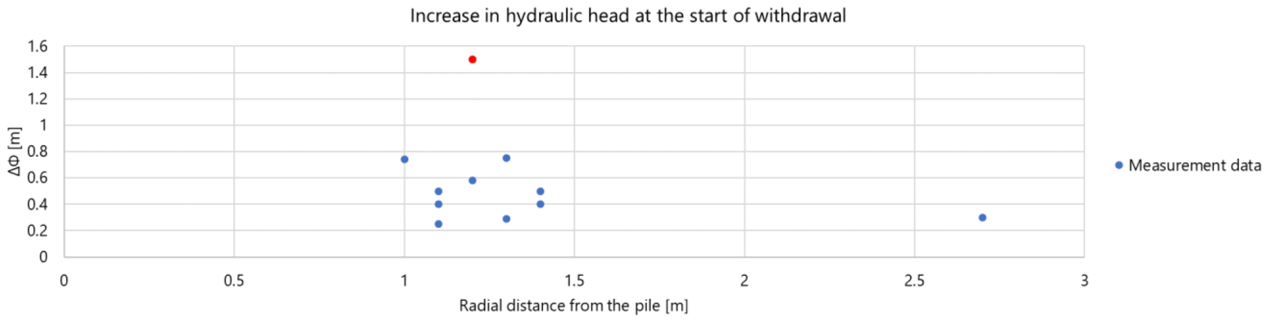


Figure 4.15: Maximum measured values of the hydraulic head after the start of withdrawal

The measurements show larger values than modelled, this can be explained because the measurement data points are the maximum values that were measured after the start of withdrawal, so this will be effected by the continuous process of withdrawal. In addition, the rough estimation in the model of the area that is affected by the force of the machine, the additional weight of the casing itself and other installation effects can substantiate this difference. No explanation was found for the outlier in the measurements, indicated in red, as another measurement device at approximately the same radial distance from the pile shows only half of the value. Finally, the crane registration of the pull up force show good agreement with the increase and dissipation of the hydraulic head and it is therefore concluded that the direct increase in hydraulic head can be caused by the resulting force of the machine on the ground.

4.3.2.3 Conclusion

From the measurements the following three things can be concluded:

1. The increase in hydraulic head generated at the onset of withdrawal added up to the existing water pressure from the installation process.
2. The different values of the hydraulic head will probably be caused by variations in the distance from the measurement devices to the machine, the pull up force, the rate of withdrawal or other installation effects.
3. No general trend was found for the generation and dissipation of the water pressures after the increase caused by the machine. At some of the piles the water pressures dissipated, at other piles the water pressure kept relatively constant and in others they kept on increasing.

The potential causes of the increase in hydraulic head at the start of withdrawal are discussed and it can be concluded that a relation exist between the pull up force of the machine and the increase in hydraulic head. But other installation effects cannot be excluded.

4.3.3 Second increase of the hydraulic head

The possible causes of the second increase of the hydraulic head was discussed earlier. From this analysis it was found that one of the mechanisms that could cause this increase is the volume displacement from the outflow of the fresh concrete and the subsequent concrete pressure. This mechanism is tested in this paragraph.

4.3.3.1 Outflow of Concrete

During withdrawal of the casing, the fresh concrete will flow out and exert a pressure on the surrounding soil. The final pile diameter will depend on the concrete flow, which is determined by the concrete rheology, reinforcement spacing and rate of withdrawal, and subsequent equilibrium between the concrete lateral pressure and the soil pressure at that depth. Figure 4.9 and Figure 4.10 showed clearly that a sharp increase in the hydraulic head is measured starting at a certain moment during withdrawal. The increase is considerable and together with the fact that at that moment the fresh concrete gets in contact with the surrounding soil and water pressure this might be the event that triggers the bleeding mechanism. Hence the effect of the concrete pressure on the surrounding soil will be further elaborated on in this paragraph.

First of all, the moment of the increase of the hydraulic head was looked at because this should give information on the interaction between the outflow of the fresh concrete and the sand layer. Initially, only the starting time of withdrawal and the time that the casing was fully withdrawn was known hence it was not possible to determine the exact position of the casing at the start of the increase. Thus, as a first approximation an average velocity was calculated to determine the depth of the casing. From the average velocity and the starting time of the measured hydraulic head increase it was calculated that the casing did not yet reach the sand layer at all the piles in both test fields. As a result, the following hypothesis was proposed.

Hypothesis 1:

'The measured hydraulic head increase in the sand layer is caused by vertical compression from the clay layer underneath it. Under undrained conditions the clay layer will react without volumetric changes and thus will the horizontal displacement, resulting from the lateral concrete pressure after withdrawal of the casing, lead to vertical displacement of the clay layer thereby compacting the sand layer above it.'

The hypothesis is tested with Plaxis. In Plaxis a two-dimensional axisymmetric model is used to model the effects of the lateral concrete pressure on the surrounding soil. It is not possible in Plaxis to model the fluent and continuous process of the installation of a pile (Broere & van Tol, 2006) and Dijkstra, Broere & van Tol (2008) discuss that in the small strain finite element code large strains are not considered and the installation method could only very schematically be modelled. In addition, no data was available on the horizontal deformation due to the installation process. Therefore, these steps are excluded from the analysis and the model is only focused on the withdrawal process.

Soil parameters

The soil stratigraphy is similar to that presented in Table 2, so the intermediate sand layer is assumed to be 1m thick. The water level was assumed 0.5m below ground surface following from project data. The soil stratigraphy is again presented in Figure 4.16 and the corresponding soil parameters are presented in Table 7. No soil parameters were available from the project data so the soil parameters were based on project data from a nearby project location where extensive soil tests were performed to obtain the parameters. The project data is confidential but the soil stratigraphy was relatively similar and therefore assumed to be more realistic than Table 2.b of the NEN-9997 (2017).

The soil-structure interface (R_{inter}) is assumed very low because the piles were installed with grout-injection.

Table 7 Plaxis input parameters

Parameter	1. Sand (Toplayer)	2. Clay1	3. Sand (Intermediate)	4. Clay2	5. Peat	6. Sand (Deep)
General						
Material model	HS	HS	HS	HS	HS	HS
Drainage type	Drained	Undrained (A)	Undrained (A)	Undrained (A)	Undrained (A)	Drained
γ_{unsat}	17	16	17	18	12	18
γ_{sat}	19	16	19	18	12	20
Soil parameters						
E_{50}^{ref}	30.00E3	6000	6500	6000	7500	30.00E3
E_{oed}^{ref}	30.00E3	3000	6500	3000	3800	30.00E3
E_{ur}^{ref}	90.00E3	25.00E3	33.00E3	15.00E3	38.00E3	90.00E3
m	0.5	1	0.5	0.9	0.9	0.5
c_{ref}'	0	2	0	2	4	0
ϕ'	30	22.5	30	22.5	17.5	32.5
ψ	0	0	0	0	0	2.5
Interface						
Interface strength	Manual	Manual	Manual	Manual	Manual	Manual
R_{inter}	0.01	0.01	0.01	0.01	0.01	0.01

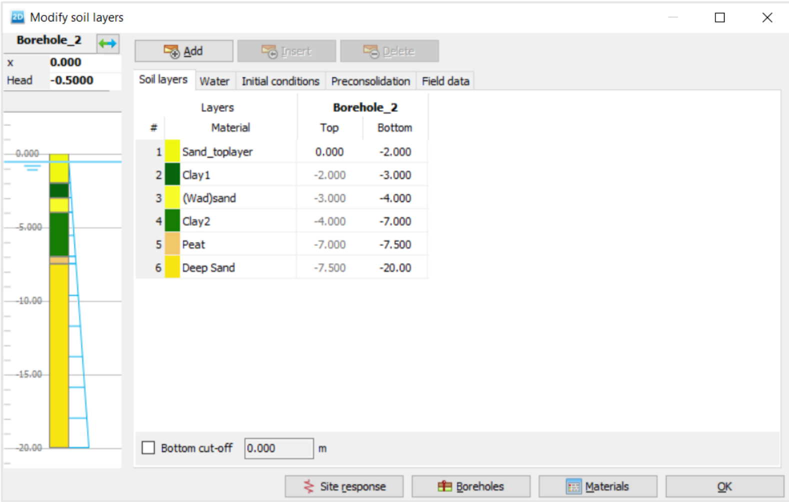


Figure 4.16: Soil stratigraphy for Plaxis model

Dimensions and structures

The dimensions of the model are 40m width (radius) and 20m depth. This is chosen such that the boundary conditions do not influence the displacements and water pressures. The steel casing of the pile has a diameter of 0.53m so the radius of the pile is 0.265m in the axisymmetric model and the pile length is 15.5m. The steel casing is modelled as a plate element with material properties as presented in Table 8.

Table 8 Plate material set

Parameter	Name	Value	Unit
Material type	Type	Elastic; Isotropic	-
Normal stiffness	EA	$2.1*10^6$	kN/m
Flexural rigidity	EI	17.5	kNm^2/m
Plate thickness	d	0.01	m

Within the steel casing a fresh concrete column is present. This can be modelled in Plaxis in three ways. Firstly, Plaxis developed 'The Concrete model' which is an elastoplastic model for simulating the time-dependent strength and stiffness of the concrete, strain hardening-softening in compression and tension as well as creep and shrinkage (PLAXIS, 2019a). Secondly, the soil elements can be deactivated and user-defined water conditions can be specified to simulated a liquid concrete column. In this case a constant pressure increment with depth is assumed. Thirdly, a distributed load can be applied to simulated the concrete lateral pressure. This allows for changes in the pressure distribution along the depth if the lateral pressure of the concrete is not hydrostatic.

As a first indication of the pressure a hydrostatic concrete pressure distribution is assumed. This can be modeled by either one of the two latter models as the resulting pressure distribution will be equal. For convenience the user-defined water conditions is used, increasing 24 $kN/m^2/m$ with depth, because this automatically insert the horizontal lateral pressure and the vertical pressure on the pile toe.

Mesh

In Plaxis 2D two options are available for the elements, these are 15-node or 6-node triangular elements. In this model 15-node elements are chosen as these are default elements and recommended for axisymmetric analysis. The size of the elements are medium but enhanced mesh refinement is applied that takes the mesh refinement around the plate into account (PLAXIS, 2019b).

Boundary conditions
At all the boundaries the default settings apply. This means that the displacements in the x-direction are fixed and the displacements in the y-direction are free at the vertical boundaries. At the top boundary (surface level) all the displacements are free and for the bottom boundary all the displacements are fixed. The waterflow is open in all direction except in the bottom horizontal boundary where it is closed and no groundwater flow can occur (PLAXIS, 2019b).

Phases
In practice, withdrawal of the casing is a continuous process were the casing is pulled out in an oscillating manner. This is hard to model with Plaxis as it does not allow for large strains. It was tested if it was possible to model the withdrawal process by means of prescribed displacements on the plate element. But the combination of the length of the casing (15.5m) and the restriction of large strains this procedure was not feasible.
Therefore it is chosen to stepwise disable the plate elements to simulate the withdrawal of the casing. The calculation phases are presented in Figure 4.17.

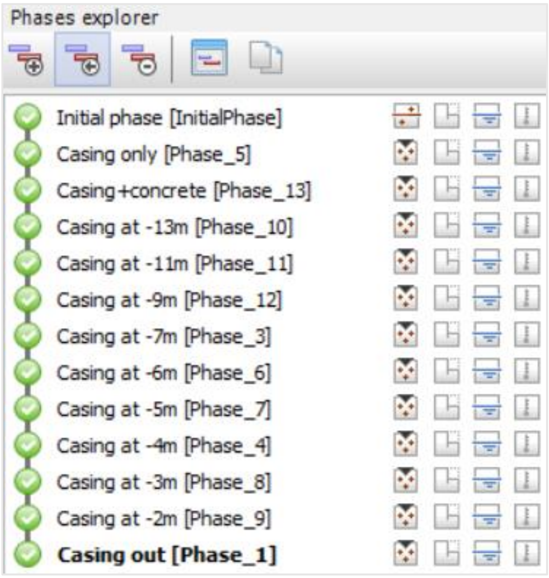


Figure 4.17: Plaxis - Phases explorer

Results
At each phase a part of the casing is deactivated and subsequently the hydrostatic concrete pressure distribution exerts a horizontal stress increment on the surrounding soil elements. How the stress increment on the soil will be converted to strain increments will depend on the drainage type and material stiffness matrix governed by the input parameters.

Firstly, focusing on the hypothesis the effect of the concrete pressure on the clay layer underneath the sand layer will be discussed. From the axisymmetric model it was found that almost all the radial compression, indicated as negative horizontal strains ($-\varepsilon_{xx}$) were translated in circumferential elongation ($+\varepsilon_{zz}$). This is illustrated in Figure 4.18, where the phase strains in the clay layer are plotted against the radial distance for the phase in which the casing was deactivated from -7m to -6m in the model. The same reaction was found throughout the entire withdrawal process, i.e. in all the phases, in all the soil layers. Hence little vertical displacements were generated in the clay layer underneath the sand layer. The small vertical phase strains ($P\varepsilon_{yy}$) that are generated show settlement very close to the pile perimeter ($R \approx r_0$) and further away the vertical strains are positive indicating upward displacements. The phase displacements for the three phases in which the casing is deactivated at the depth of the second clay layer is shown in Figure 4.19. This clearly

shows the displacement field following from the lateral concrete pressure, which mostly result in horizontal displacement. Due to undrained behaviour of the clay layer the horizontal displacement result in vertical upward displacement at larger radial distance.
The conversion of radial strains to circumferential strains indicate that the axisymmetric model incorporates the cavity expansion method to model the strains and subsequent displacements resulting from the increase in pressure inside the cavity. This strain behaviour was compared to a plane strain model in which all the horizontal strains following from the lateral concrete pressure are converted to positive (upward) vertical strains because of its assumption of plane strain conditions, thus no strains develop in the out of plane direction. The subsequent displacement field in the plane strain model shows a horizontal and vertically upward displaced field. Compared to the axisymmetric model a different displacement field is found close to the pile circumference, where the soil subsides in the axisymmetric model and less vertical displacements are modelled because the radial strains are converted to circumferential strains.

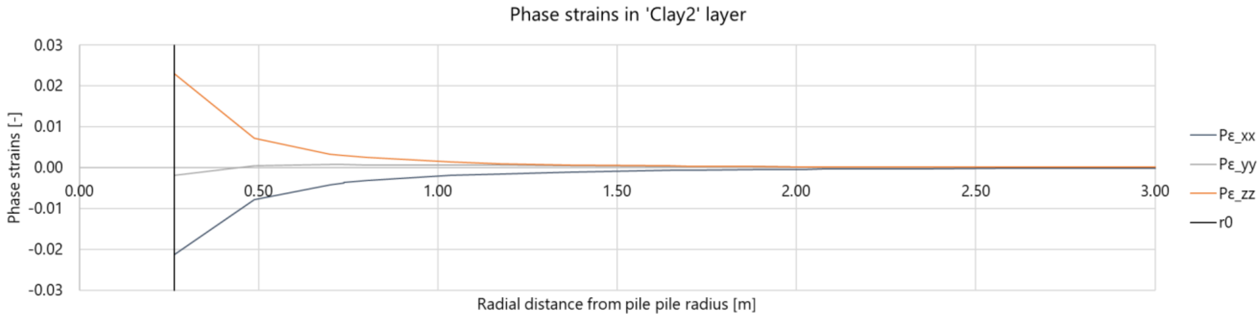


Figure 4.18: Phase strains in Clay2 layer resulting from deactivating the plate from -7m to -6m

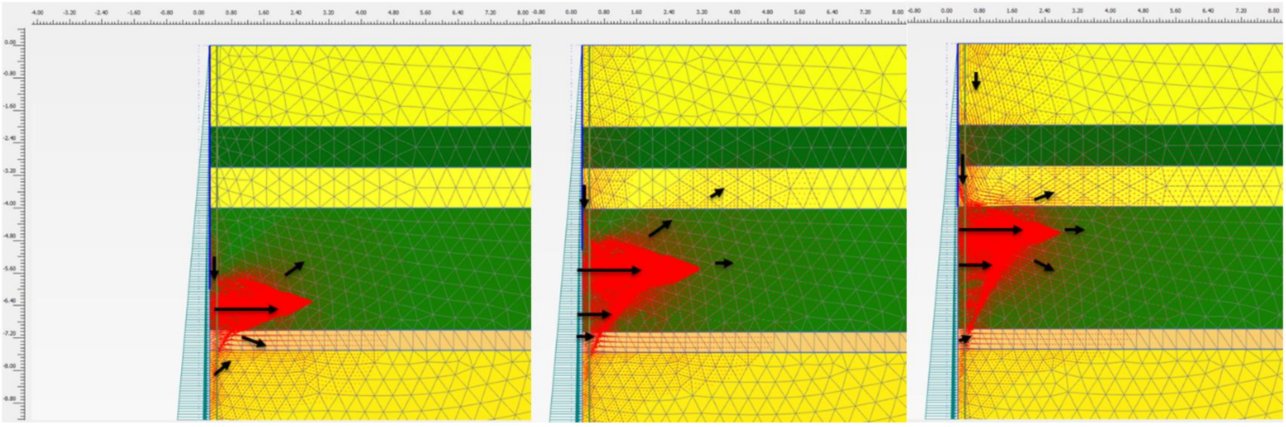


Figure 4.19: Phase displacement for the three phases where the casing is deactivated in the clay2 layer

Subsequently, little excess pore water pressure was generated in the intermediate sand layer during the modelled withdrawal process as only very small vertical displacements are modelled. This is illustrated in Figure 4.20, that shows the cumulative vertical displacements at the bottom of the sand layer. The values of the vertical displacements are very small but increasing as the casing progresses towards the sand layer. The hypothesis poses that the displacements of the clay layer would affect the water pressure in the sand layer above it. The calculated excess pore water pressures at mid depth of the sand layer show that the modelled outflow of concrete and subsequent displacements of the clay layer have very limited influence on the water pressure in the sand layer. And due to the settlement close to the pile radius above the deactivated plate even negative excess pore water pressure is modelled as a result of the volumetric expansion that is associated with this settlement underneath the sand layer. Only positive excess pore water pressure was generated when the lateral pressure of the concrete was exerted on the sand layer (between -4m and -3m under ground level) itself, see Figure 4.21.

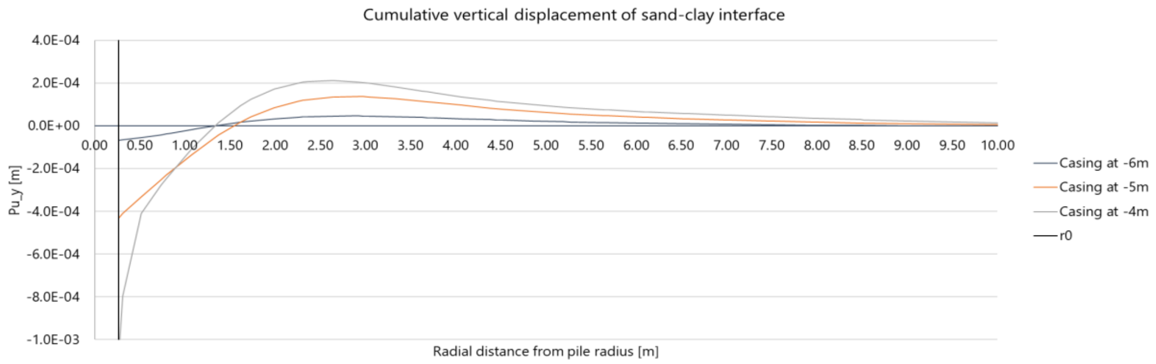


Figure 4.20: Cumulative vertical displacement at the bottom of the sand layer

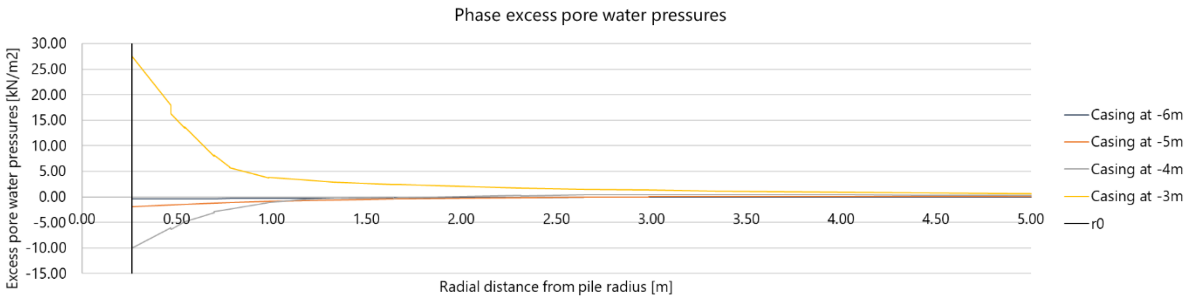


Figure 4.21: Phase excess pore water pressures at mid depth of the sand layer

Discussion

From the axisymmetric model it was found that the horizontal compressive strains, resulting from the modelled lateral (concrete) pressure, mainly resulted in circumferential strains. Subsequently, limited vertical displacement was modelled in the clay layer underneath the sand layer and thus limited excess pore water pressure was generated before the casing reached the sand layer itself.

The results from the axisymmetric model do not confirm the hypothesis as only limited vertical displacements were modelled as a result of the concrete lateral pressure. Hence positive excess pore water pressure was only modelled when the concrete column exerted a lateral pressure on the sand layer itself. This is contradictorily to the measurements as the hydraulic head only increased during the period that the casing was withdrawn through the clay layer. It must be noted that none of the effects during the descending phase, i.e. the volume displacement exerted on the surround soil, was taken into account in the model which is a clear limitation of the model as this would have disturbed the in-situ soil state and would change the reaction of the soil to the lateral pressure of the concrete.

In addition, the crane registrations (which were obtained after the analysis) showed that, at the onset of the second increase of the hydraulic head, the casing was still under the clay layer at the depth of the deep sand layer. This disproves the hypothesis as the outflow of concrete was certainly not at the depth of the clay layer and thus this mechanism cannot have caused the start of the increase of the hydraulic head. In relation to this, the axisymmetric model showed that the influence on the water pressure in the intermediate sand layer increased if the concrete pressure approached the sand layer. Vice versa, the effect on the water pressure decreases with increasing casing depth and thus the effect of the outflow of the concrete at the depth of the deep sand layer would have negligible influence on the water pressure in the intermediate sand layer. Hence it is concluded that the outflow of concrete and subsequent lateral pressure could not have caused the second increase of the hydraulic head during the withdrawal process. This mechanism can only add up to the increasing hydraulic head at the moment that the casing was withdrawn from the intermediate sand layer.

Whether the strains and subsequent displacements were modelled correctly can be substantiated when the outflow of concrete is regarded as pure cavity expansion. In literature, cavity expansion models often assume plane strain conditions ($\varepsilon_{zz} = 0$) hence the radial strains would equal the circumferential strains (Randolph et al., 1979 & Baguelín et al., 1978). This was also found in the axisymmetric Plaxis model. Additionally, the clay layer showed an upward displaced field at larger radial distance which was expected beforehand as the volume displacement following from the concrete pressure would cause such a displaced field. In comparison to a plane strain model, the upward displacements were very small but this is again substantiated by the cavity expansion theory.
The assumption of a hydrostatic concrete pressure, modeled by means of user-defined water conditions with a larger volumetric weight, is a conservative estimation as it is the largest possible lateral concrete pressure. This should subsequently result in the largest possible strains, displacements and stress increments. This suggests that the results presented could be even smaller as the concrete pressure will presumably be lower. A non-hydrostatic pressure distribution can be implemented by means of a distributed load.
Finally, an attempt was made to model the fresh concrete as a soil volume with adjusted soil parameters. But the right parameters were not obtained and so this option was discarded. Further research would be necessary to be able to describe fresh concrete behaviour by means of soil parameters. A conclusion that followed from this quick research was that the soil friction angle, ϕ', cannot be changed in a later phase compared to initial soil conditions, as this leads to failure (MC model). But the Young's Modulus , E, could be decreased significantly without problems. Changing the volumetric weight of a soil volume to 24 kN/m^3 did lead to excess pore pressures in the concrete column, which might be assumed correct as the water pressure in the fresh concrete will probably have a larger gradient due to the larger volumetric weight of concrete.

4.3.3.2 Conclusion

The analysis to the influence of the outflow of concrete on the surrounding soil could not substantiate the large second increase of the hydraulic head. Together with the other possible causes no conclusive mechanism was found that can explain the cause of the second increase. It can at least be concluded that the outflow of the concrete cannot have caused the increase as the start of the increase of the hydraulic head does not correspond to the depth of the concrete outflow. In addition, the crane registrations did not show an unique link to the trend of the hydraulic head measurements.

4.4 Equilibrium Analysis between Concrete and Water Pressure

At this moment only the pore water pressures are discussed. In this analysis it was found that a large part of the excess pore water pressures following from the installation of the piles was dissipated before the casing was withdrawn. Hence this could not have influenced the concrete-soil interaction of that pile. Then, it was found that during withdrawal again excess pore water pressures were generated, which had a cumulative effect on the hydraulic head. These excess pore water pressures can influence the interaction between the fresh concrete and surrounding soil. In this paragraph an equilibrium analysis is implemented to see how the external water pressure relate to the concrete pressure.

4.4.1 Water pressure in the surrounding soil

From the measurements of the hydraulic head in the intermediate sand layer the water pressure can be determined. The crane registrations showed that the measured increment of the hydraulic head started before the casing reached the intermediate sand layer and quickly started to dissipate after the casing passed that layer. So the highest and most disadvantageous hydraulic head is the largest value measured, as this would result in the largest water pressure on the fresh concrete. Therefore the maximum value of the hydraulic head is assumed in this equilibrium analysis. This value quickly dissipated and a less critical situation arises, but the small time interval between the interaction of the outflow of fresh concrete and the current water pressure might trigger the intrusion into the fresh concrete. The measured maximum hydraulic head is converted to a water pressure (multiplied with y_w=10kPa) and listed in Table 9 in the column P_{water}. It must be noted that the tabulated water pressures are point values at the location of the measurement devices. Therefore, it is likely that the water pressure next to the casing is even larger.

4.4.2 Concrete Pressure

In contrast to the extensive measurements of the hydraulic head, no information was available on the concrete pressure. Hence the concrete lateral pressure is based on literature on formwork pressure. The lateral pressure of concrete is determined primarily by several or all of the following factors (Oberlender & Peurifoy, 2010):

- Rate of placing concrete in forms
- Temperature of concrete
- Weight or density of concrete
- Cement type or blend used in concrete
- Method of consolidating the concrete
- Method of placement of the concrete
- Depth of placement
- Height of form

The American Concrete Institute (ACI) as well as the construction industry research and information association (CIRIA) present formulas that incorporate some of these effects which are well known to determine form work pressure. But the construction of cast-in-situ piles is somewhat different as the concrete will be disturbed again after it was placed when the casing is withdrawn. Therefore, the thixotropic behaviour of the concrete will also play a role. Additionally, the rate of placing is very large and the piles have large lengths compared to the construction of walls or columns mostly constructed with form works. The amount of reinforcement and the rate of withdrawal together with the concrete rheological behaviour will also influence the outflow of concrete thus on the concrete pressure. None of these later effects are incorporated in the design formulas of formwork pressure and limited information on the items listed above was available. In the design formulas the smallest value of the concrete pressure is normative. But to be able to make a comparison between the water pressure and the concrete pressure a hydrostatic concrete pressure is assumed. Note that this will be a rough estimation of the true concrete pressure, as this is the largest possible value.

When considering a hydrostatic concrete pressure, the concrete lateral pressure is determined by the volumetric weight of concrete times the depth ($\gamma_c h$). The mid depth of the intermediate sand layer is approximately -7.5m NAP which is -3.5m and -4m below ground level for Test Field 1 and Test Field 2, respectively. If no additional concrete head is taken into account and the volumetric weight of concrete is assumed to be 24kN/m^3 this will result in a concrete pressure of 84kPa and 96kPa, respectively, see Table 9. Note that the concrete pressure at the moment that the fresh concrete gets in contact with the surrounding soil and water pressure at the depth of the intermediate sand layer will depend on the additional concrete head inside the casing and the possibility of the outflow of concrete. The latter will be influenced by several factors including but not limited to the concrete rheology, the time between placement and withdrawal, the placement procedure indicated by the factors listed above, the amount of reinforcement and the rate of withdrawal.

It is highly plausible that the concrete pressure deviates from a hydrostatic pressure distribution as is illustrated in Figure 4.22. The figure shows the broad variation in results in terms of relationship between the casting rate and the form pressure relative to the hydrostatic pressure of different papers discussing formwork pressure from placing self-compacting concrete (SCC). The most surprising results are those of casting rates approximately from 20 m/h and upwards showing pressures below 70 %. As a high casting rate, which is also found for the construction of the cast-in-situ concrete piles, should approximate hydrostatic pressures. These examples show that assuming full hydrostatic pressure would be conservative and a reduction in the concrete pressure that would result in equal or even lower concrete pressure compared to the measured water pressure is not uncommon. Note that the illustrated form pressures are an indication after placement and do not consider the effects of withdrawal which can result in even lower lateral pressures.

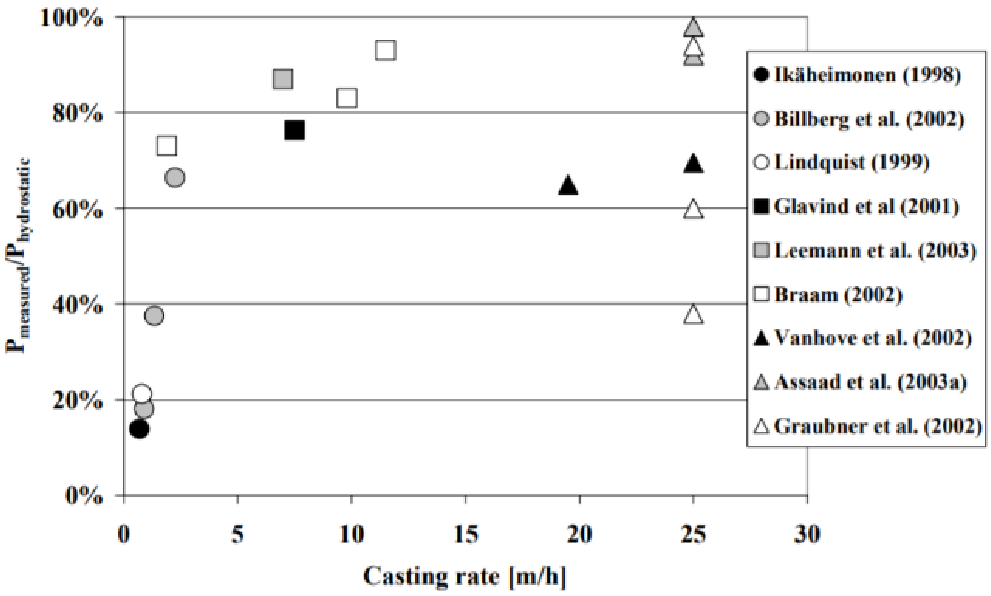


Figure 4.22: Examples of reported form pressure when using SCC (From Billberg, 2006)

4.4.3 Pressure Equilibrium

Table 9 present the measured water pressure at the radial distance from the pile and the assumed hydrostatic concrete pressure. The last column tabulates the ratio between the measured water pressure and the assumed hydrostatic lateral concrete pressure. All ratios are below 100% indicating a larger concrete pressure than water pressure which would suggest that the water would not be able to penetrate the concrete. But taking into account the assumptions discussed in Paragraph 4.4.1 and Paragraph 4.4.2, it cannot be excluded that the water pressure close to the pile is larger than the measured values and the lateral concrete pressure way lower than the assumed hydrostatic pressure. Therefore, it is not possible to conclude that the water pressure will not be larger than the concrete pressure and hence will not tend to penetrate the fresh concrete.

Additionally, the hydraulic head, Φ, is considerably higher than ground level. If a possible channel is formed during the interaction of the outflow of the fresh concrete and the surrounding soil, the water will take this path as the water will chose the path of least resistance. The possibility of a preferential path or channel through the fresh concrete pile will further be elaborated in Chapter 5.

Table 9 Measured water pressure versus assumed hydrostatic concrete pressure

Pile	r [m]	Φ [m NAP]	P_{water} [kPa]	$P_{concrete}$ [kPa]	P_{water}/ $P_{concrete}$ [%]
Test Field 1					
115	1.4	-1.86	56.4	84	67.1
	2.7	-2.66	48.4	84	57.6
319	1.2	-0.85	66.5	84	79.2
	1.3	-1.35	61.5	84	73.2
Test Field 2					
421	1.4	-0.98	65.2	96	67.9
252	1.1	-0.26	72.4	96	75.4
	1.1	-0.81	66.9	96	69.7
57	1.0	-0.81	66.9	96	69.7
	1.2	-1.27	62.3	96	64.9
246	1.1	-2.98	45.2	96	47.1
417	1.3	-1.96	55.4	96	57.7

4.5 Influencing Factors and Mitigating Measures

As introduced in Chapter 2, soil displacement pile systems will cause excess pore water pressures in subsequent soil strata. In this chapter the focus was on an aquifer system affected by the installation of a soil displacement pile with lost tip (i.e. Fundex pile system) where the different processes that causes an increase in the pore water pressure around a pile were discussed. To conclude Chapter 4, the factors that influences the generation of the excess pore water pressures will be discussed herein and where possible mitigating measures will be presented that can limit the effect on the fresh concrete.

Firstly, the generation and subsequent dissipation of the excess pore water pressure depend on the soil stratigraphy encountered at the project location. In this case study it was found that large excess pore water pressures were measured in a thin intermediate sand layer confined by clay layers. This showed that such alternating soft soil layers might trigger large increases in the hydraulic head caused by volume displacement resulting from the installation of displacement piles. The transmissivity and storativity of the intermediate sand layer have large influence on the accumulation and dissipation of the excess pore water pressure. Therefore, it is advisable to put more focus on the encountered soil stratigraphy and to measure the influence of the pile installation on the water pressure in susceptible soil layers in an early project phase by means of test piles for example. Following from the pressure equilibrium analysis the depth of the susceptible soil layers plays also a role in the interaction between the water pressure on the fresh concrete column as the concrete pressure increases with depth.

Secondly, a very large increase in the hydraulic head was measured during the descending phase of the pile. The main cause of this increase was assigned to the volume displacement of the pile. Within the geohydrological model the calculated hydraulic head depends on the discharge put in the system, which in turn depends on the volume of the pile, and the time that is needed to penetrate the soil layer. The main factors that influence this discharge are the diameter of the pile and the rate of installation. But it is hard to change these parameters as the diameter of the pile is important for the bearing capacity of the foundation and a smaller diameter would result in the installation of more or longer piles which would negatively influence the necessary time to install the complete foundation. On this note, decreasing the rate of installation would also ensue longer installation times, although it would only be necessary to decrease the rate at the depth of the susceptible soil layers. In this project location the intermediate sand layer was thin (i.e. ≈1m thick) and a doubling of the time to penetrate the layer had negligible influence on the modelled excess pore water pressure. In addition, the increase in hydraulic head had a large radial extent. This is not elaborated on in detail, but it can be concluded that a previously installed pile at a limited radial distance can be affected by the additional increase in hydraulic head. Therefore, also the pile spacing and installation sequence should be taken into account when considering bleeding potential. Increasing the radial distance between subsequent piles or adjust the pile sequence to different regions within the project area will limit this influence, but will increase the movements of the piling machine which can also have influence on fresh piles that have not reached sufficient strength.

Thirdly, the water pressure in the intermediate sand layer increased again after the start of the withdrawal of the casing. A cumulative effect was found in the measurements of the hydraulic head as not all of the excess pore water pressure was dissipated after descending phase. This will fully depend on the time between the descending and ascending phase in which the reinforcement cage is placed and the concrete poured. Again, considering the installation time this will probably be kept to a minimum based on practical reasoning.

The immediate increase in hydraulic head after the start of withdrawal was appointed to the reaction force of the machine on the ground. Influencing factors on this behalf depend on the necessary pull up force. This will be influenced by the pile length, diameter of the pile, soil stratigraphy, rheology of the concrete, the presence of a bentonite or grout cake and the time between reaching the final depth and the start of

withdrawal. In other words, it will depend on the shear forces and weight of the casing that the machine has to overcome. But will also depend on the soil conditions and depth of the susceptible soil layer. Additionally, the rate of withdrawal might also influence the hydraulic head but this should be investigated further. This could be controlled by the driller ('boormeester').
As no conclusive mechanism was found that caused the second increase in hydraulic head during withdrawal no influencing factors or mitigating measures can be presented.

Finally, when focusing on the excess pore water pressures only, either the amount of the excess pore water pressure should be limited or the possibility of dissipation should be increased. Most of the aspects that affect the increase in hydraulic head are discussed and it can be concluded that it will be beneficial to focus on the encountered soil stratigraphy to see if susceptible soil layers are present. If these are present it would be wisely to control how the soil reacts to the installation of the piles and test the hydrological behaviour of susceptible layers. During the testing the crane registrations should be closely monitored to see if adjustments can be made in the installation procedure that decrease the amount of excess pore water pressures.
Next to the generation of the excess pore water pressure its dissipation is also very influential. The dissipation of water pressure in a soil layer can only be enhanced by decreasing the drainage path length. This can be done by means of vertical or horizontal drains or pumping of that layer. These are drastic measures and hard to apply as the increase in hydraulic head is very quick, the generation is only locally around the piles and has limited radial influence compared to the application of pumping. In the considered project vertical drains were applied in Test Field 2 next to the installed piles, but these did not have any effect on the accumulated hydraulic head and no flow of water was observed through the vertical drains. This was assigned by the reaction time of the vertical drains which was larger than that of the aquifer system.

5. Mechanism of (Excessive) Bleeding

In the preceding chapter the causes of excess pore water pressure generated in the soil were discussed. It was found that considerable water pressures are generated, but how these water pressures influence the bleeding phenomena observed in practice is further elaborated in this chapter. By starting with a literature review it can be seen which models relating to bleeding exist and on which kind of processes they are based. It is tried to provide an overview addressing the different types of bleeding, such as internal, external and channelled bleeding. Conditions are found that substantiate the different kind of bleeding processes. Finally, it is tried to relate these mechanisms to problems observed in practice.

5.1 Literature Review

First of all, an overview of the literature on self-weight bleeding resulting from gravitational forces is given in Paragraph 5.1.1. All these models were only based on normal bleeding while this research focusses on the mechanism of excessive bleeding resulting in channels observed in practice. Therefore, Paragraph 5.1.2 describes the literature that has been focusing on this phenomenon and a postulation was found that defines the initiation of channelling. Then, in Paragraph 5.1.3 some literature is presented addressing fresh concrete tested under pressure, which take into account the pressure situation in fresh concrete in deep foundation piles. Finally, concluding remarks on the literature are given that put the current knowledge in perspective to the specific problem of excessive bleeding that is of interest.

5.1.1 Self-weight Bleeding

Bleeding has been the subject of considerable research efforts and was first seen as a sedimentation process by Powers (1939). Many years later bleeding was researched by Tan, Wee, Tan, Tam, and Lee (1987) who suggested that the bleeding phenomenon was governed by the mechanics of self-weight consolidation and not sedimentation as previously thought. Their concept is based on the premise that in practice the concentration of cement paste is high and the cement particles are in close contact with each other, thus suggesting the presence of interparticle forces. Analogous to soil, these interparticle forces can be considered as the effective stress in the paste, and bleeding can be considered as a process of self-weight consolidation.

Tan et al. (1987) stated that the accumulation of water at the surface of a freshly mixed cement paste, mortar or concrete after some time is known as bleeding which is the result of a complicated interaction between gravitational forces, interparticle forces and other physical-chemical forces. They also made a distinction between “normal bleeding” as the accumulation takes place gradually and “channelled bleeding” if the water accumulates through the formation of channels. In their paper a self-weight consolidation model was proposed that only deals with the normal bleeding of cement paste and neglects hydration although they concluded that it would be necessary to account for this. In a further study, Tan, Loh, Yong, and Wee (1997) incorporated the effect of hydration in the small-strain model presented earlier by allowing the compressibility to change with time. By means of this new model Tan et al. (1997) studied cement pastes as well as mortar specimens and found that the model was able to predict the bleeding process for both types of specimen.

In another research, Clear and Bonner (1988) also found that mathematical relationships derived for the analysis of soils apply for fresh concrete. They did not consider hydration in their research as negligible cement hydration took place in the short time period of testing. They concluded that a relationship exists between the settlement of fresh concrete and effective stress but differentiate between two components of settlement. The first component results from immediate settlement due to compression of the pore fluid and the intergranular structure in response to the applied load and the second component is from the consequence of consolidation settlement created by the dissipation of pore water with time.

Further research on the hydration effect on concrete bleeding was done by Josserand, Coussy, and de Larrard (2006). They improved the self-weight consolidation model of Tan et al. (1987) by considering finite transformations, as was done by Tan et al. (1997). But Josserand et al. (2006) used an ageing function to account for the stiffening of the soil skeleton resulting from the hydration process before the actual setting of concrete in their numerical model.

Morris and Dux (2010) presented new analytical finite- and small-strain solutions for the bleeding of cement pastes, cement mortar and concrete that account for the effects of hydration and material setting. Morris and Dux (2010) state that their model is simpler to implement than the comparable numerical solutions of Tan et al. (1997) and Josserand et al. (2006). But to enable the model to apply for the prediction of bleeding in large structures, like concrete piles for deep foundation of interest in this study, it will be necessary to develop quick tests or correlations for several mix- and pour-specific parameters that are required as input data.

To conclude the literature study on self-weight bleeding phenomenon Yim, Kim, Kwak, and Kim (2013) distinguish between internal and external bleeding. In their paper a comprehensive model based on small-strain theory of linear poroelasticity was proposed in order to understand and predict the different bleeding phenomena. Yim et al. (2013) concluded that an excessive amount of internal bleeding decreases the strength and durability of hardened concrete.

All models described above only concentrated on the rate or amount of accumulated bleeding water as the outcome of self-weight consolidation testing. The models are confined to the study of normal bleeding and do not consider channelled bleeding or excessive bleeding. Hence the models cannot describe the mechanism of excessive bleeding that results in the problems observed in practice. Therefore, the following paragraph focusses on literature that research channelled bleeding.

5.1.2 Channelled Bleeding

As described in the previous paragraph a distinction can be made between normal bleeding and channelled bleeding. Under certain conditions during the normal bleeding process, channelling in which some localized channels carrying water to the surface may develop (Tan et al., 1997). When this happens, the bleeding rate will increase significantly, see Figure 5.1, and the channels formed will remain as conduits in the hardened concrete.

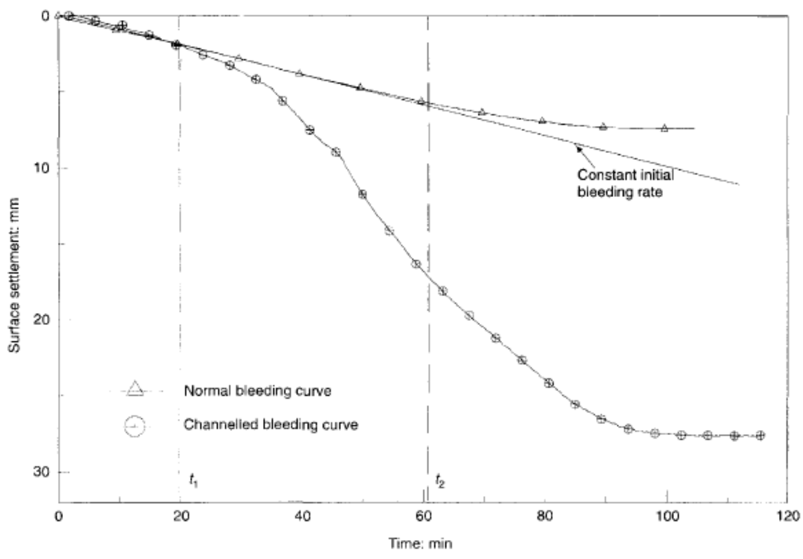


Figure 5.1: Typical Bleeding Curve (From Tan et al., 1997)

Loh, Tan, Young, and Wee (1998) investigated the bleeding process including channelling of cement paste and mortars. From experimental tests Loh et al. (1998) concluded that channelling is controlled by the mix proportion (water-cement ratio) of a mixture and a specific 'minimum' height and that channelled bleeding occurs if there is negligible effective stress in the cement-based material. From these results, Loh et al. (1998) stated that the difference between 'normal bleeding' and 'channelled bleeding' is the presence of an effective stress in the concrete mix hence normal bleeding can be described by consolidation processes as discussed in Paragraph 5.1.1 and channelled bleeding by sedimentation processes. Furthermore, they proposed a postulation on channelling, in which they stated that channelling can be visualized as a phenomenon where the upward hydrodynamic forces in a plug exceed the downward forces, allowing the formation of a channel. The postulation of Loh et al. (1998) was used by Kog (2009a) to test if this postulation agrees with channelling that was found in a project where concrete bored piles were installed. Kog (2009a) found that the postulation agreed with the integrity tests based on the premise that the occurrence of bleeding and channeling is more pronounced in the larger diameter bored piles. In the analysis the water pressure in the surrounding soil was not taken into account and the postulation is only based on the diameter of the piles and concrete properties. An interesting finding of Kog (2009a), following from integrity tests, was that bleeding was not confined to the upper portion of the piles only and channels were found at various depths in the piles.

5.1.3 Pressurized Bleeding

Most of the above presented models were validated by self-weight bleeding tests on specimens of limited height. But the length of construction piles is much larger than the length of the test samples hence the pressure gradients are also different and other mechanisms may play an important role. This was also concluded by Yim, Kim, and Kwak (2014) that tested fresh cement-based materials in a pressure vessel and found that bleeding of cement-based material showed different phenomena under pressure. Yim et al. (2014) related the total amount and the rate of bleeding to the mixture's oedometric modulus and diffusivity, respectively. Vanhove and Khayat (2016) also used a pressure vessel to differentiate between different stability levels of flowable concrete and lean concrete that are prone to bleeding and segregation. Concrete mixtures were tested on different overhead pressures and it was concluded that the initial rate of bleeding at 1 minute for the lowest overhead pressure compared well with the final external bleeding in a self-weight consolidation test. In addition, the lowest overhead pressure (138 kPa) showed the most deviation of the rate of bleeding at 1 minute for different mixtures and therefore provided an adequate comparison basis for the quantification of the stability of concrete. Based on a comparison with the segregation index and the homogeneity index of the different concrete mixtures, Vanhove and Khayat (2016) concluded that the values of the initial rates of forced bleeding at 1 minute less than 8, 8 to 11, and greater than 11mL/min, for an overhead pressure of 138 kPa, can be considered to correspond to relatively high, medium, and low stability levels, respectively.

For comparison, guide values have been found for the Bauer Concrete Filter Press tests (EFFC/DFI, 2018) for fresh concrete stability of special deep-casting concrete. Where values of ≤22ml, ≤40ml and ≤60ml correspond to high requirements, medium requirements and low requirements of the amount of filtration water, respectively.

Yim et al. (2014) and Vanhove and Khayat (2016) showed that the total amount of bleeding water was larger for the concrete mixtures tested in a pressure vessel compared to the amount of bleeding from a self-weight consolidation test. This clearly indicates that the extraction of water from the concrete under a certain applied pressure is determined by filtration and not by self-weight bleeding. This mechanism is also expected in fresh concrete in deep foundations. Neither of the two above mentioned tests focused on the possible occurrence of channelling. And the measurements of a pressure vessel are restricted to the amount of extracted water, the rate of bleeding and the produced filter cake which restricts the outflow of water (Vanhove and Khayat, 2016).

Finally, Larisch (2019) described the mechanism of excessive bleeding or channelling in deep foundations with a 'modified' Terzaghi's theory of one-dimensional consolidation. The excess pore pressures in the concrete are due to the self-weight of the fresh concrete column (filtration). The difference between Terzaghi's theory and the modified application to model the behaviour of fresh concrete in deep foundations is cement hydration. Additionally, Terzaghi's one dimensional consolidation theory assumes that one-dimensional, external loading is applied vertically onto a fully saturated soil sample, and that horizontal confinement is given. The latter suggests that this model can only be used if horizontal drainage is prohibited such as in piles that are installed with a steel casing, if a filter cake is present (e.g. from grout) or in impermeable ground conditions. In the application of the modified model, the excess pore water pressure can be released by pushing some fluid out of the system (e.g. bleed water or cement paste) if a drainage path is unlocked. If no drainage path is available, no bleeding under pressure will occur, and the excess pore water pressure will be converted into effective stresses as the cement will begin to hydrate and set (Larisch, 2019). Possible drainage paths are pile reinforcement or steel liners (casing). In large diameter bored piles or diagram walls concreted with a tremie pipe it is also found that the center of the pile acts as a potential flow path. The excess pore water pressure inside the cement paste is the starting point of this excessive bleeding model. Besides that, the occurrence of excessive bleeding will depend on the workability and the stability which are governed by the aggregate grading, shape and form, the fines content, the optimal water content and chemical admixtures used in the concrete mix. The most important recommendations following from Larisch (2019) on this matter are the importance of a minimum of 25% of fines passing the combined grading curve at 600 microns and the description of the optimal water content. The optimal water content was determined by testing the fresh concrete's stability, assessed by the filtration press test, of different concrete mixes with varying water contents. It appears that each mix had an optimal water content for the given aggregate configuration, and when water was added above the optimal water content, it was almost completely pushed out during the filtration test. Finally, Larisch (2019) also related excessive bleeding to a sand boiling analogy. As the mouth of a bleeding channel in the center of a pile has an appearance which is comparable to that of a sand volcano.

5.1.4 Conclusion Literature Review

From a geotechnical perspective it was interesting to see that several researchers have concluded that self-weight bleeding of cement-based materials (i.e. cement paste, mortar or concrete) can be described with a consolidation model. The self-weight bleeding models were only tested for small size specimens but Larisch (2019) used a consolidation model to describe the mechanism of excessive bleeding in deep foundation piles under pressure of its own weight.

On the other hand, an important finding is that very limited literature has been found that focused on channels that are formed during bleeding especially in relation to (cast-in-situ) concrete piles. And no literature has been found that relate excessive bleeding to external water pressure in the surrounding soil. But following from the literature on channelled bleeding, two important aspects are found. Firstly, Loh et al. (1998) described that channelling can happen inside a cement-based material within a testing environment and thus unaffected by external sources of water, indicating that channelled bleeding can start within the concrete mixture itself. Secondly, Kog (2009a) found that channels formed by bleeding was not only confined to the upper part of the piles and channels were detected at different depths in the pile. None of the above-mentioned models or tests can satisfactorily describe the excessive bleeding phenomenon. This might be explained by the fact that excessive bleeding increased by external pore water pressure is not of importance to the scientific world of concrete technology and plays only a limited but not less important role in the production of cast-in-situ piles.

Some water percolation from the pile head can be expected in deep foundations, mostly along the reinforcement bars or at the perimeter of the pile but sometimes large water flows are noticed forming channels and remain as conduits in the hardened concrete. The only model that focused on channelled bleeding was found in the postulation formed by Loh et al. (1998). Kog (2009a) tested this postulation which seemed to correspond well with the anomalies found in a project with bored piles of different diameters. The model states that the occurrence of channelling will only be dependent on the diameter of the pile and the yield diameter which in turn is determined by the fresh concrete yield stress, the weight of the concrete constituents and the solid volume concentration. Assuming that the yield diameter would be constant for the concrete used in a project, it would suggest that channelling would always occur if the pile diameter is larger than the yield diameter or vice versa.

5.2 Influence of Excessive Bleeding on Integrity

It is not uncommon to notice a certain amount of bleeding at the top of a freshly concreted pile. This will be due to bleeding or to filtration. Expel of water from the mixture is to some degree to stiffen the concrete i.e. to change the rheological properties to higher yield stress and higher viscosity (EFFC/DFI, 2018). Most of the water loss from filtration will not be observed as it radially flows into the surrounding soil if the soil is permeable enough. But if the soil is impermeable, a filter cake is formed by grout that was injected during installation or a casing is present, restricting the loss of water into the soil, than the water will find another way out. This might result in water percolation at the shaft surface or vertically in the pile if insufficiently stable concrete mixes are used. This can lead to the formation of flow channels, which also allow fines to be flushed out of the concrete. If the amount and time of water percolation is limited it normally will not have a large influence and the effect of bleeding is assumed to be negligible in practice. However, if the flow velocity in the channels becomes large and the duration of flow is long which is referred to as excessive bleeding in this report, channels will form that eventually can result in conduits of considerable diameter (≈50mm) in the hardened concrete that will certainly impair the durability of the pile, see Figure 5.2 for example. Other effects of bleeding will be discussed below.

Figure 5.2: Conduit in hardened concrete resulting from excessive bleeding ($D_{conduit}$≈30mm)

5.2.1 Effect on the Durability

If excessive water is trapped within the mixture it will probably be captured under and alongside coarse aggregate particles or underneath reinforcement bars. This is prone to happen when differential settlement occurs between the aggregates and the paste and when the concrete settles leaving a void underneath the reinforcement which is easily accessible for water (Kosmatka, 2006). Space occupied by excessive internal bleeding can permanently remain as porosity in a material, even though the space is partially filled by cement hydration. The excessive porosity due to internal bleeding weakens the interfacial transition zone around aggregates and the bonding strength of reinforcing bars, this can also promote corrosion of the steel. Excessive internal bleeding thus causes deterioration of durability and strength performance (Yim et al., 2013).
In addition, excessive bleeding of fresh concrete in deep foundation is often recognized by water flowing out of the pile head. Due to the large pressures occurring in deep foundation, fines and cement paste can be washed out, resulting in voids or channels. The effect on the durability will depend on the severity of the flow and the drainage path of the water. If the water flows through the center of the pile, the channel in the hardened concrete will have limited influence on the durability, see Figure 5.2. But when reinforcement bars act as drainage paths, the water flow impairs the steel-paste which has a clear effect on the durability, this is illustrated in Figure 5.3.

Figure 5.3: Bleed water travelling upwards along reinforcement bars (From Larisch, 2019)

5.2.2 Effect on the Strength

Considering the effect of bleeding on the strength, Giacco and Giovambattista (1986) and Loh et al. (1998) found that the compressive strength of test columns of concrete and mortar, respectively, increases with depth and the difference between the top and the bottom parts reached up to 30%. The density profiles were found to increase also with depth. The density profile is determined by the stability of the concrete which in turn is influenced by bleeding, but also by filtration and static segregation (EFFC/DFI, 2018). But it indicates that concrete stability can be very influential and thus (excessive) bleeding should be limited where possible.

To assess the local effect of channels, the compressive strength of cored samples with and without channels were compared by Loh et al. (1998). The results showed that the compressive strength was not significantly affected by the presence of channels. This was argued by the fact that the surface area of the channels are very small compared to the overall area of the columns. This is supported by Kog (2009b) who concluded that the effect of channelling on the axial capacity and compressibility of an affected concrete pile was negligible. This was tested by means of a pile load test on a large diameter pile where channelling was detected.

5.3 Influencing Factors and Mitigating Measures

As the mechanism that causes excessive bleeding found in practice is still not fully understood no conclusive mitigating measures can be presented. Therefore, the influencing factors and possible mitigating measures that were found in the literature study are limited to bleeding of cement-based materials and channelling in general. Together with the overview that was provided in Paragraph 4.5 on the mitigating measures related to excess pore water pressure in the soil this should give some guidance on the possibilities to control (excessive) bleeding.

As a starting point it can be stated that a stable concrete mixture would limit the outflow of water from the concrete. EFFC/DFI (2018) summarizes some of the influencing factors of concrete mix design as follows: “The extent to which bleeding will occur in deep foundations depends on many factors including, but not limited to, the water to fines content, the aggregate particle size distribution, the efficiency of admixtures over time, the total concrete height and the time when the concrete reaches final consolidation.” A lot of research can be found on the mix proportions and admixture to limit the potential of bleeding and it is therefore not further discussed in this paragraph. Some, but surely not limited, literature on concrete mix design relating to bleeding can be found in Powers (1939), Neville (2011), Kosmatka (2006) and EFFC/DFI (2018).

Following from the postulation of Loh et al. (1998) channelling would be hindered by increasing the plastic yield stress of the fresh concrete or decreasing the pile diameter. Kog (2009a) found that channelling can occur at different depths within a pile and is thus not only limited to the upper part of the pile. This suggest that channelled bleeding will therefore not always be visible at the pile head. Additionally, internal bleeding can have adverse effects on the durability and strength of the concrete and so it can be concluded that a stable concrete mixture is very important.

When considering the consolidation theory for bleeding under high pressure the dissipation of the excess pore water pressure inside the concrete depends on the possible drainage paths. Such drainage paths are reinforcement cages and steel liners. Excess bleed water can also escape sideways through the soil formation if drainage is possible. A permanent casing, bentonite-or grout cakes or impermeable soil layers would hinder this horizontal dissipation. The specific influence of these limiting boundary conditions should further be researched. Excessive bleeding was also found to be concentrated in the center of the pile. This can be explained by an analogy of a pile in a pile. As the water in the center of the pile has to travel further to dissipate radially into the soil, it will be in a more fluid state than the outer concrete. Additionally, the reinforcement cage in this outer part locally increases the shear strength of the material. So if a connection is formed between the surrounding ground and this inner more permeable concrete, the water flow will be concentrated here.

“The existence of potential drainage paths, the lack of fines in the fresh concrete matrix in combination with insufficient aggregate grading and the addition of too much design water (above the optimal water content for a given aggregate combination) have been identified as key factors contributing to concrete bleeding and channelling in deep foundations (e.g. bored piles and diaphragm walls)” (Larisch, 2019).

From a pressure equilibrium point of view, the concrete pressure should be increased to prevent external ground water to penetrate into the fresh concrete. Therefore, one of the mitigating measures would be to increase the concrete pressure. When considering a hydrostatic pressure distribution, the concrete pressure is determined by the total height of the concrete and its volumetric weight. Thus an additional concrete head inside the casing or a high volumetric weight of the concrete will be beneficial. On the other hand, the concrete pressure will be influenced by the possibility to flow out of the casing, this can be promoted by decreasing the rate of withdrawal, retarders and a lower viscosity of the concrete and is negatively influenced by longer waiting times between concrete pour and withdrawal (thixotropy) and dense reinforcements.

6. Conclusion and Recommendations

In this chapter the conclusion and recommendations will be discussed. Firstly, the final conclusions of the different analyses will be presented and thereafter the recommendations following from this research.

6.1 Conclusion

From the problem overview in Chapter 3 it was found that little research is available on the mechanisms that can cause problems with cast-in-situ concrete piles. Therefore, more research is necessary to the installation effects of the different piling systems on the surrounding soil. More quantitative descriptions of soils and soil properties are necessary to create knowledge on soils that are susceptible to certain problems and develop possible mitigating measures for these problems. Especially, the latter misses in current guidelines. An example of such an overview was presented in Table 1, where problems were linked to susceptible soil layers and possible measures. This table is limited to the information that was gathered, but this could be extended with more project data, measurements and future research. When focusing on the soil conditions, the overview of the problems showed that specific attention should be paid if any of the following soil conditions are encountered;

1. Very soft soil layers at shallow depth;
    a. Undrained shear strength, $c_u < 25$ kPa
    b. Cone resistance, $q_c < 0.5$ MPa
2. Artesian conditions;
    a. When auger piles are considered
    b. When pile construction level is significantly below ground level
    c. When the difference in hydraulic head is larger than approximately 1m
3. Strongly variable and layered soil stratigraphy and;
4. Confined sand layers at shallow depth.

Additionally, it can be concluded that a risk analysis on the possible problems should be incorporated in the design phase when considering cast-in-situ concrete piles. This will help to focus on the practicability of the pile system and to prevent the occurrence of problems.

When looking at the hydraulic head, in an intermediate confined sand layer, during the installation of displacement piles, all measurements showed a consistent trend. During the descending phase a large increase in the hydraulic head was measured which can be described by volume displacement in the sand layer. In addition, the hydraulic head increased again during withdrawal of the temporary casing which was unexpected and a new finding in this thesis.
The generation of excess pore water pressure during the descending phase of the installation was modelled with a geohydrological model for unsteady state flow in a leaky aquifer system. It was found that the model was able to describe the generation and subsequent dissipation quantitatively if the soil parameters were chosen correctly. But it must be concluded that the model is not able to adjust to the different reaction over radial distance for a fixed set of aquifer parameters, which is often assumed in geohydrology. In addition, the model is very sensitive to the storativity of the aquifer.
Another important finding was the dissipation of the excess pore water pressure before the withdrawal of the casing. This suggests that excessive bleeding cannot be caused by the excess pore water pressure generated during the descending phase only. But it is not excluded that this increase can influence previously installed piles, that have not reached sufficient strength, within the recommended radial distance set by the guidelines. As the measurements showed significant water level increases at radial distances up to 10 times the pile diameter. The radial extent of the excess pore water pressure in relation to the minimum recommended radial distance set by the guidelines could be interesting for further research.

A cumulative effect in the hydraulic head was found at the start of withdrawal of the temporary casing as the excess pore water pressure, resulting from the volume displacement, was not fully dissipated. During withdrawal, two additional pore water pressure increments can be distinguished.
The direct increase starting from the onset of withdrawal has been interpreted as being caused by the reaction force of the machine on the ground resulting from the pull up force on the casing necessary to get the casing moving. The rate of withdrawal and the weight of the casing itself might have an additional influence on the force of the piling equipment.
Unfortunately, no conclusive cause was found for the second increase in hydraulic head regarding the processes that take place during withdrawal. The analysis to the influence of the outflow of concrete on the surrounding soil could not describe the large increase found in the measurements. But following from the crane registrations, the depth of the casing at the moment that the hydraulic head started to increase again was far below the intermediate sand layer and thus seems to disprove that this mechanism can cause the increase. In addition, not an unique link was found between the crane registrations and the hydraulic head measurements. And finally, the shear force was neglected assuming the lubricant effect of the grout cake surrounding the casing. This is an unsatisfactory result as the finding of the increase in hydraulic head during withdrawal might be very important in the subsequent interaction between the fresh concrete and the surrounding soil.

From the literature study on (excessive) bleeding it can be concluded that in order to capture the full bleeding phenomenon three processes can be distinguished. Firstly, if the governing process is sedimentation, which is characterized by negligible effective stress in the cement based material, segregation and channelled bleeding can arise. After settling or directly, depending on the stability of the mixture, the governing process in the cement based material is consolidation which means that the excess pore water pressure inside the concrete wants to dissipate, either horizontal (filtration) or vertical (bleeding). The existence of flow paths herein is very important. Finally, cement hydration should be incorporated in the analysis as the bleeding process is also governed by the chemical processes in the cement paste. Literature, however, confirms that the onset of hydration takes approximately 30 to 90 minutes which means that sedimentation and consolidation are the initial mechanisms.
Channelled bleeding has been confirmed in laboratory tests. These tests showed channels of approximately 20 mm diameter depending on specimen height, diameter and mix design. The mechanism of forming channels is some internal instability during sedimentation.
In addition, it is concrete under very high pressures (100-500 kPa) that describes the pressure situation in the fresh concrete piles thus filtration tests or forced bleeding tests are recommended to test the stability of concrete for cast-in-situ concrete piles.
No literature was found that succeeds to fully capture the intrusion of external ground water into the fresh concrete. However it is likely that when a high external ground water pressure is present it will follow the path of least resistance and add up to the water flow through the fresh concrete pile if a connection is found between a drainage path in the fresh concrete and the excess pore water pressure in the surrounding soil.

6.2 Recommendations

From the project documents, guidelines and personal communication with several people with practical experience it would be beneficial if earlier in the design phase attention would be paid to the possible risks of a certain cast-in-situ pile system at a project location. The knowledge of a geotechnical engineer could be valuable to give advice on the risks relating to the soil conditions. This is substantiated by the fact that the CROW, a technology platform for transport, infrastructure and public space, is currently working on a publication about problems occurring with cast-in-situ piles.

The problem overview and mitigating measures presented in Paragraph 3.7 should be extended with more project data and practical solutions to increase the understanding in mitigating measures that can be taken into account in the design phase.

In general, it would be helpful to do more often extensive water pressure measurements as this can give valuable information on the effects of the installation procedure on the surrounding soil, as is illustrated in this report.

To elaborate on the possibilities of the geohydrological model for predicting the amount and radial influence of the excess pore water pressures generated by the volume displacement of water, it is recommended to do field experiments with measurements of the hydraulic head at different radial distances plus geotechnical investigation on the aquifer parameters HPT-tests (Hydraulic Profiling Tool) and MPT's (Mini Pumping Test). Additionally, it is advised to control the measurement frequency in relation to the time that volume displacement is expected as it was seen that it is not possible to substantiate between different processes if the measurement frequency is too low.

The installation of a foundation pile is a continuous process. This is once again illustrated by the measurements of the hydraulic head governed by several processes. First of all, it is recommended to add the installation effects (descending phase) on the surrounding soil before considering the withdrawal process as this will have a large influence on the soil state, this is fully neglected in this study. More advanced computational models, such as the Material Point Model, are able to model such a continuous process and are not limited to small displacements. The installation effects will certainly affect the reaction during withdrawal and the subsequent pressure that the soil will exert on the fresh concrete column.

Further research is necessary to determine the exact cause of the second pore water pressure increase during withdrawal. If possible, other field measurements of the hydraulic head during pile installation should be compared to see if the finding of excess pore water pressure during withdrawal can be expected in other projects or at other pile systems. Otherwise, it is recommended to do more field testing to be able to exclude possible mechanisms and to find the cause of this considerable increase.

It would be interesting to develop mitigating measures. For the excess pore water pressures this would suggest that the dissipation should be promoted. It this case study, plastic vertical drains were installed. These had no effects on the hydraulic head as their reaction or sensitivity was too low. Other drains or pumping could be tested but the applicability of these measures is limited by the large variation in such small intermediate sand layers ('wadzand').

Further research is necessary to asses and study the mechanism of excessive bleeding. Different models are presented but a model based on the consolidation theory seems to be most suitable. If this is tested, the excess pore water pressures in the concrete due to filtration should be incorporated. Permeable and impermeable boundary conditions at the pile circumference can be tested. In addition, it would be interesting to test if concrete can be described by consolidation parameters, unit weight, permeability (void ratio dependent) and volume compressibility. In case the dissipation of the excess water pressures is modelled with a consolidation model the hydraulic gradients, total volume of expelled water and time scale are interesting parameters that can give essential information on the bleeding phenomenon. On the other hand, the formation of the channels may be partly comparable to the piping mechanism. This could also be compared. Finally, it was not clear how the drainage paths or channels inside the fresh concrete link to the external pore water pressure and how these intrude into the concrete. This might possibly be researched in a laboratory where the influence of an external pressure on the fresh concrete can be tested for different boundary conditions (drainage possibilities).

Gathering more information about the bleeding phenomenon would be the final recommendation. Currently, the only information that is reported is whether bleeding occurred or not. Information on the amount of expelled water, the diameter of the channel in the hardened concrete and the height of the water fountain on top of the pile head would give additional information that could be used as a starting point for further analyses. Whether the amount of water comes from the concrete mix or from the surrounding ground water would still be hard to distinguish but may be derived from ground water quality parameters (such as salinity or electrical conductivity).

References

ASTM D5882–16, Standard Test Method for Low Strain Impact Integrity Testing of Deep Foundations, *ASTM International,* West Conshohocken, PA, 2016, www.astm.org

Baguelin, F., Jézéquel, J.F., & Shields, D.H. (1978) The Pressuremeter and Foundation Engineering. *Series on Rock and Soil Mechanics*, *2*(4), Aedermannsdorf, Switzerland: Trans Tech Publications.

Billberg, P. (2006) – Form Pressure Generated by Self-Compacting Concrete – Influence of Thixotropy and Structural Behaviour at Rest (Doctoral dissertation). Royal Institute of Technology, Stockholm, Sweden.

Bot, B. B. (2016). Grondwaterzakboekje Gwz2016. Bot Raadgevend Ingenieur.

Broere, W., & van Tol, A. F. (2006). Modelling the bearing capacity of displacement piles in sand. *Proceedings of the Institution of Civil Engineers - Geotechnical Engineering*, *159*(3), 195–206. https://doi.org/10.1680/geng.2006.159.3.195

Butterfield, R., & Bannerjee, P.K. (1970) The Effect of Porewater Pressures on the Ultimate Bearing Capacity of Driven Piles, *Proceedings of the second Southeast Asian Conference on Soil Engineering*, 385-394.

Carter, J. P., Booker, J. R., & Yeung, S. K. (1986).- Cavity expansion in cohesive frictional soils. *Géotechnique*, *36*(3), 349–358. https://doi.org/10.1680/geot.1986.36.3.349

Carter, J. P., Randolph, M. F., & Wroth, C. P. (1979) – Stress and Pore pressure changes in clay during and after the expansion of a cylindrical cavity. *International journal for numerical and analytical methods in geomechanics,* (3), 305-322.

Clear, C. A., & Bonner, D. G. (1988). Settlement of fresh concrete—an effective stress model. *Magazine of Concrete Research*, *40*(142), 3–12. https://doi.org/10.1680/macr.1988.40.142.3

Collins, I. F., Pender, M. J., & Wang Yan. (1992) – Cavity expansion in sands under drained loading conditions. *International journal for numerical and analytical methods in geomechanics,* (16), 3-23.

Collins, I. F., & Yu, H.S. (1996) – Undrained cavity expansions in critical state soils. *International journal for numerical and analytical methods in geomechanics,* (20), 489-516.

Cooper, H.H., & Jacob, C.E. (1946). A generalized graphical method for evaluating formation constants and summarizing well field history, *Am. Geophys. Union Trans*, *27*, 526-534.

CUR Bouw & Infra. (2009). Aanbeveling 114, Toezicht op de realisatie van paalfunderingen.

Dijkstra, J., Broere, W., & van Tol, A. F. (2008). MODELLING DISPLACEMENT PILE INSTALLATION IN A FINITE ELEMENT METHOD. *Foundations Proceedings of the Second International British Geotechnical Association Conference on Foundations, ICOF 2008*, *2*, 555–564.

Dijkstra, J. (2009). On the Modelling of Pile Installation (Doctoral dissertation). Delft University of Technology, Delft, The Netherlands.

EFFC/DFI. (2018). *Guide to Tremie Concrete for Deep Foundations* (2nd edition). Retrieved from http://www.dfi.org/viewpub.asp?tid=CPF-2015-DRSH-1

El Wakil, A. Z., & Kassim, M. (2010). Bulging as a pile imperfection. *Alexandria Engineering Journal*, *49*(4), 387–391. https://doi.org/10.1016/j.aej.2010.07.004

Fleming, K., Weltman, A., Randolph, M., & Elson, K. (2008). Piling Engineering, Third Edition. New York, USA: Taylor & Francis.

Giaccio, G., & Giovambattista, A. (1986). Bleeding: Evaluation of its effects on concrete behaviour. *Materials and Structures*, *19*(4), 265–271. https://doi.org/10.1007/bf02472109

Hagerty, D.J., & Peck, R.B., (1971). Heave and lateral movements due to pile driving. *ASCE Journal of the Soil Mechanics and Foundations Division. 97(11),* 1513-1532.

Hantush, M.S., & Jacob, C.E. (1955). Non-steady radial flow in an infinite leaky aquifer, *Am. Geophys. Union Trans*.36(1),95-100

Hobbs, N. B. (1957). Unusual Necking of Cast-in-situ Concrete Piles. *Proc. 4th ICSMFE*, *2*, 40–42.

Hoefsloot, F. J. M. (2001). Wateroverspanningen voor het boorfront: een eenvoudig geohydrologisch model. *Geotechniek*, 26-33.

Josserand, L., Coussy, O., & de Larrard, F. (2006). Bleeding of concrete as an ageing consolidation process. *Cement and Concrete Research*, *36*(9), 1603–1608. https://doi.org/10.1016/j.cemconres.2004.10.006

Karandikar, D. V. (2017). Challenges to Quality Control in Bored Cast-In-Situ Piling in Growing Urban Environment. *Indian Geotechnical Journal*, *48*(2), 360–376. https://doi.org/10.1007/s40098-017-0277-z

Kog, Y. C. (2009a). Bleeding and channelling in bored piles. *Magazine of Concrete Research*, *61*(10), 759–765. https://doi.org/10.1680/macr.2008.61.10.759

Kog, Y. C. (2009b). Integrity Problem of Large-Diameter Bored Piles. *Journal of Geotechnical and Geo-environmental Engineering*, *135*(2), 237–245. https://doi.org/10.1061/(asce)1090-0241(2009)135:2(237)

Kosmatka, S. H. (2006). Chapter 12: Bleed Water. *Significance of Tests and Properties of Concrete and Concrete-Making Materials*, 99–122. https://doi.org/10.1520/stp37730s

Kruseman, G. P., & de Ridder, N. A. (1991). Analysis and Evaluation of Pumping Test data. *International Institute for Land Reclamation and Improvement*, Wageningen.

Larisch, M. D. (2019). Fundamental Mechanisms of Concrete Bleeding in Bored Piles. Presented at Concrete 2019, Sydney, Australia. Retrieved from https://www.researchgate.net/publication/337033612

Loh, C.-K., Tan, T.-S., Yong, T.-S., & Wee, T.-H. (1998). An experimental study on bleeding and channelling of cement paste and mortar. *Advances in Cement Research*, 10(1), 1–16. https://doi.org/10.1680/adcr.1998.10.1.1

Maertens, J., & Huybrechts, N. (2003). Belgian Screw Pile Technology - Design and Recent Developments. Lisse, The Netherlands: Swets & Zeitlinger.

Massarsch, K. R., & Wersäll, C. (2013). Cumulative Lateral Soil Displacement Due to Pile Driving in Soft Clay. *Sound Geotechnical Research to Practice*, 463–480. https://doi.org/10.1061/9780784412770.031

Morris, P. H., & Dux, P. F. (2010). Analytical solutions for bleeding of concrete due to consolidation. *Cement and Concrete Research*, *40*(10), 1531–1540. https://doi.org/10.1016/j.cemconres.2010.06.007

NEN-EN 12699 'Uitvoering van bijzonder geotechnisch werk – Verdringingspalen'. Nederlands Normalisatie-instituut, Delft, 2015.

NEN 9997-1+C2 'Geotechnisch ontwerp van constructies – Deel 1: Algemene regels'. Nederlands Normalisatie-instituut, Delft, 2017.

Neville, A. (2011). *Properties of concrete* (5th ed.). Harlow, England: Pearson. Retrieved from https://tudelft.on.worldcat.org/oclc/825820921

Oberlender, D. D. D., & Peurifoy, R. L. (2010). *Formwork for Concrete Structures* (4th ed.). New York: McGraw-Hill Education.

PLAXIS (2019a).PLAXIS material models manual. Plaxis B.V.

PLAXIS (2019b).PLAXIS 2D reference manual. Plaxis B.V

Powers, T. C. (1939). The Bleeding of Portland Cement Paste, Mortar, and Concrete. *ACI Journal Proceedings, 35*(6)*,* 465-480.

Randolph, M. F., & Wroth, C. P. (1979) – An analytical solution for the consolidation around a driven pile. *International journal for numerical and analytical methods in geomechanics,* (3), 217-229.

Randolph, M. F., Carter, J. P., & Wroth, C. P. (1979) – Driven piles in clay – the effects of installation and subsequent consolidation. *Géotechnique*, *29*(4), 351-393.

Randolph, M. F. (2003). Science and empiricism in pile foundation design. Géotechnique, 53(10), 847–875. https://doi.org/10.1680/geot.2003.53.10.847

Smienk, E., & De Quelerij, L. (2010). Handboek funderingen: deel B systemen. : SBR.

Tan, T. S., Wee, T. H., Tan, S. A., Tam, C. T., & Lee, S. L. (1987). A consolidation model for bleeding of cement paste. *Advances in Cement Research*, *1*(1), 18–26. https://doi.org/10.1680/adcr.1987.1.1.18

Tan, T.-S., Loh, C.-K., Yong, K.-Y., & Wee, T.-H. (1997). Modelling of bleeding of cement paste and mortar. *Advances in Cement Research*, *9*(34), 75–91. https://doi.org/10.1680/adcr.1997.9.34.75

Theis, C.V., (1935). The relation between the lowering of the piezometric surface and the rate and duration of discharge of a well using groundwater storage, *Am. Geophys. Union Trans*,*16*, 519-524.

Vanhove, Y., & Khayat, K. H. (2016). Forced Bleeding Test to Assess Stability of Flowable Concrete. *ACI Materials Journal*, *113*(6), 753–758. https://doi.org/10.14359/51689240

Verruijt, A. (2001). *Soil Mechanics*. Retrieved from ocw.tudelft.nl/wp-content/uploads/SoilMechBook.pdf.

Yim, H. J., Kim, J. H., & Kwak, H.-G. (2014). Experimental simulation of bleeding under a high concrete column. *Cement and Concrete Research*, *57*, 61–69. https://doi.org/10.1016/j.cemconres.2013.12.011

Yim, H. J., Kim, J. H., Kwak, H.-G., & Kim, J. K. (2013). Evaluation of internal bleeding in concrete using a self-weight bleeding test. *Cement and Concrete Research*, *53*, 18–24. https://doi.org/10.1016/j.cemconres.2013.05.015

Appendix A

Appendix A.1 CPT Profile

In this appendix a representative soil profile for the project location is given The small increase in cone resistance between -7 and -8m NAP represents the intermediate soil layer.

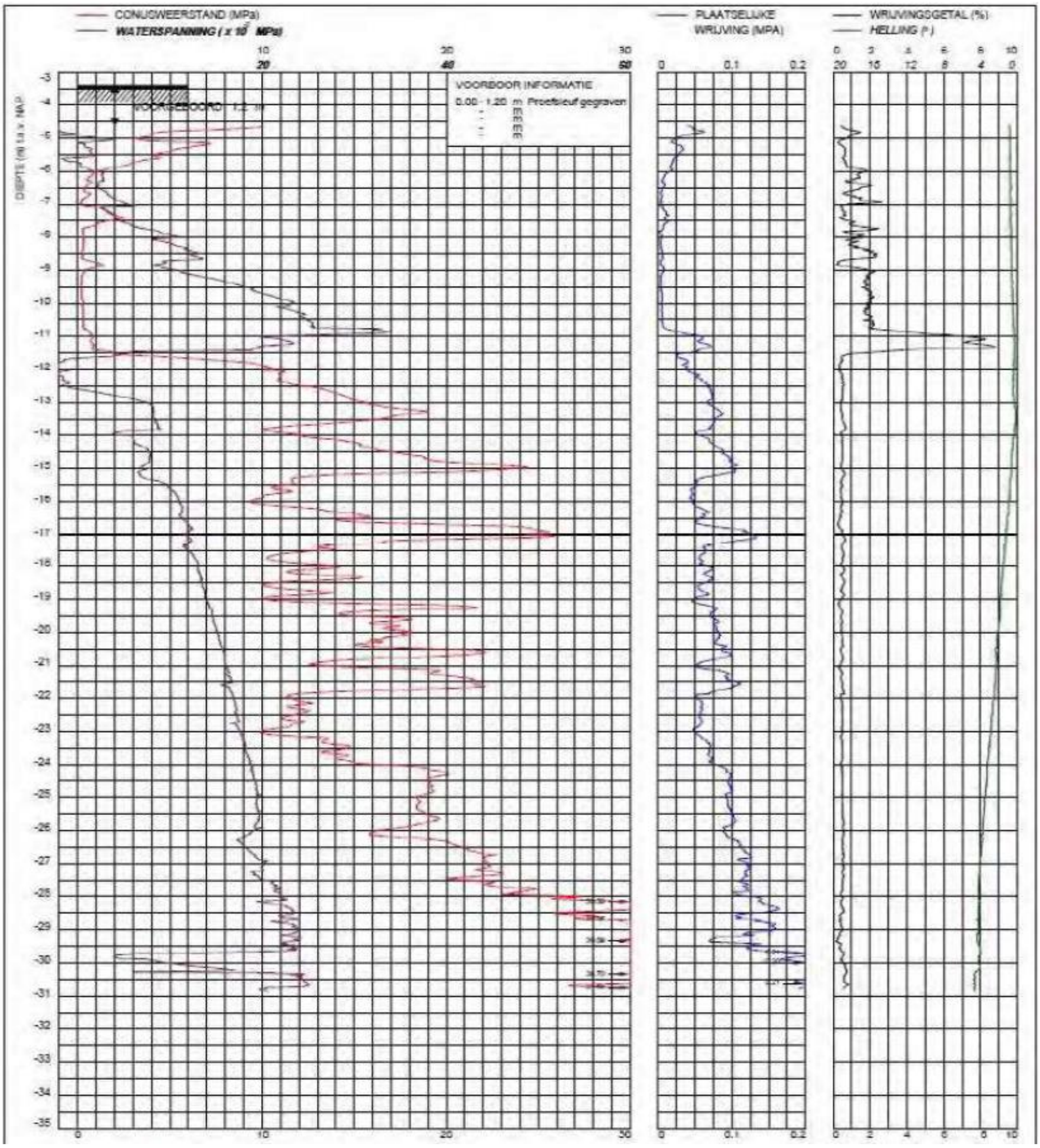


Figure A.1: Representative CPT Profile

Appendix A.2 Pile Test Location Overview

An overview of the two test fields are presented in this appendix. The installation effects were monitored in two test fields. In Test Field 1, four piles were installed but only the measurements of two of them were useable. In Test Field 2, all the measurements near the piles were correctly obtained.

Test Field 1:

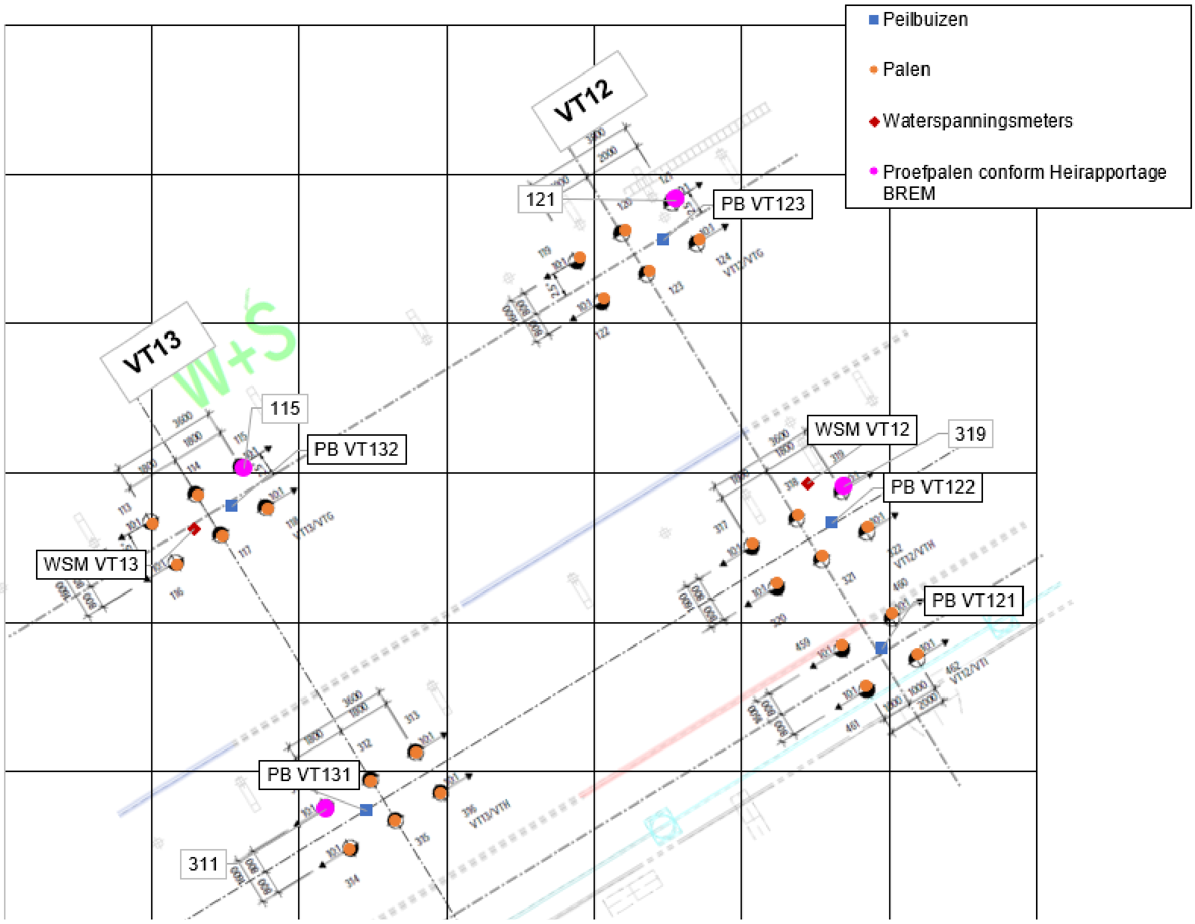


Figure A.2: Locations of the piles and measurement devices in Test Field 1

Table 10 Information of measurement devices Test Field 1

			Radial distance in [m] to pile number			
z [m]	ID	Bottom Filter	121	319	115	311
-4.1493	MW	-7.3633	16.6	5.6	22.5	19.6
-4.2423	MW	-7.4633	12.1	1.3	20	19.6
-4.1351	MW	-7.2331	1.4	10.3	16.2	22.2
-4.1274	MW	-7.0534	23	19.5	12.2	1.4
-4.0523	MW	-7.1833	18.3	20.8	1.4	10.6
-4.1872	EP	-7.47	10.5	1.2	19.1	19.6
-3.9943	EP	-7.0943	19.7	22.1	2.7	10.4

Test Field 2:

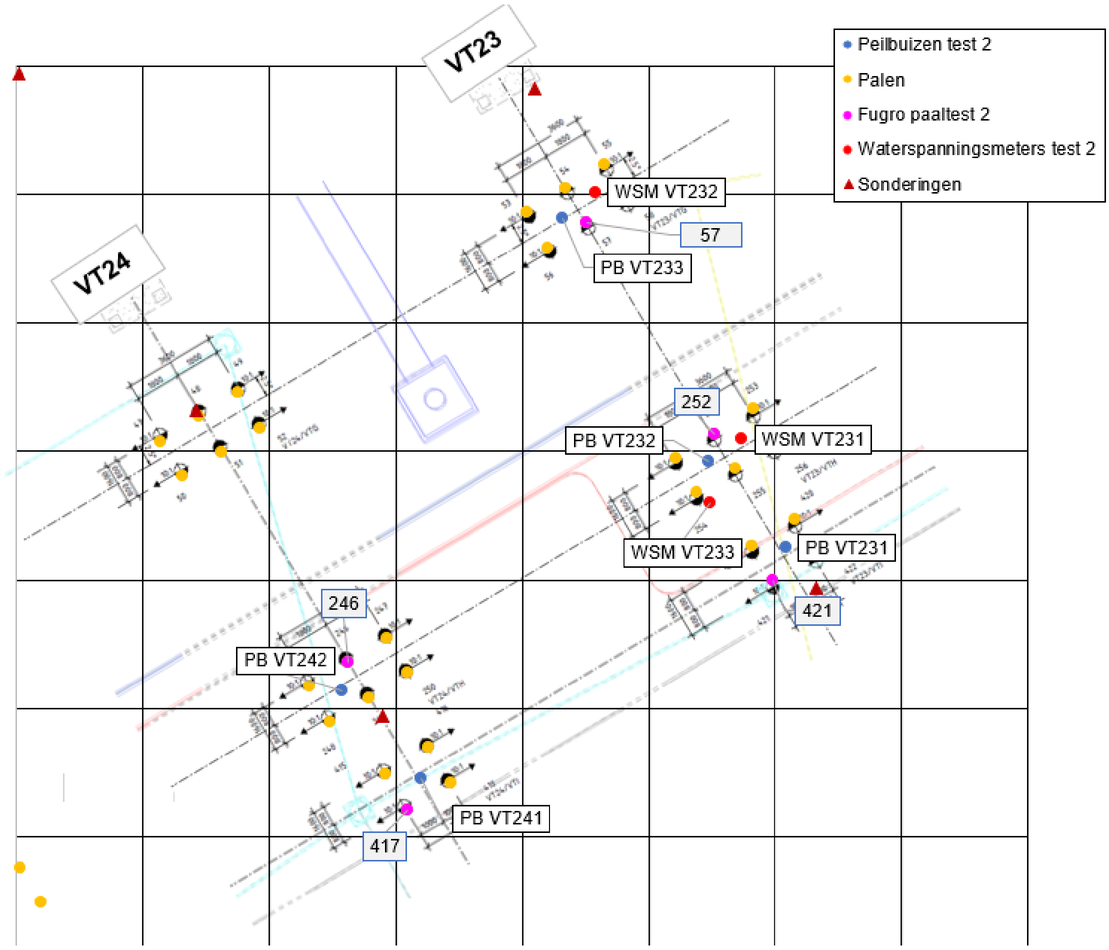


Figure A.3: Locations of the piles and measurement devices in Test Field 2

Table 11 Information of measurement devices Test Field 2

			Radial distance in [m] to pile number				
z [m]	ID	Bottom Filter	421	252	57	246	417
-3.4563	MW	-7.5223	1.4	5.3	14.9	17.9	18.1
-3.5132	MW	-7.4322	5.3	1.1	10.4	16.3	18.1
-3.6691	MW	-7.7501	16.4	10.3	1	19.2	23.8
-3.6424	MW	-7.9934	15.9	17.7	22.6	5.4	1.3
-3.5324	MW	-7.4644	17.6	17.8	20.6	1.1	5.3
-3.4992	EP	-7.1642	5.6	1.1	10.4	17.8	19.6
-3.6321	EP	-7.3331	16.7	10.5	1.2	20.7	25.1
-3.5132	EP	-11.9452	3.9	2.7	11.9	15.6	16.9

Appendix A.3 Hydraulic Head Measurements

In this appendix the measurement of the hydraulic head around the seven installed piles are presented. In the different graphs the same trend can be observed, indicating that increases in water level are related to the same processes.

Test Field 1:

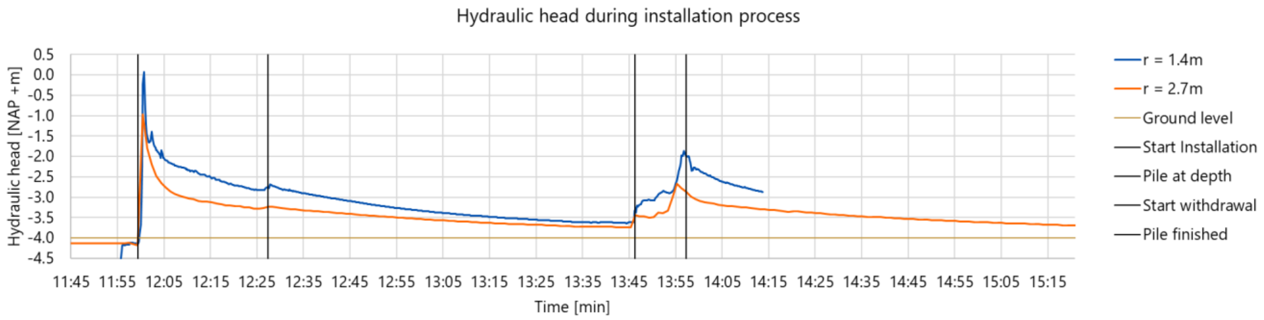


Figure A.4: Measurements of the hydraulic head around pile 115

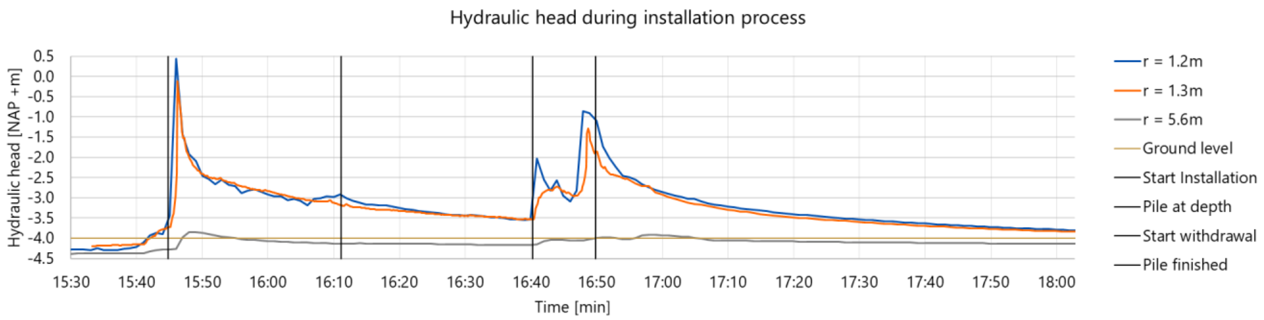


Figure A.5: Measurements of hydraulic head around pile 319

Test Field 2:

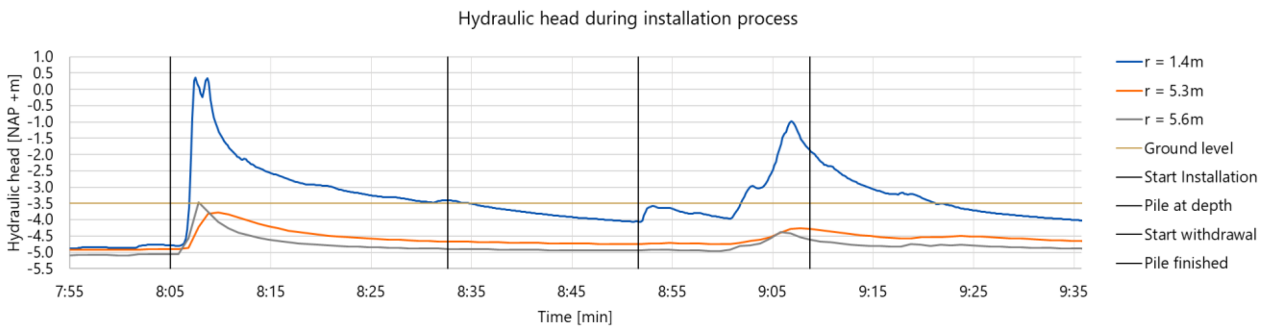


Figure A.6: Measurements of hydraulic head around pile 421

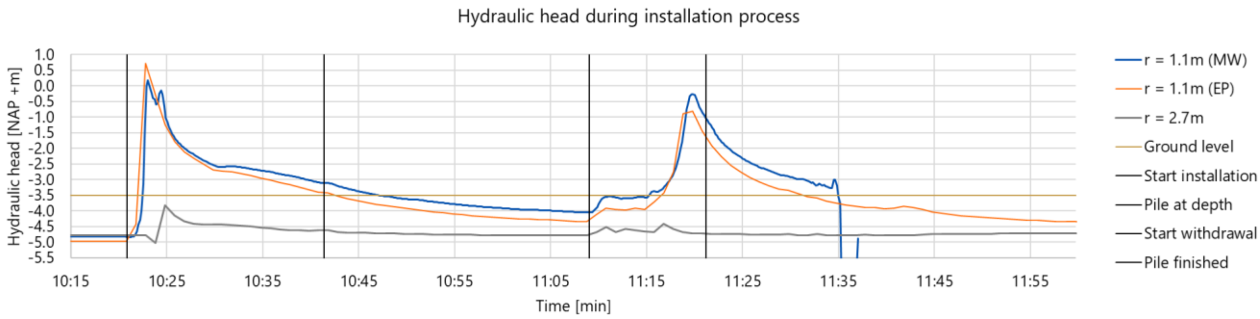


Figure A.7: Measurements of hydraulic head around pile 252

Near pile 252, a monitoring well (MW) and an electric piezometer (EP) were placed at r = 1.1m distance from the pile. The electric piezometer at r = 2.7m was installed in the deeper sand layer at approximately -12m NAP. It clearly seen that the water level variation in this layer is substantially lower.

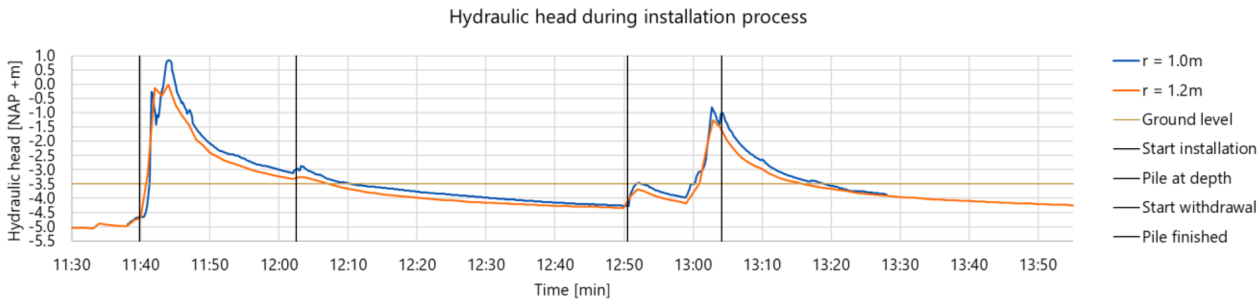


Figure A.8: Measurements of hydraulic head around pile 57

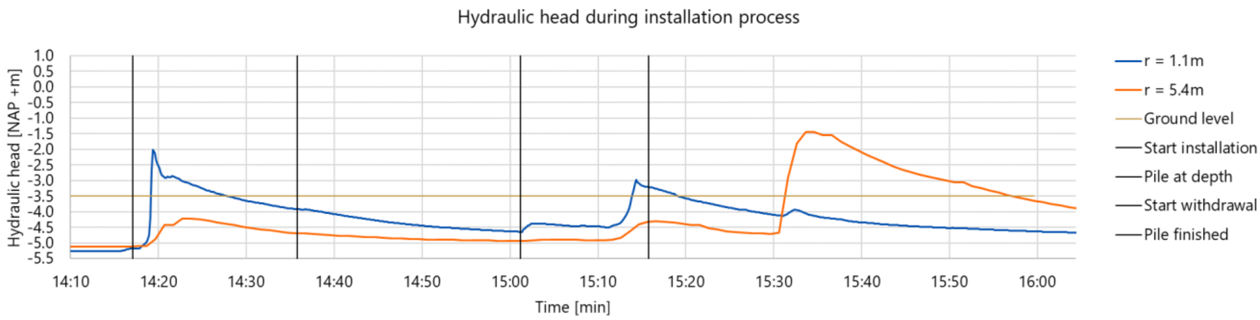


Figure A.9: Measurements of hydraulic head around pile 246

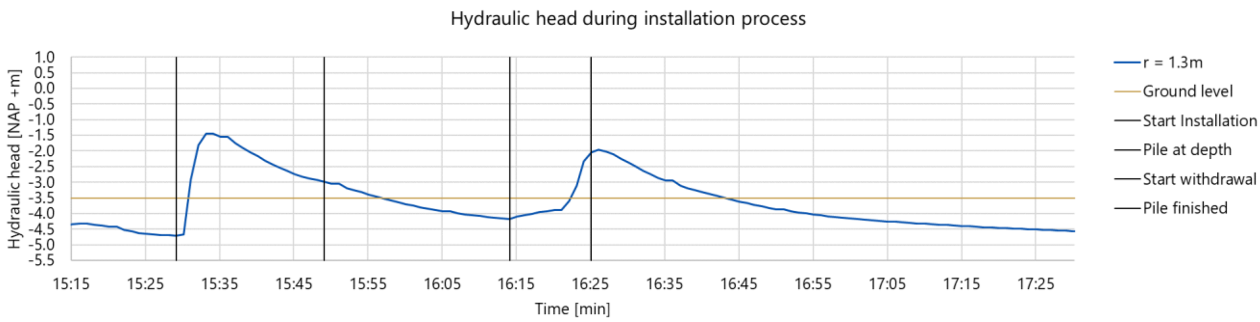
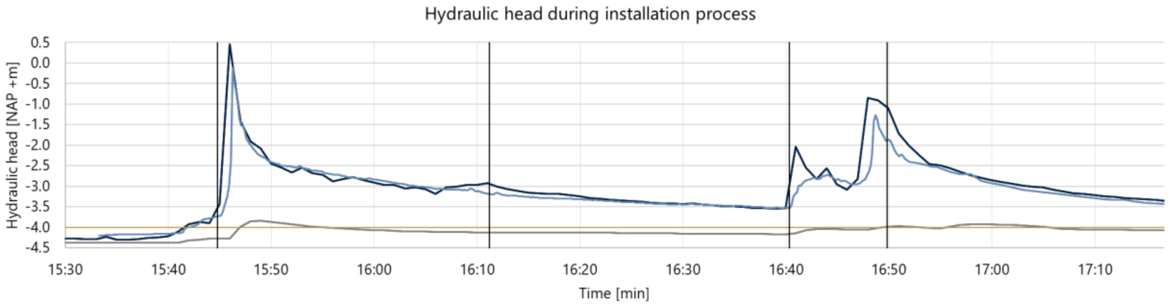


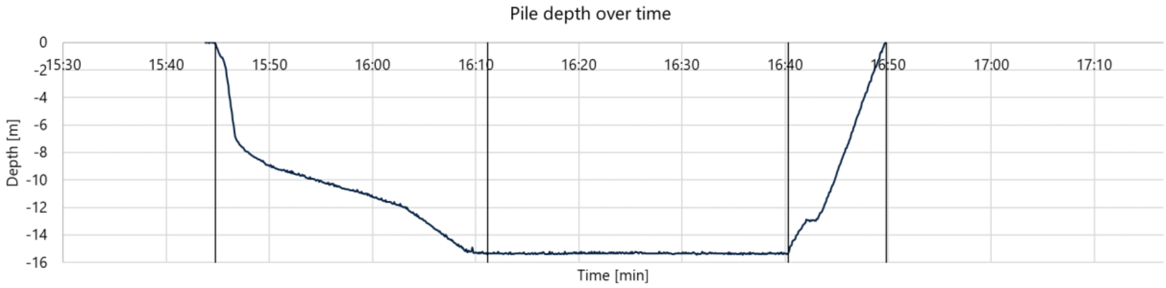
Figure A.10: Measurements of hydraulic head around pile 417

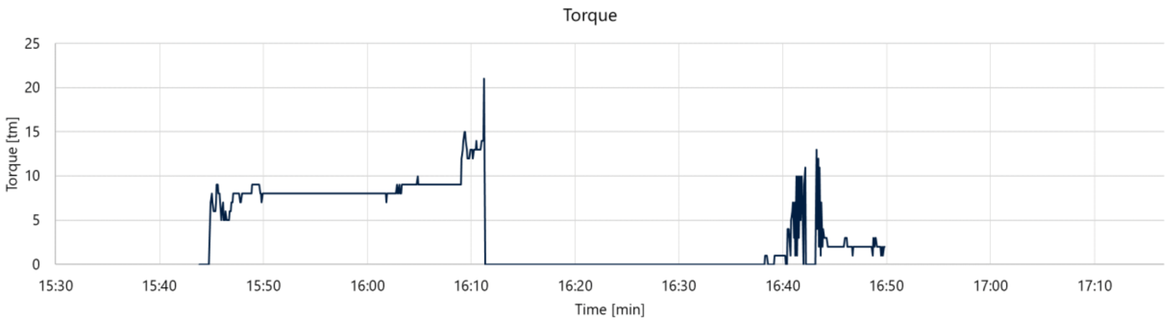
Appendix A.4 Crane Registrations

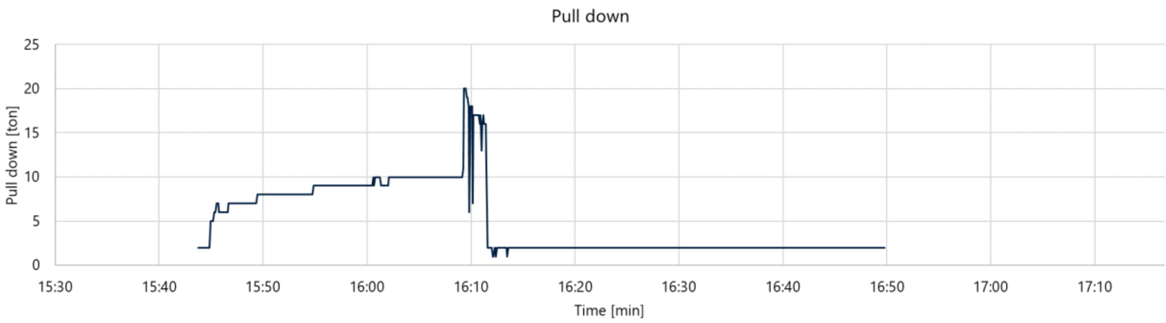
In this appendix all the crane registrations are presented under the graph of the measurements of the hydraulic head to be able to compare the installation processes to the effects measured in the surrounding soil. The crane registrations of Pile 115 were not correctly measured so these could not be used and the comparison between Pile 417 and the crane registrations was not useful as the only measurement device near Pile 417 measured only 1 per minute which was not enough to monitor the different processes.

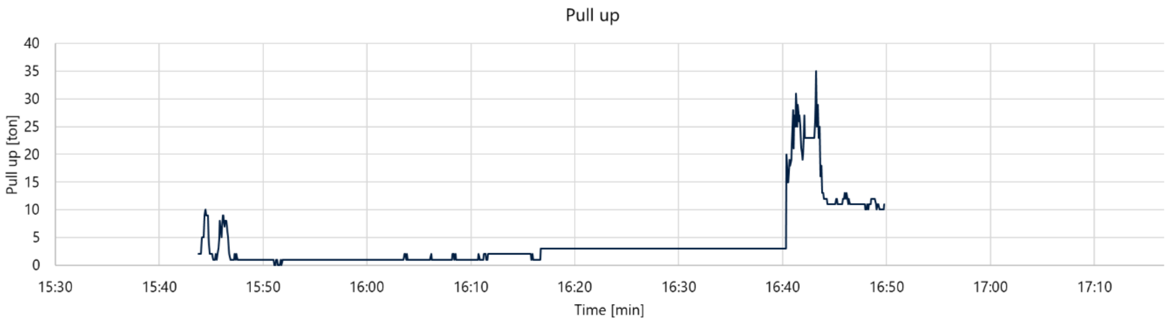
Pile 319:

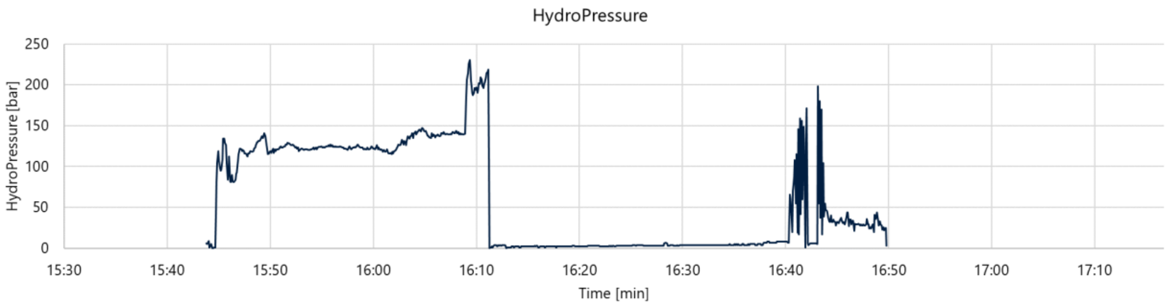


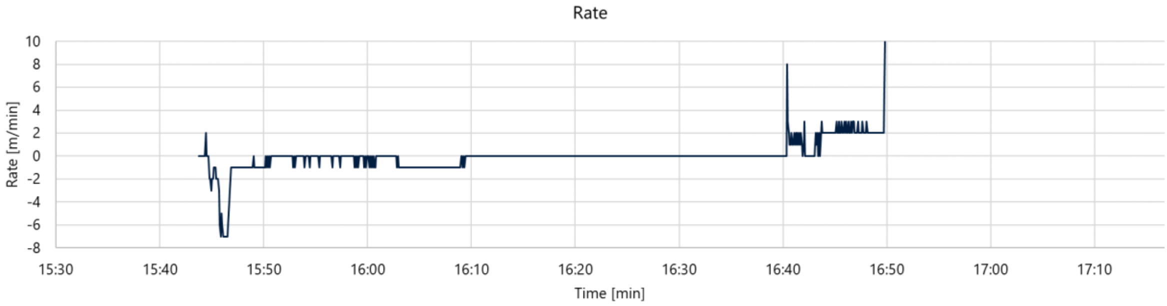


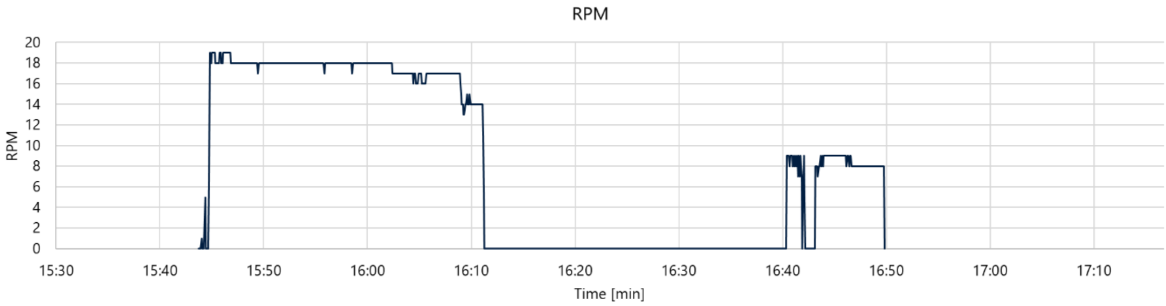




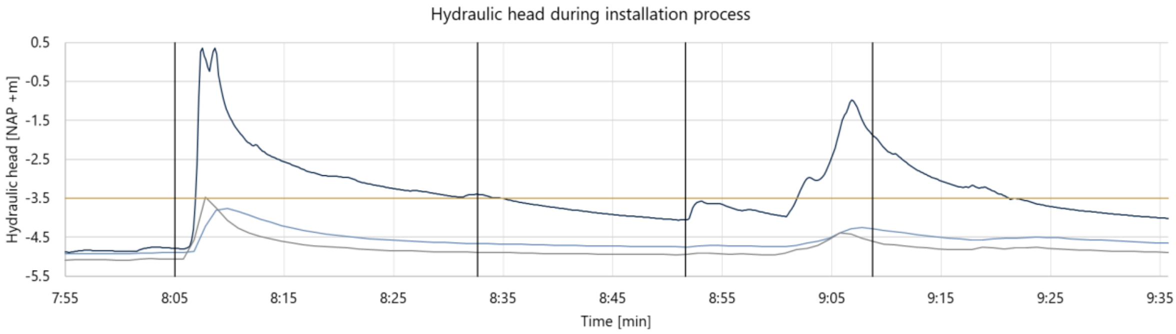

Pull up
Pull up [ton]
0
5
10
15
20
25
30
35
40
15:30
15:40
15:50
16:00
16:10
16:20
16:30
16:40
16:50
17:00
17:10
Time [min]

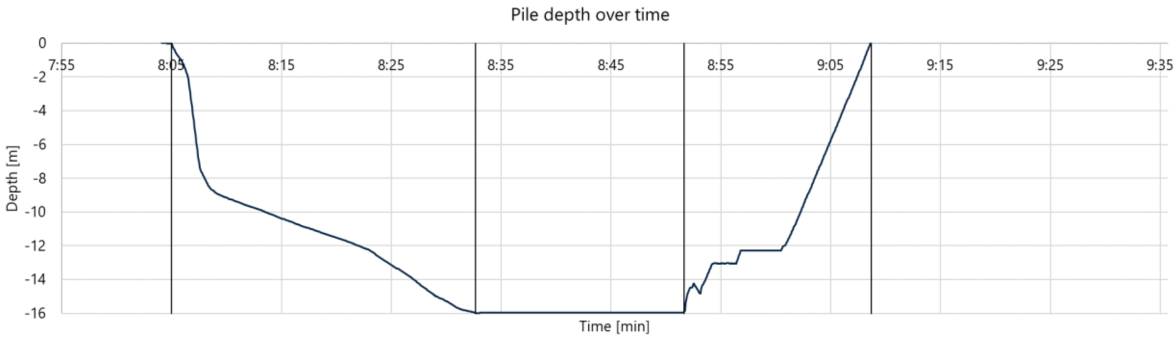

HydroPressure
HydroPressure [bar]
0
50
100
150
200
250
15:30
15:40
15:50
16:00
16:10
16:20
16:30
16:40
16:50
17:00
17:10
Time [min]

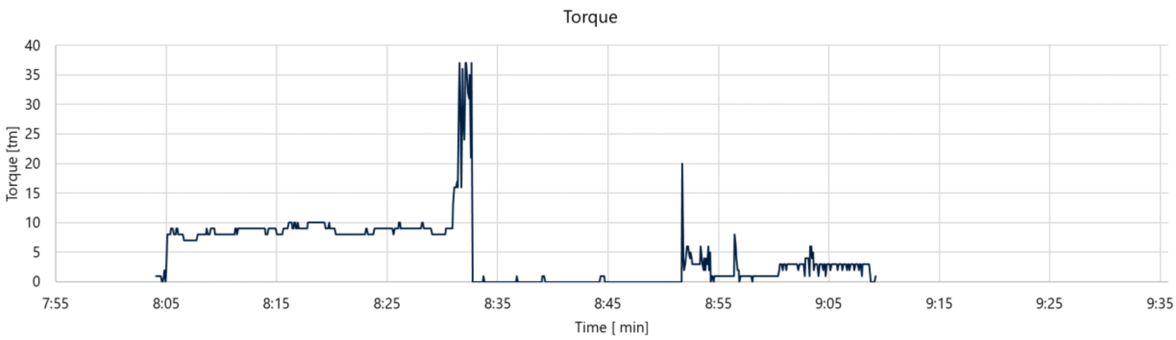

Rate
Rate [m/min]
-8
-6
-4
-2
0
2
4
6
8
10
15:30
15:40
15:50
16:00
16:10
16:20
16:30
16:40
16:50
17:00
17:10
Time [min]

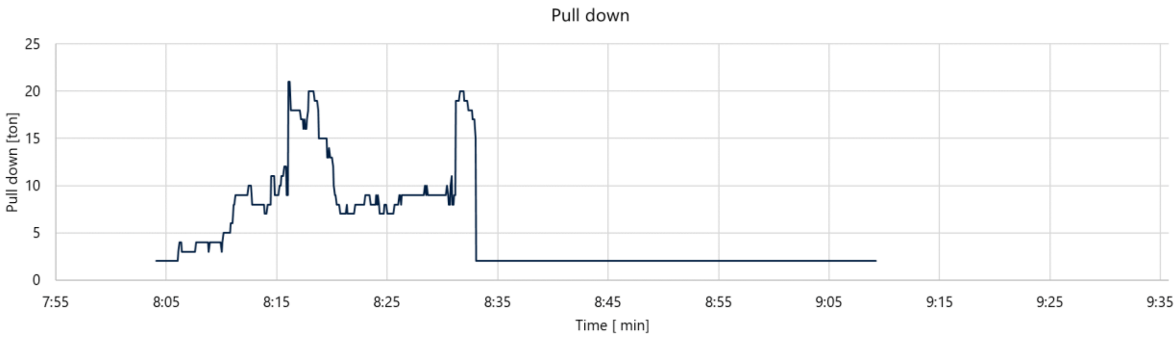

RPM
RPM
0
2
4
6
8
10
12
14
16
18
20
15:30
15:40
15:50
16:00
16:10
16:20
16:30
16:40
16:50
17:00
17:10
Time [min]

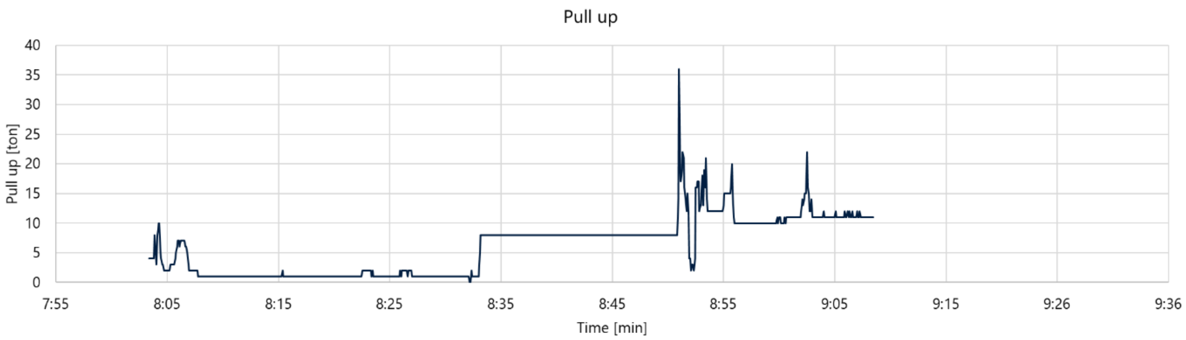
Pile 421:

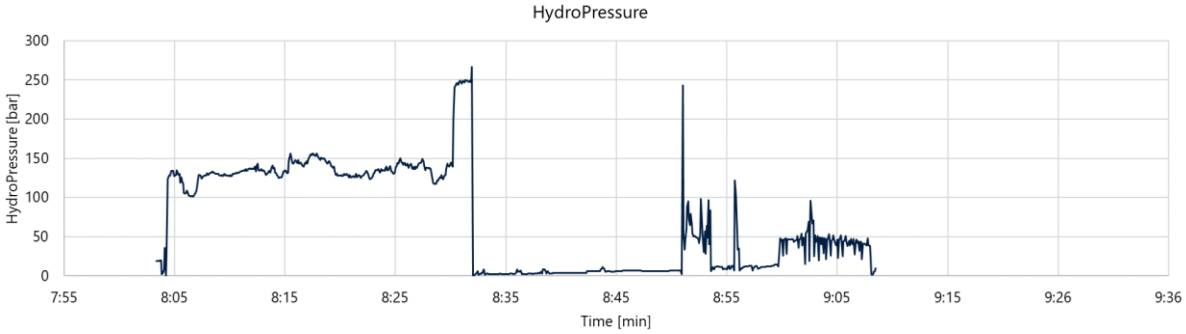

Hydraulic head during installation process
Hydraulic head [NAP +m]
0.5
-0.5
-1.5
-2.5
-3.5
-4.5
-5.5
7:55
8:05
8:15
8:25
8:35
8:45
8:55
9:05
9:15
9:25
9:35
Time [min]

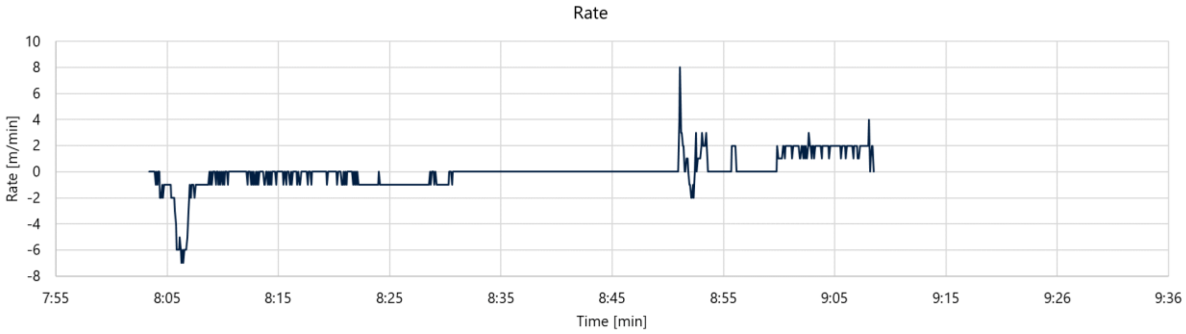

Pile depth over time
Depth [m]
0
-2
-4
-6
-8
-10
-12
-14
-16
7:55
8:05
8:15
8:25
8:35
8:45
8:55
9:05
9:15
9:25
9:35
Time [min]

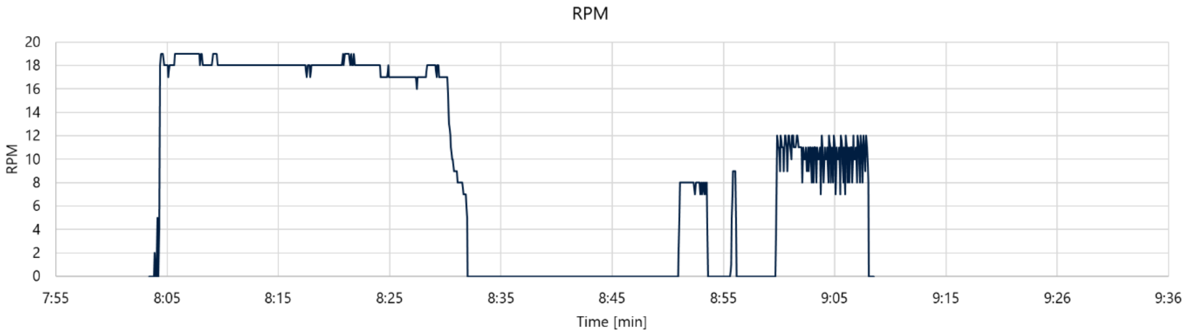

Torque
Torque [tm]
40
35
30
25
20
15
10
5
0
7:55
8:05
8:15
8:25
8:35
8:45
8:55
9:05
9:15
9:25
9:35
Time [min]


Pull down
Pull down [ton]
25
20
15
10
5
0
7:55
8:05
8:15
8:25
8:35
8:45
8:55
9:05
9:15
9:25
9:35
Time [min]

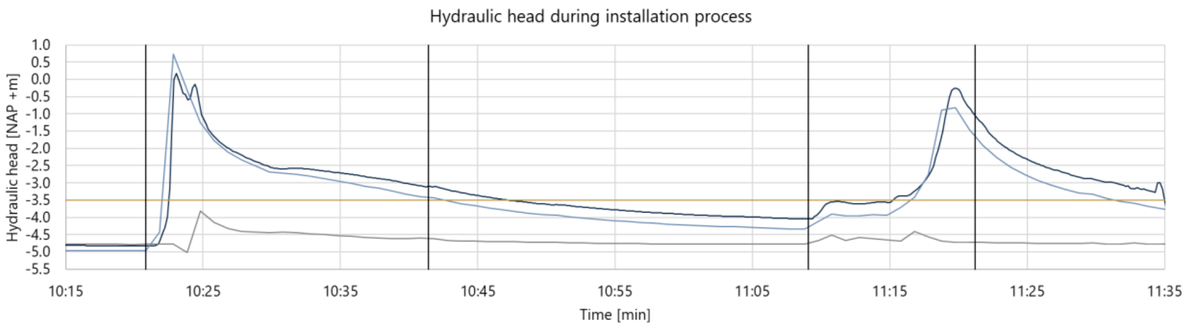

Pull up
Pull up [ton]
40
35
30
25
20
15
10
5
0
7:55
8:05
8:15
8:25
8:35
8:45
8:55
9:05
9:15
9:26
9:36
Time [min]

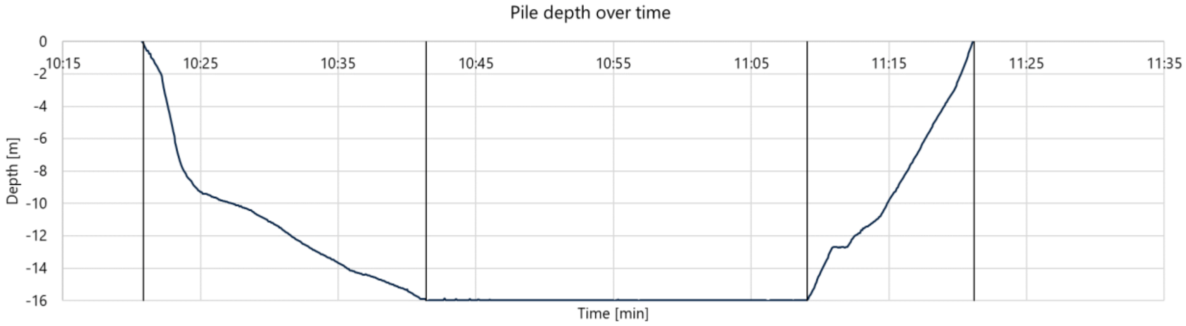

HydroPressure
HydroPressure [bar]
300
250
200
150
100
50
0
7:55
8:05
8:15
8:25
8:35
8:45
8:55
9:05
9:15
9:26
9:36
Time [min]

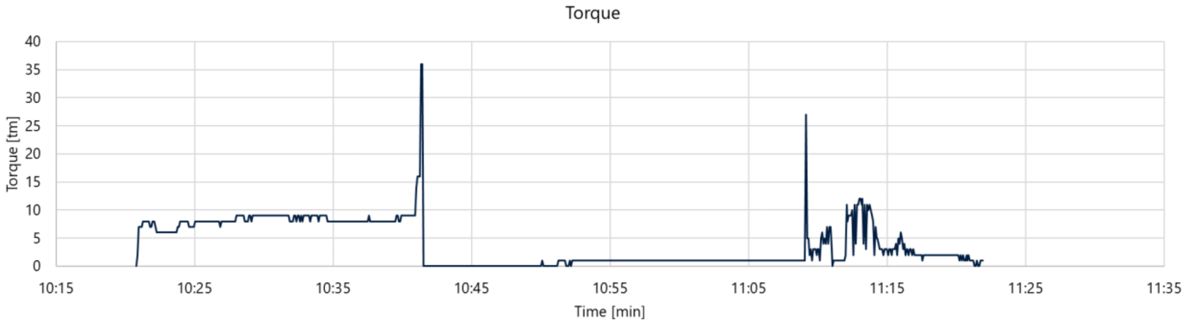

Rate
Rate [m/min]
10
8
6
4
2
0
-2
-4
-6
-8
7:55
8:05
8:15
8:25
8:35
8:45
8:55
9:05
9:15
9:26
9:36
Time [min]

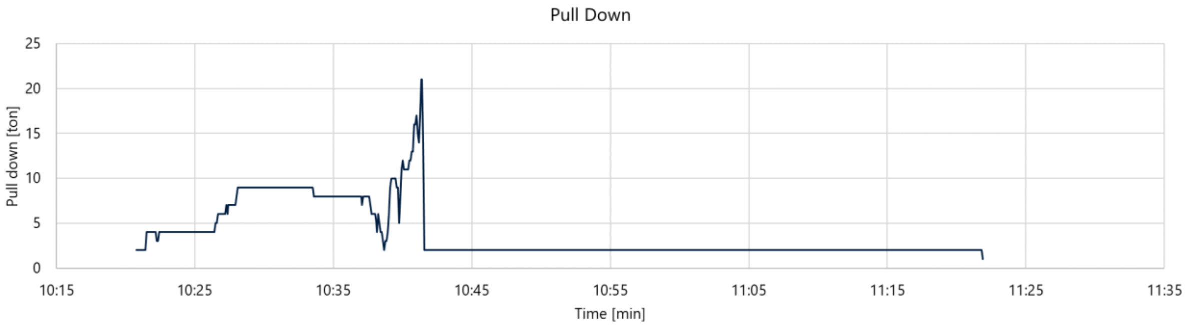

RPM
RPM
20
18
16
14
12
10
8
6
4
2
0
7:55
8:05
8:15
8:25
8:35
8:45
8:55
9:05
9:15
9:26
9:36
Time [min]

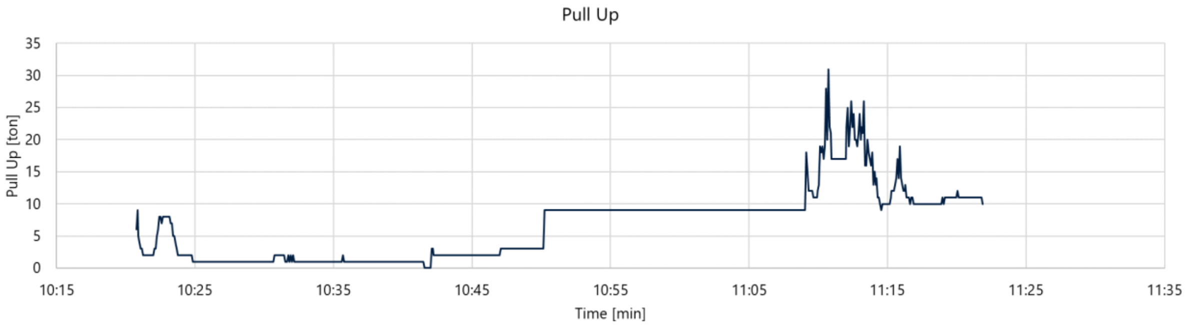
Pile 252:

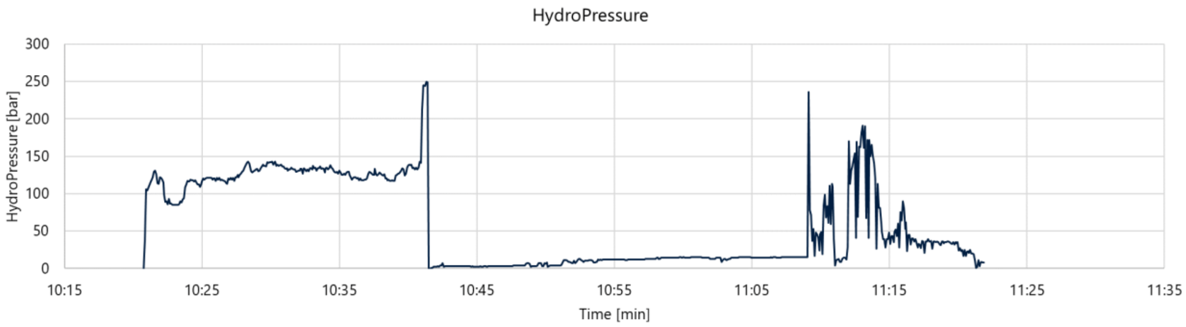

Hydraulic head during installation process
Hydraulic head [NAP +m]
1.0
0.5
0.0
-0.5
-1.0
-1.5
-2.0
-2.5
-3.0
-3.5
-4.0
-4.5
-5.0
-5.5
10:15
10:25
10:35
10:45
10:55
11:05
11:15
11:25
11:35
Time [min]

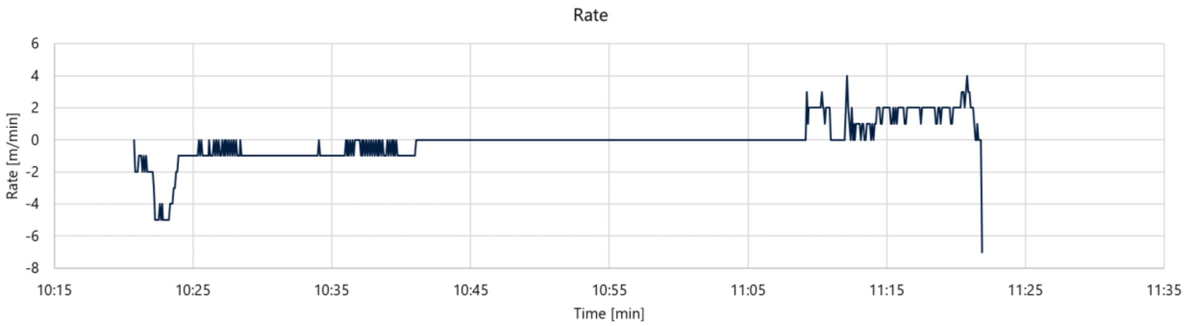

Pile depth over time
Depth [m]
0
-2
-4
-6
-8
-10
-12
-14
-16
10:15
10:25
10:35
10:45
10:55
11:05
11:15
11:25
11:35
Time [min]

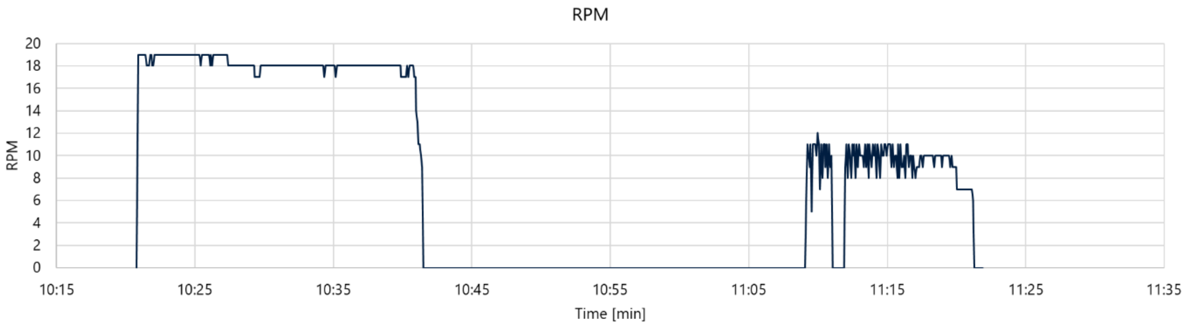

Torque
Torque [tm]
40
35
30
25
20
15
10
5
0
10:15
10:25
10:35
10:45
10:55
11:05
11:15
11:25
11:35
Time [min]


Pull Down
Pull down [ton]
25
20
15
10
5
0
10:15
10:25
10:35
10:45
10:55
11:05
11:15
11:25
11:35
Time [min]

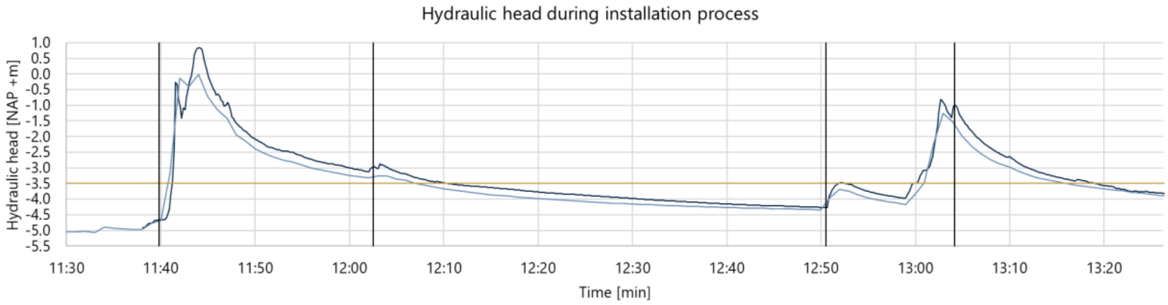

Pull Up
Pull Up [ton]
35
30
25
20
15
10
5
0
10:15
10:25
10:35
10:45
10:55
11:05
11:15
11:25
11:35
Time [min]

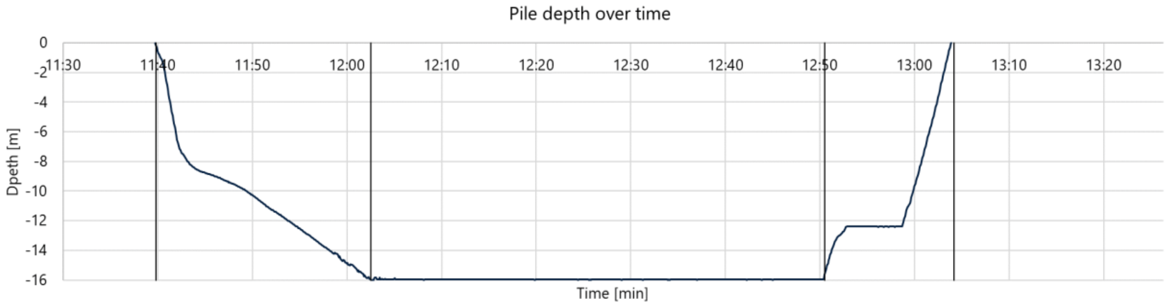

HydroPressure
300
250
200
150
100
50
0
HydroPressure [bar]
10:15
10:25
10:35
10:45
10:55
11:05
11:15
11:25
11:35
Time [min]

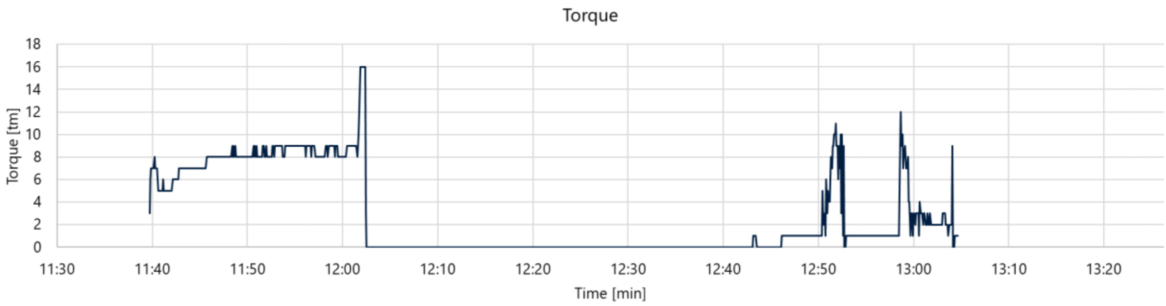

Rate
6
4
2
0
-2
-4
-6
-8
Rate [m/min]
10:15
10:25
10:35
10:45
10:55
11:05
11:15
11:25
11:35
Time [min]

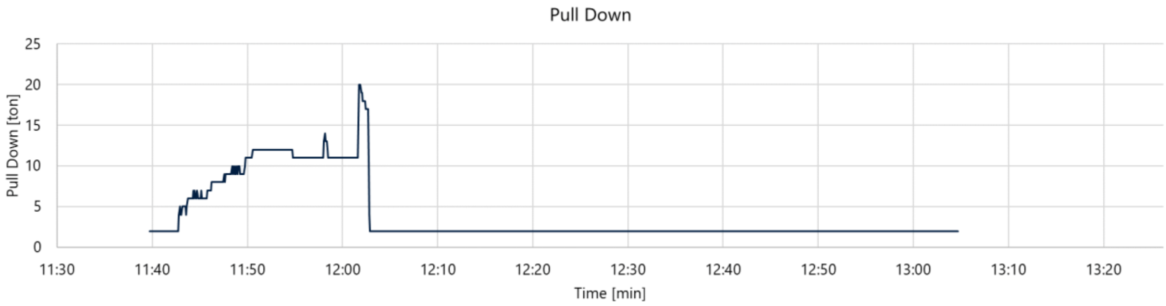

RPM
20
18
16
14
12
10
8
6
4
2
0
RPM
10:15
10:25
10:35
10:45
10:55
11:05
11:15
11:25
11:35
Time [min]

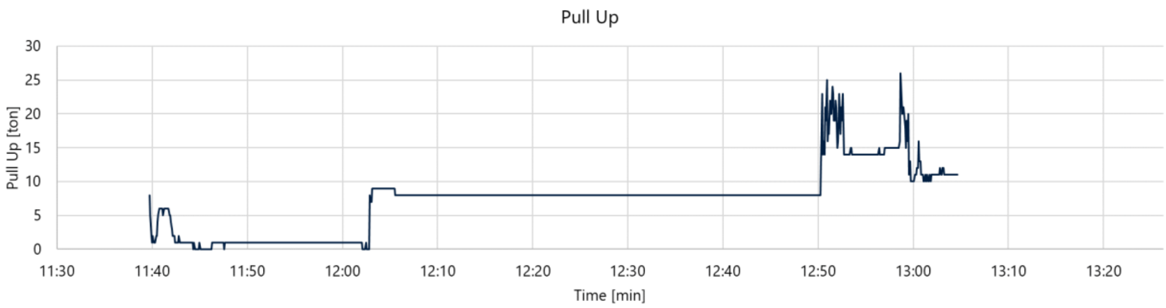
Pile 57:

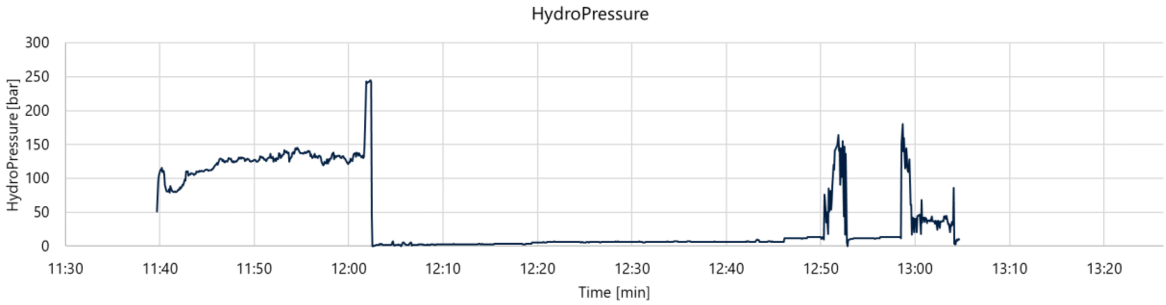

Hydraulic head during installation process
Hydraulic head [NAP +m]
1.0
0.5
0.0
-0.5
-1.0
-1.5
-2.0
-2.5
-3.0
-3.5
-4.0
-4.5
-5.0
-5.5
11:30
11:40
11:50
12:00
12:10
12:20
12:30
12:40
12:50
13:00
13:10
13:20
Time [min]

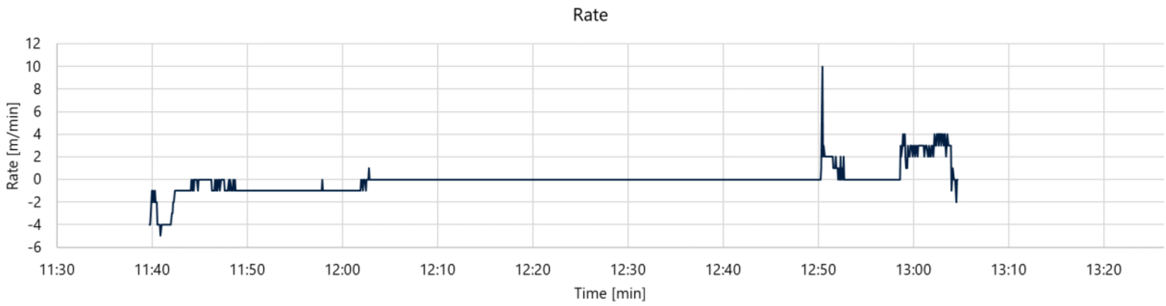

Pile depth over time
Dpeth [m]
0
-2
-4
-6
-8
-10
-12
-14
-16
11:30
11:40
11:50
12:00
12:10
12:20
12:30
12:40
12:50
13:00
13:10
13:20
Time [min]

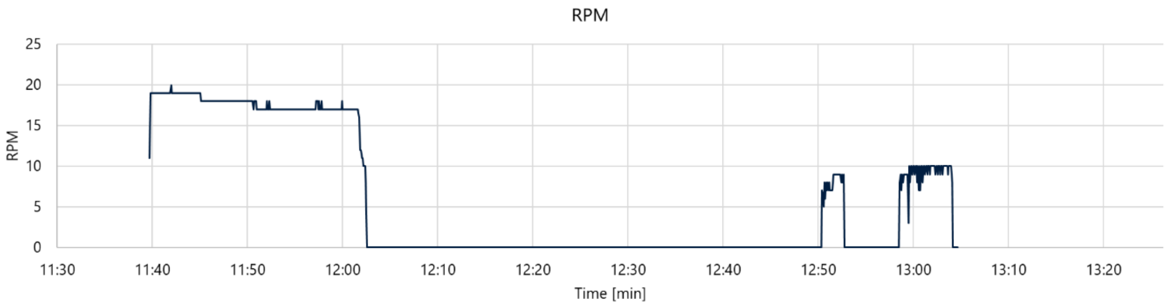

Torque
Torque [tm]
18
16
14
12
10
8
6
4
2
0
11:30
11:40
11:50
12:00
12:10
12:20
12:30
12:40
12:50
13:00
13:10
13:20
Time [min]


Pull Down
Pull Down [ton]
25
20
15
10
5
0
11:30
11:40
11:50
12:00
12:10
12:20
12:30
12:40
12:50
13:00
13:10
13:20
Time [min]

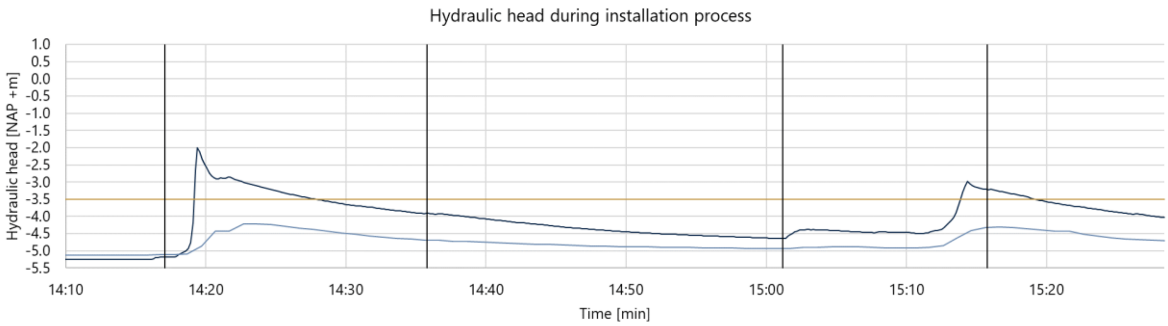

Pull Up
Pull Up [ton]
30
25
20
15
10
5
0
11:30
11:40
11:50
12:00
12:10
12:20
12:30
12:40
12:50
13:00
13:10
13:20
Time [min]


HydroPressure
HydroPressure [bar]
300
250
200
150
100
50
0
11:30
11:40
11:50
12:00
12:10
12:20
12:30
12:40
12:50
13:00
13:10
13:20
Time [min]

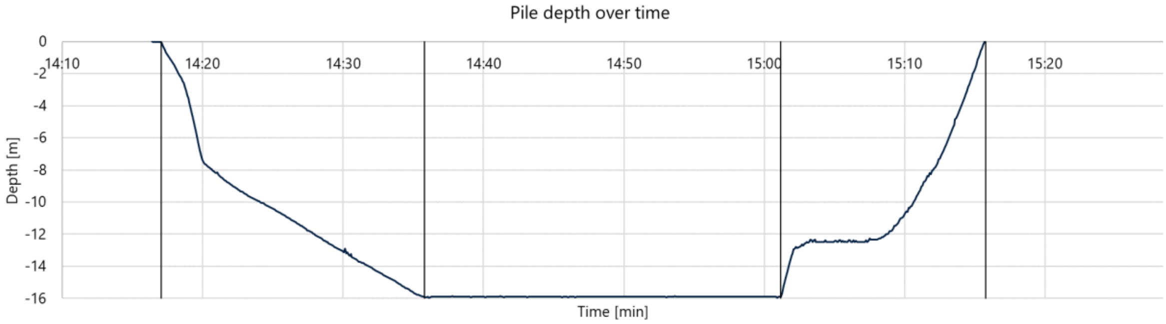

Rate
Rate [m/min]
12
10
8
6
4
2
0
-2
-4
-6
11:30
11:40
11:50
12:00
12:10
12:20
12:30
12:40
12:50
13:00
13:10
13:20
Time [min]


RPM
RPM
25
20
15
10
5
0
11:30
11:40
11:50
12:00
12:10
12:20
12:30
12:40
12:50
13:00
13:10
13:20
Time [min]

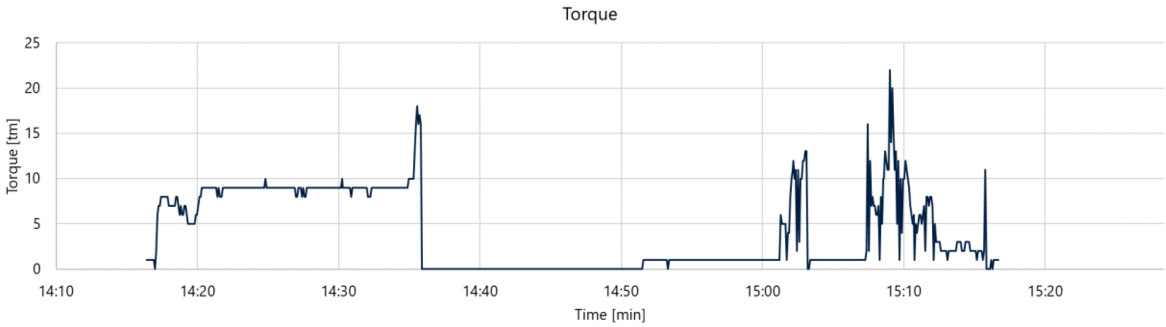
Pile 246:



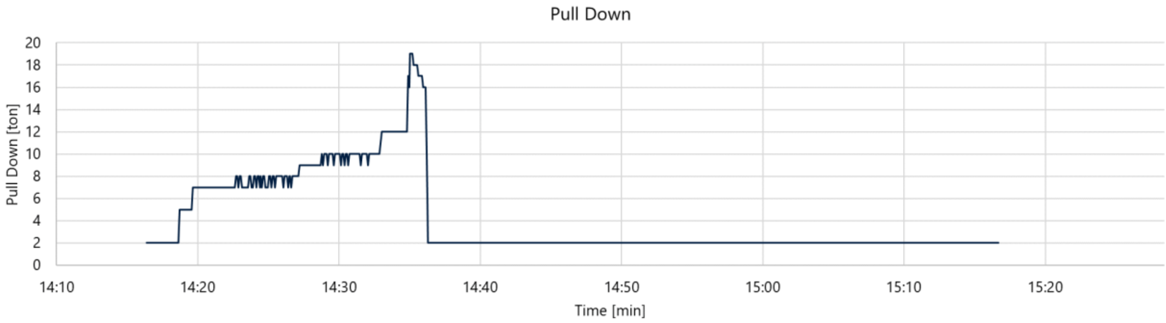
Hydraulic head during installation process
Hydraulic head [NAP +m]
1.0
0.5
0.0
-0.5
-1.0
-1.5
-2.0
-2.5
-3.0
-3.5
-4.0
-4.5
-5.0
-5.5
14:10
14:20
14:30
14:40
14:50
15:00
15:10
15:20
Time [min]



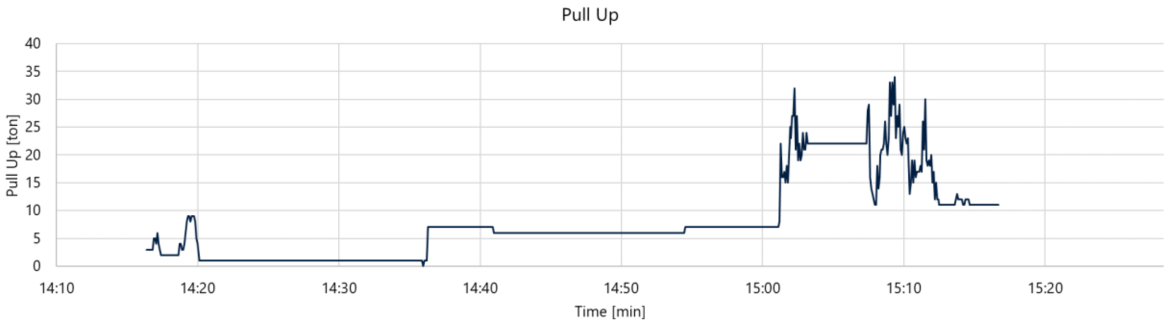
Pile depth over time
Depth [m]
0
-2
-4
-6
-8
-10
-12
-14
-16
14:10
14:20
14:30
14:40
14:50
15:00
15:10
15:20
Time [min]



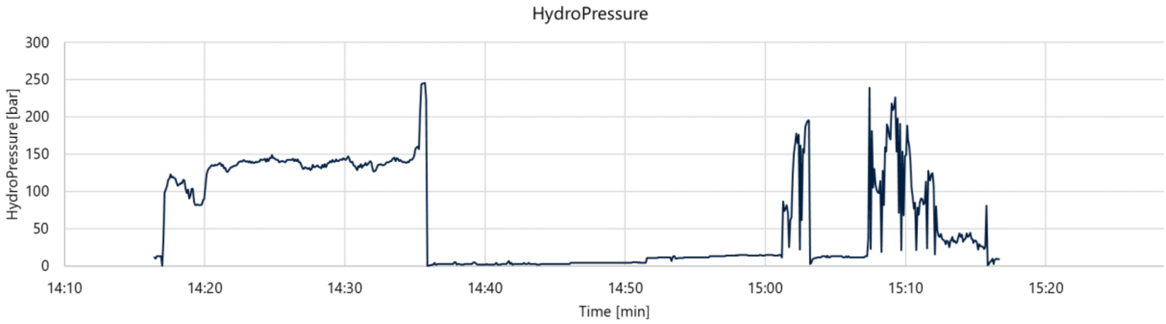
Torque
Torque [tm]
25
20
15
10
5
0
14:10
14:20
14:30
14:40
14:50
15:00
15:10
15:20
Time [min]

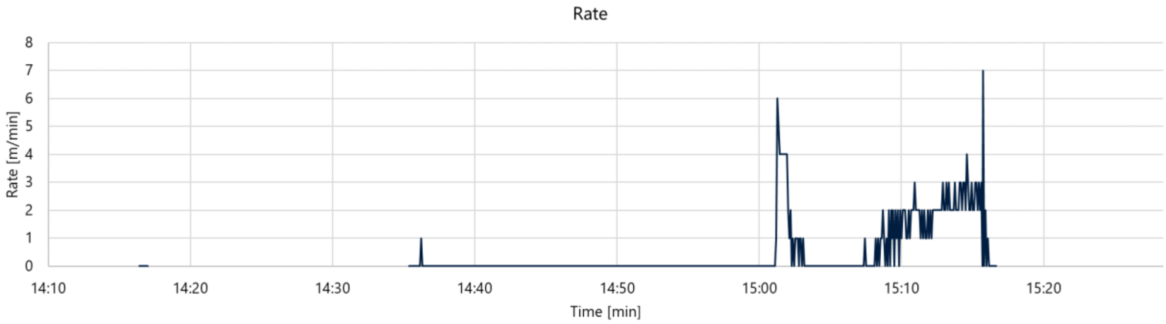


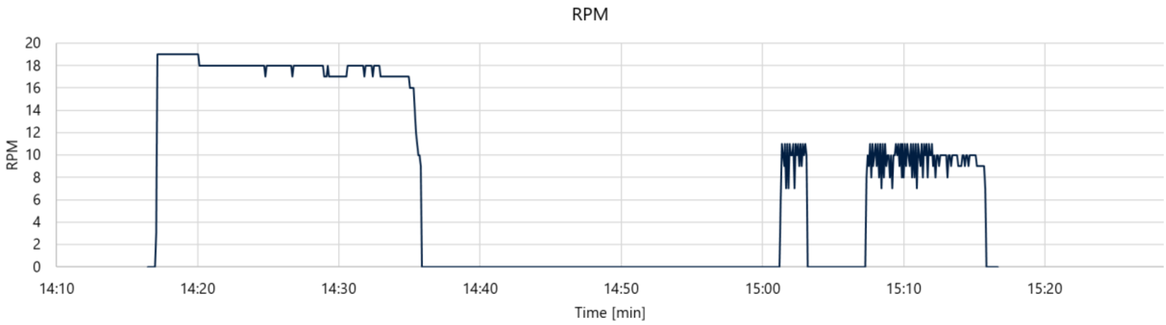
Pull Down
Pull Down [ton]
20
18
16
14
12
10
8
6
4
2
0
14:10
14:20
14:30
14:40
14:50
15:00
15:10
15:20
Time [min]



Pull Up
Pull Up [ton]
40
35
30
25
20
15
10
5
0
14:10
14:20
14:30
14:40
14:50
15:00
15:10
15:20
Time [min]


HydroPressure
HydroPressure [bar]
300
250
200
150
100
50
0
14:10
14:20
14:30
14:40
14:50
15:00
15:10
15:20
Time [min]


Rate
Rate [m/min]
8
7
6
5
4
3
2
1
0
14:10
14:20
14:30
14:40
14:50
15:00
15:10
15:20
Time [min]


RPM
RPM
20
18
16
14
12
10
8
6
4
2
0
14:10
14:20
14:30
14:40
14:50
15:00
15:10
15:20
Time [min]

Appendix A.5 Comparison Geohydrological Model versus Measurements

In this appendix the comparison between the measurements of the hydraulic head and the modelled hydraulic head with the geohydrological model are presented. Again, only the measurements with a frequency of 6 times per minute were used for the comparison because otherwise different processes would be modelled which would be incorrect.

Test Field 1:

Pile 319 – Due to the large disturbance before the pile reached the intermediate sand layer it was not possible to compare the measurements with the geohydrological model.

Test Field 2:

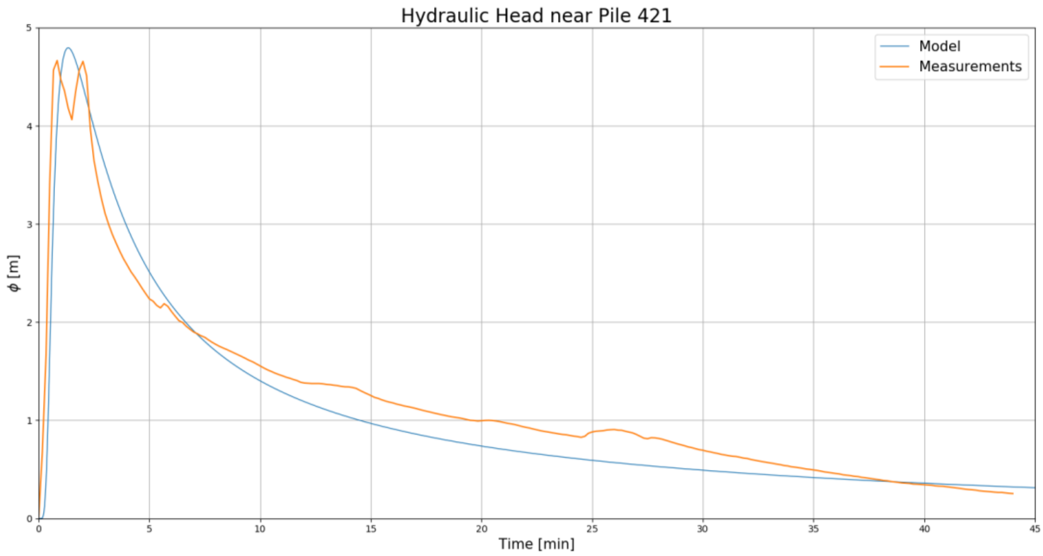


Figure A.11: Measurement of the hydraulic head near pile 421 compared to the output of the Geohydrological model

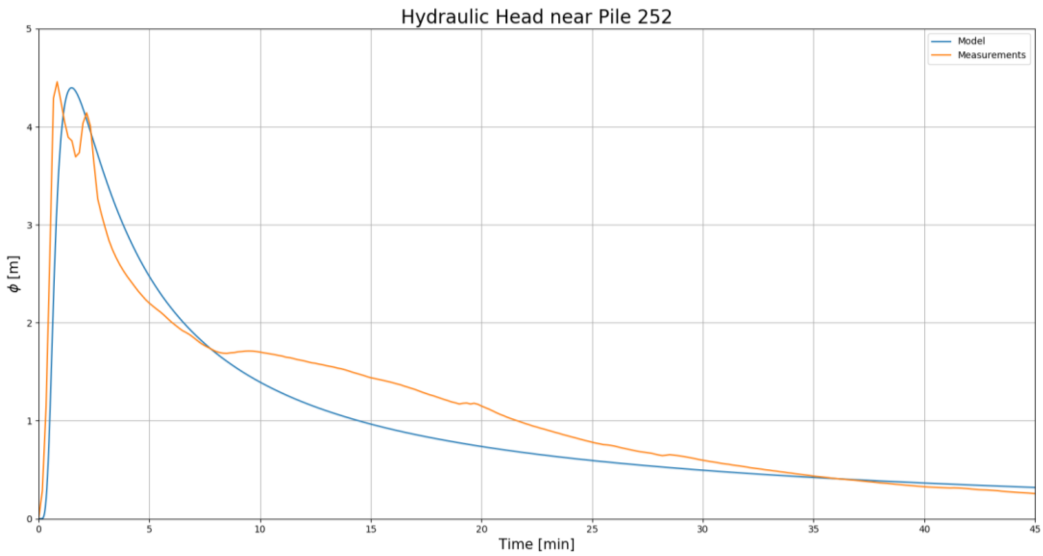


Figure A.12: Measurement of the hydraulic head near pile 252 compared to the output of the Geohydrological model

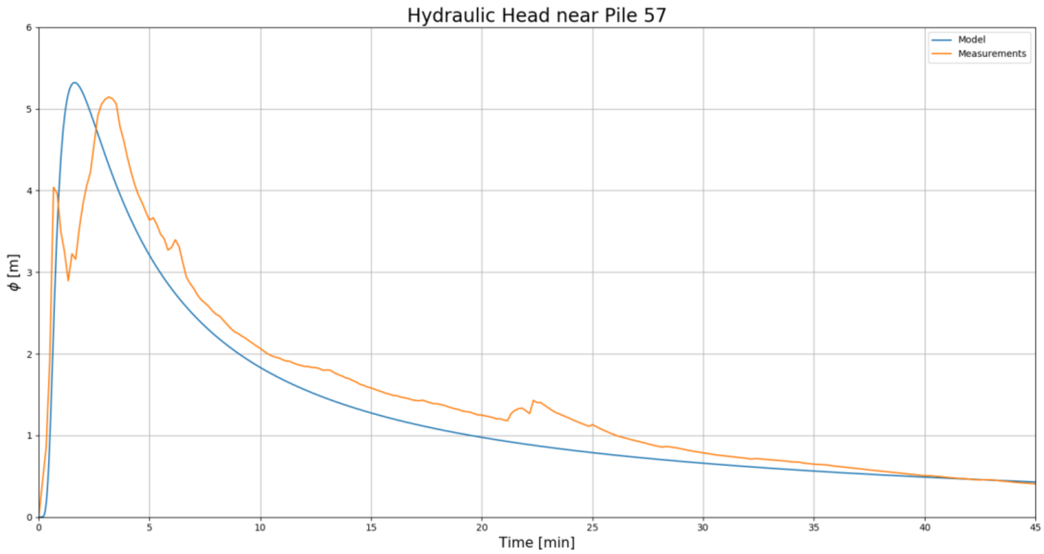


Figure A.13: Measurement of the hydraulic head near pile 57 compared to the output of the Geohydrological model

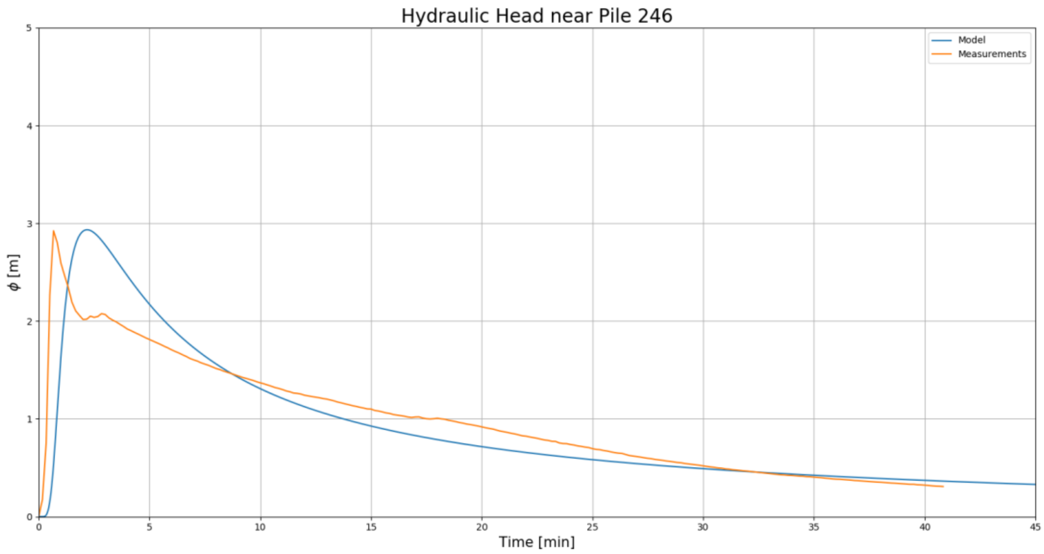


Figure A.14: Measurement of the hydraulic head near pile 246 compared to the output of the Geohydrological model