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Evaluation of unwanted drilling effects: In-situ measurements compared to finite element simulations

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Abstract:
In the latter years, there has been an increasing interest in unwanted settlements and damages occurring on neighboring infrastructure due to foundation work. There has been a research project going from 2012 to 2015 focusing on problems regarding this, called BegrensSkade. This research project has amongst other causes identified drilling for piles and anchors as a cause to unwanted settlements and damages. Several negative effects due to drilling have been assumed, amongst them a local suction around the drill bit (Venturi effect) and mechanical disturbance around the pile-/anchor casing. These two effects are both considered to create settlements in the adjacent soil, either due to loss of soil from excess flushing or due to direct disturbance (straining) and following reconsolidation of the surrounding soil.

The two effects are in this thesis evaluated through literature survey, measurements from two cases, and implementation of the effects in Finite Element Analysis (FEA). The cases presented are two construction projects where both temporary pore pressure fluctuations and settlements occurred around the time of drilling for piles. With the basis in these, the interest is to further highlight the possibility of these two effects.

The results from FEA show that through a parametric study of transient groundwater flow, the same pore pressure reductions as observed in the projects could be achieved for some parameter variations. The question for further work is then to figure out the reason for the assumed loss of soil mass; if it is mainly due to the entrainment from water on soil particles, if the suction itself drags the soil particles, or if there occurs a combination of the two.

Keywords:

1. Unwanted drilling effects
2. Venturi effect
3. Soil disturbance
4. Finite Element Analysis

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BACKGROUND

Foundation works with deep excavations, piling or anchor installations are far too often causing damages on neighboring infrastructure. The problem has been addressed in the research project *BegrensSkade* that started in 2012, where potential sources of unintended settlements and damages have been analyzed and evaluated. BegrensSkade is financed by the Research Council of Norway and industry partners. NGI is the host institution while NTNU is a research partner. The project has gathered and presented data from several construction projects where unexpected settlement or damages have occurred.

PROBLEM DEFINITION

One of the causes of problems is found to be the drilling for piles and tie-back anchors. An ongoing PhD study by Einar John Lande (NGI/NTNU), related to BegrensSkade, is focusing on measured pore pressure fluctuations and unexpected settlements in the soil adjacent to the drilling. Einar Jon Lande has suggested this MSc study in order to look deeper into some of the observations and do finite element simulations in order to explain the observations.

Two effects related to drilling for piles and tie-back anchors have been suggested and should be addressed. These effects are: (1) Assumed suction or local under-pressures as a reason for loss of soil mass around the drill bit. This has possibly similarities to the Venturi effect. (2) Mechanical disturbance around the casing due to the drilling operation, causing volume reduction from the collapse of soil structure.
A literature study shall be made. The student shall consider proposed ideas from BegrensSkade and E. J. Lande and use previous work on the subject and study observations and experiences from real construction projects. Simple laboratory tests on the Venturi effect can be considered. The effects (1) and (2) shall be studied using Finite Element Analyses (FEA). The results shall be presented and discussed.

Professor Steinar Nordal
Advisor

Preface

The work presented below is a Master's thesis in TBA4900 Geotechnical engineering at Norwegian University of Science and Technology (NTNU). The work has been carried out during the Spring of 2017, and is a part of the five-year MSc program in Civil engineering at NTNU.

The topic of the thesis has been proposed by NGI (Norwegian Geotechnical Institute) and the problem definition has been further adapted in cooperation with the supervisor at NTNU and the student. The idea has been to further highlight the topic in question, namely unwanted effects from pile drilling, through a side by side comparison/evaluation of software analyses and field measurements. The aim is to make a good foundation for further investigations on the topic, which is still relatively new in geotechnical circuits.

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Trondheim, 06.06.2017

Sondre Sagmoen

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Acknowledgements

Summary

In the latter years, there has been an increasing interest in unwanted settlements and damages occurring on neighboring infrastructure due to foundation work. There has been a research project going from 2012 to 2015 focusing on problems regarding this, called BegrensSkade. This research project has amongst other causes identified drilling for piles and anchors as a cause to unwanted settlements and damages. Several negative effects due to drilling have been proposed, amongst them a local suction around the drill bit and mechanical disturbance around the pile-/anchor casing. These two effects are both considered to create settlements in the adjacent soil, either as loss of soil due to excess flushing or as direct disturbance (straining) and following reconsolidation of the surrounding soil.

The first effect can be related to the Venturi effect, which is an effect related to Bernoulli's equation on fluid flow. The Venturi effect implies that when constraining the cross-section of a fluid flow, the velocity will increase and due to energy and mass conservation, the pressure will fall. This creates a suction at the constraint. This effect is assumed to occur when drilling in permeable non-cohesive soil layers, such that both excess water and soil is flushed out through the casing when drilling with air or water as driving force and flushing fluid.

The second theory is a continuation of the known volume displacement and straining due to pile driving. It is assumed that some straining can occur on the soil adjacent to the pile, and this will further lead to volume reduction when reconsolidation. This theory, as opposed to the Venturi effect, is a secondary effect due to the time duration of reconsolidation.

These effects have been evaluated trough case studies on two recent construction projects, the Hobøl River bridge and Gladengveien 10, and implementation in Finite Element Analysis. From pore pressure measurements and settlement measurements in the two projects, considerations on what is most likely to have caused the measurements, have been made. The effects have further been tried implemented in the FE program PLAXIS, to see what parameters one could manipulate to get the measurement results resembling the two cases.

The straining and reconsolidation has been implemented as a volume reduction in a zone adjacent to the casing. The results from FEA shows that a considerable large zone must be effected to create settlements near the measured settlements. As drilling piles is considered a cautious pile installation technique, it is considered that the applied strains and size of effected zone is too large to be a reasonable explanation. The results from the FEA on the Venturi effect is however more interesting. As the Venturi effect is considered a temporary effect, it could

better explain the temporary pore pressure reductions seen in the projects. Through the FEA, it is observed that the large sudden drop in pore pressure could occur when varying some of the parameters.

The parameter analyses on the Venturi effect shows that the boundary conditions in the FE model is the most interesting when evaluating the pore pressure distribution in radial direction. Relating this to in-situ conditions, the key points are the hydrogeological conditions in the firm masses between a clay layer and the bedrock. This is considered the most interesting topic for further investigations regarding the risk of loss of soil mass around the drill bit.

The representative suction in PLAXIS is modelled as a drain, such that this action will only affect the groundwater. Thus, the relationship between groundwater flow and possible loss of soil masses should be given further interest. The results in PLAXIS show that the flow gradients close to the applied drain are very large, and this could also be subject for further investigations.

Sammendrag

I de senere år har det vært en økende interesse for uønskede setninger og skader på omkringliggende infrastruktur som følge av grunnarbeid. Fra 2012 – 2015 har det vært et forskningsprosjekt, BegrensSkade, som har sett på dette temaet, og blant annet identifisert boring for peler og stag som årsak til uønskede setninger og skader. Det har blitt antatt flere negative effekter som følge av boring, blant annet et lokalt sug rundt borkronen og omrøring av jorden rundt foringsrøret. Disse to effektene antas begge å skape setninger i omkringliggende jord, enten som tap av jordmasser på grunn av suget rundt borkronen eller som følge av direkte forstyrrelser (tøyninger) og påfølgende rekonsolidering av omkringliggende jord.

Den første effekten kan relateres til Venturi-effekten, som er en effekt relatert til Bernoullis likning for strømning av fluider. Venturi-effekten antyder at ved en innsnevring i tverrsnittet for en strøm, vil hastigheten øke og på grunn av energi- og massekonservering vil trykket falle. Dette skaper et sug ved innsnevringen. Den samme effekten antas å oppstå når det bores gjennom permeable, ikke-kohesive jordlag, slik at både ekstra vann og jord blir spylt ut gjennom foringsrøret når det bores med luft eller vann.

Den andre teorien er en videreføring av den kjente volumfortrengningen som følge av peleramming. Ved boring av peler antas det at noe av den lignende forstyrrelsen kan forekomme, og dermed gi tøyninger og påfølgende rekonsolidering av jorda. I motsetning til Venturi-effekten, er denne teorien en sekundær effekt da rekonsolidering er avhengig av et lengre tidsperspektiv.

Disse effektene har blitt vurdert gjennom case-studier på to nyere byggeprosjekter, Hobølelva bru og Gladengveien 10, og deretter forsøkt implementert i Elementanalyse. Fra poretrykk- og setningsmålinger i de to prosjektene er det vurdert hvilken effekt som mest sannsynlig kan forklare disse. Effektene har videre blitt prøvd implementert i Elementprogrammet PLAXIS, for å se hvilke parametere man kan manipulere for å få lignende resultater som i prosjektene.

Tøyning og påfølgende rekonsolidering er implementert som en volumreduksjon i en omkringliggende sone rundt foringsrøret. Resultatene fra Elementanalyser viser at en betydelig stor sone må påvirkes for å få setninger i nærheten av målingene fra Hobølelva bru og Gladengveien 10. Ettersom boring regnes som en forsiktig peleinstallasjonsteknikk, antas det at de påførte volumtøyningene og sonestørrelsene er for store til å representere en virkelig situasjon. Resultatene fra Elementanalyser på Venturi-effekten er derimot mer interessant.

Ettersom Venturi-effekten betraktes som en midlertidig effekt, kan den bedre forklare de midlertidige pore-trykksreduksjonene som er sett i prosjektene. Gjennom Elementanalyser er det observert at den raske poretrykksreduksjonen kan forekomme ved manipulering av noen av parameterne.

Parameteranalysene for Venturi-effekten viser at grensebetingelsene i PLAXIS-modellen er mest interessante ved vurdering av poretrykksfordelingen i radiell retning. Med bakgrunn i dette er det grunn til å tro at hydrogeologiske forhold i de faste massene mellom leire og berg er viktig å ta i betraktning ved borevurderinger. Dette anses som den mest interessante observasjonen for videre undersøkelser på risiko for tap av jordmasse rundt borkronen.

Det representative suget i PLAXIS er modellert som et dren, og dette vil kun påvirke grunnvannet. Forholdet mellom grunnvannstrømmen og mulig tap av jordmasse bør derfor gis ytterligere interesse. Resultatene i PLAXIS viser at strømningsgradientene i nærheten av foringsrøret er svært store, noe som kan danne grunnlag for videre undersøkelser.

Content

1 Introduction

This thesis is centered around settlements occurring on neighboring infrastructure due to drilling for piles and anchors. The problem in question has been of increasing interest the latter years, probably due to rapid increase in construction work in already dense populated areas as well as difficult soil conditions. From 2012 to 2015 there has been a project concerning this, called BegrensSkade (in English: limiting damage), where a total of 23 partners from the Norwegian construction industry have participated (Veslegard and Simonsen 2013). Through the project, they have gathered several cases with reported damage and identified the assumed causes for the damage or settlements done to neighboring property/infrastructure. With basis in this they have come up with suggestions to improvements regarding the different causes.

The interest in this thesis is especially one of the causes identified by BegrensSkade: drilling of piles and anchors. Drilling of piles, as opposed to driving, is carried out to avoid the known unwanted effects which follows when driving piles. These effects occur mainly due to the mass displacement of the soil because of the direct installation of piles. Drilling however aims to flush out the same volume of soil as the pile volume. The disturbance of neighboring soil, and hence the neighboring infrastructure, is then assumed to be minimal. However, amongst the many reported incidents, it turns out that there have been larger settlements than expected or other unexpected damage, both in short-term and long-term perspective. These settlements/damages are probably a result of one of the unwanted effects of drilling identified by BegrensSkade (Simonsen 2015):

1. Change in pore pressure and groundwater table:
   - Pore pressure increase due to mass displacement and/or uncontrolled blowout of pressured air in the ground.
   - Temporary pore pressure reduction because of flushing with air pressure during drilling.
   - Long term drainage/leakage along the pile/strut.
2. Disturbance of clay with following reconsolidation due to direct mechanical impact and mass displacement.
3. Suction/flushing of mass and volume loss due to:
   - Local suction around the drill bit because of flushing mainly with air. This is resembling the "Venturi effect".
   - Collapse in bore hole (hydraulic fracturing).

- Erosion from flushing medium and/or flow of ground water into the casing.

In 1998 – 1999 there were several incidents at the same construction site at Helland in Vestfold, Norway. This is identified in a damage report done by Nordbotten (2001) for Statens Vegvesen. During pile drilling operations, they identified several large caverns in the soft clay, allegedly created by hydraulic fracturing from using air when flushing the bore holes. Because the drilling operations were done in the winter, there was a frost layer on top of the hole, such that the hole was suddenly revealed when this frost layer broke. Luckily, no lives were in danger, but one of the holes was as deep as 12 m. With less luck, things could have gone much worse. This is an example of the most critical consequence due to damage/settlements when drilling for piles. Other cases may not be so severe in the threat to HSE (Health, Environment, and Safety); instead there can be a large economic consequence due to stops in construction and the need to redo some of the work. The consequences need to be recognized and the effects should be looked more into.

The normal scientific approach would be to challenge one of the identified effects, or theory on why settlements occur, and do field investigations which would serve as a basis of discussion around the theory. However, for the execution of this thesis, there is not resources to do such investigations. First, drilling and soil investigation is an expensive field and without extensive cooperation with an entrepreneur it cannot be done. Second, the field investigation should also have been done in such an extent that one could vary the parameters (evaluating for instance the effect of soil layering, different drilling methods etc.). There has been attempts to create such a cooperation, but due to the extensive work it will take, it will not be done in the period of this thesis. Thus, some other approach will be done to further investigate/highlight the problem.

The assumed effects are all presented by Simonsen (2015), and seeing how comprehensive it would be to assess all of them, two of the effects are chosen to evaluate in this thesis. This will be 1) Local suction around the drill bit because of flushing mainly with air (Venturi effect) and 2) Disturbance of clay with following reconsolidation due to direct mechanical impact and mass displacement. The choice of Venturi effect as a probable cause is done because the idea is relatively new, and has not been assessed in a large extent yet. Disturbance and following reconsolidation is chosen partly as a continuation of the work done previous at NTNU by Borchtchev, Eiksund et al. (2015). The reason for choosing two effects is also to evaluate them against each other, and in relation to measurement data collected from BegrensSkade.

The evaluation of the drilling effects is done with literature survey on the effects, observations from construction projects that may or may not substantiate them, and with suggested implementation in Finite Element Analysis (FEA) with the software PLAXIS. The FEA is the main emphasis in this thesis, with the first parts creating a discussion foundation. The overall aim is to make some further observations on what parameters are essential to the effects, and if FEA can help explain the two assumed drilling effects. Thus, the thesis can be divided into four main parts.

1. A theory chapter where the following is presented:
   - Down-the-hole (DTH) drilling of piles and anchors
   - The assumed Venturi effect
   - Mechanical disturbance and following reconsolidation
2. Presenting the two cases:
   - The Hobøl river bridge
   - Gladengveien 10
3. FEA with effects on one single pile
4. Comparison and discussions on FEA and in-situ cases

As the thesis is related to drilling of piles, there is need for a brief introduction to how piles (and anchors) are drilled. Because the two cases presented are both done with the down-the-hole drilling method, this is the only one of interest. Also note that throughout the thesis, the writer uses the terms pile and casing interchangeably when referring to drilling of a pile/casing, so the reader should not be confused by this.

Due to the experimental nature of the thesis, a conclusion will resemble more to a suggestion on further work.

2 Theory

The two effects evaluated in this thesis are the Venturi effect, which is assumed to create a suction around the drill bit, and the direct disturbance of an adjacent zone around the casing. Presenting the ideas behind these and doing some reasoning around how they affect the soil is given the main attention in this section. To understand the concepts, some knowledge on the drilling process is needed. The projects later presented are both done with the method denoted down-the-hole drilling, and the emphasis will thus be on this method.

2.1 Down-the-hole (DTH) drilling

When drilling holes for piles and anchors there is applied both a rotating and a vertical motion for penetrating the soil. The rotation of the drill string and the casing is driven from the top (ground level), and for DTH drilling the vertical force is applied through a hammer piston in the bottom of the hall, hence the term down-the-hole. With this direct contact with the pilot bit, there is generally no loss of transmitted energy as the hammer drills deeper, as is the case with a top hammer (HalcoRockTools 2016). Figure 2-1 illustrates how a drill bit can be composed. The DTH hammer piston applies force on the guiding device.
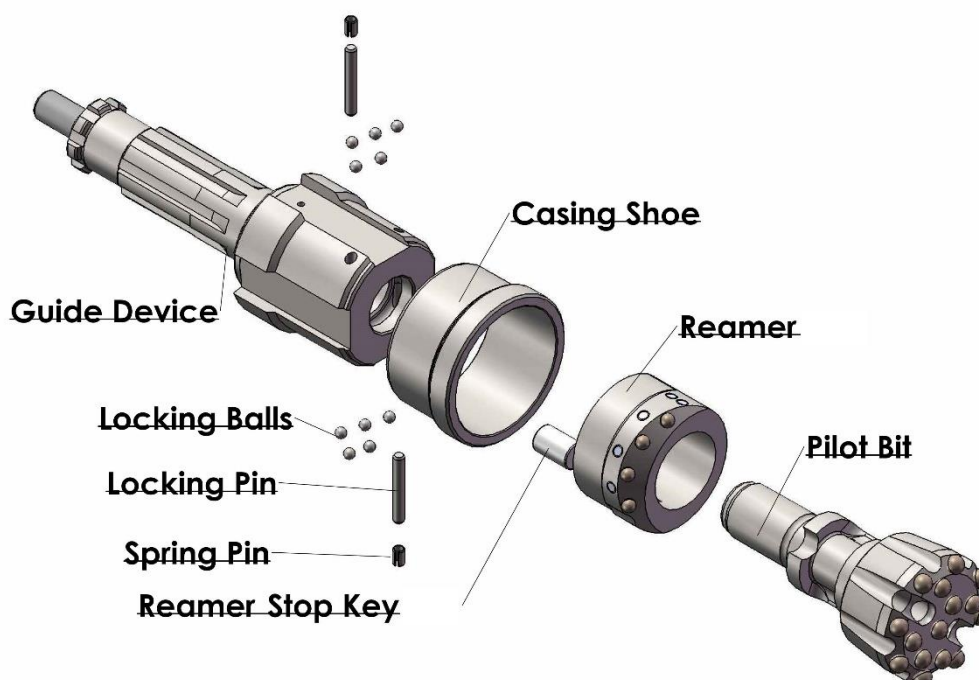

Figure 2-1: Illustration of the components in a drill bit. (Downloaded from Joytech 31.05.2017)

The DTH drill system is usually either centric or eccentric. The main difference is that the eccentric system uses an eccentric reamer which drills a hole of larger diameter than the pilot bit with the rotating motion. Figure 2-1 is an example of an eccentric system. A centric system on the other hand has the pilot bit attached directly to the casing shoe, and a ring bit is attached upon the pilot bit. This ring bit is of larger diameter than the casing shoe, and is ejected when drilling is completed such that the pilot bit can be retracted through the casing (Norsk geoteknisk 2012). Common for both is that they are overburden drilling systems, which means that they drill a hole with a larger diameter than the casing.

To drive the hammer piston, either air or water is used. The fluid is applied through a drill pipe connected to the drill string, and a compressor at the top provides pressurized air/water to create large enough force (HalcoRockTools 2016, Wassara 2017). As water is an incompressible fluid, the volume of water needed is independent of the wanted pressure. When using air however, a x times higher pressure demands x times higher air flow (in volume/time unit), i.e. the air pressure is dependent on the air volume. The exhausting fluid from a DTH hammer is used as the flushing fluid to clear the soil cuttings from the drilling operation. This is done by letting the fluid through the drill bit after driving the piston. The drill bits are created in such a way that the fluid flows in channels and flushes the cuttings/soil suspension upward inside the casing. The cuttings are then flushed out through the casing and gathered on surface level. Figure 2-2 shows an example of a drill bit with the air flow vectors. The drill bit and system in question is created by Robit. There are many different solutions to effectively clear the cuttings/soil suspension, all with the purpose to avoid unwanted disturbance to the soil.
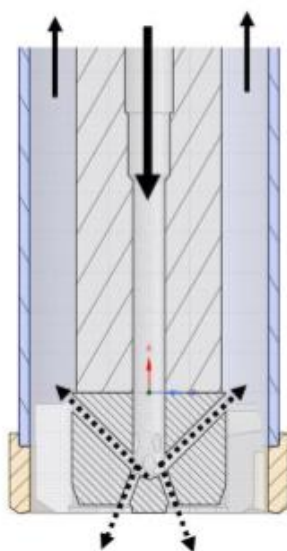

Figure 2-2: Example of a flushing system for a drill bit (Etteplan and Robit 2017)

2.2 Effects of drilling

2.2.1 Loss of mass and volume due to suction/flushing

The first problem identified here can be related to fluid mechanics with Bernoulli's equation and the Venturi effect. Bredenberg, Jönsson et al. (2014) describes this phenomenon when DTH drilling with air or water. As mentioned earlier one should have the same amount of soil being flushed out as the volume of the casing that is being installed. However, because of suction around the drill bit, the soil being flushed out can be larger than this. Bredenberg, Jönsson et al. (2014) has identified a case where the soil volume being flushed out is as large as ten times the casing volume.

Bernoulli's equation describes that for a flow in a closed system with no energy loss, both the mass and energy of the flow must be conserved (Al-Shemmeri 2012). Bernoulli's equation divides the energy contribution into three parts: kinetic energy represented through the velocity of the flow, potential energy represented through the relative height of the flow at a given time above a reference height, and pressure energy represented through the in-situ pressure of the fluid in question. With conservation of mass and energy the sum of these must be constant, giving Bernoulli's equation as:

$$\frac{P}{\rho g} + \frac{v^2}{2g} + z = constant \qquad\qquad eq.(\ 1\ )$$

P = fluid pressure (N/m^2)

ρ = fluid density (kg/m^3)

g = acceleration of gravity (m/s^2)

v = flow velocity (m/s)

z = height of flow above a reference height (m)

The Venturi effect exploits this energy conservation to create an under pressure by making a constriction on a tube and hence increasing the speed due to the law on mass conservation in a closed system. This increase in speed must be leveled with a drop in the static pressure in the fluid according to Bernoulli. Bredenberg, Jönsson et al. (2014) points out that some sort of the same phenomenon happens when flushing with air or water during drilling at large depths (with a sufficient distance to the groundwater table), and especially in fine grained non-cohesive soils, like silt or fine sand. Figure 2-3 illustrates the concept. When flushing with air, the return which is supposed to go up inside the casing will be a mixture of soil, water, and air. This new mixture will have a velocity greater than zero, while the surrounding water has no movement initially.

The water pressure will then be smaller inside the casing than in the surrounding soil, and this pressure difference is then considered to create a suction towards the drill bit and inside the casing. Then both additional water and soil can be sucked into the casing and the flushing return will be larger than expected. The following pore pressure decrease and loss of soil material will create settlements in the surrounding area.
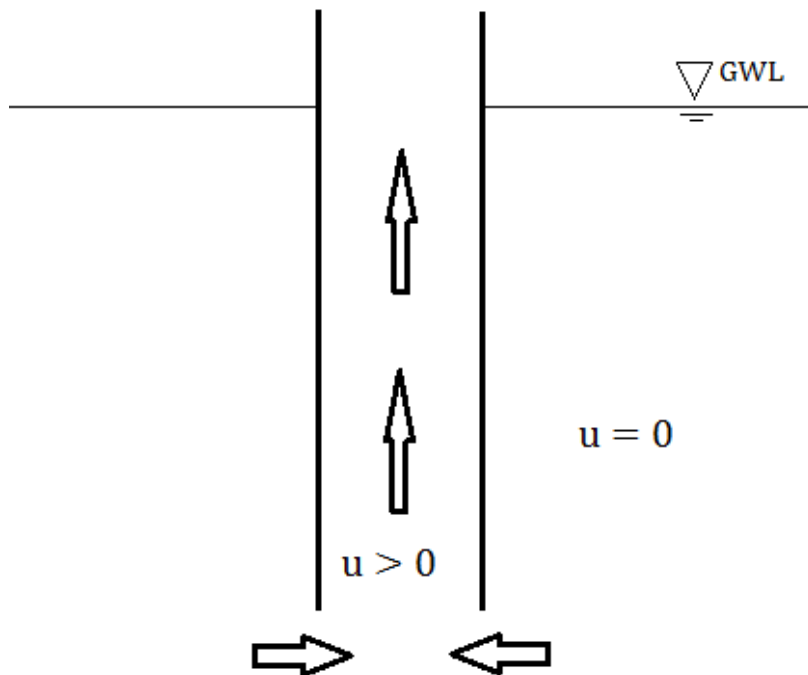

Figure 2-3: Sketch of Bredenberg, Jönsson et al. (2014) theory on flow into the pile casing

Bredenberg, Jönsson et al. (2014) propose an interesting idea on the utilization of the Venturi effect during drilling. This leads to the proposition of another way this effect could be valid. Instead of picturing the difference in velocity between the stationary water level surrounding the pile casing and the fluid flow towards the drill bit as the driving mechanism, the Venturi effect could be more directly applicable. Regardless of using only air or water, or a mix of both when driving the hammer and flushing the borehole, the fluid will be driven with either a known quantity Q or known pressure P. This, together with the known cross-section of the inlet channels in the drill bit can be utilized together with the cross-section of the flushing channels (slits/cuts) on the drill bit. Figure 2-4 shows this idea through a simplified sketch.
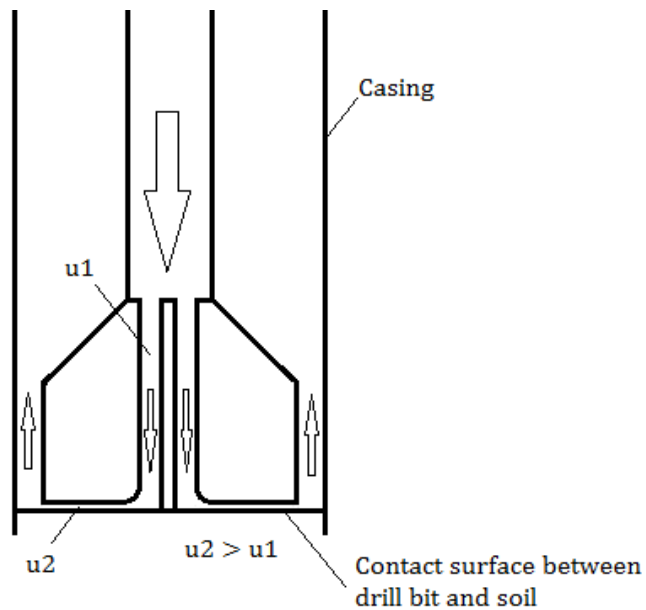
Figure 2-4: Sketch of a possible way to explain the Venturi effect

This is implying that the cross-section of the flushing channels is smaller than the inlet channels. This may be the case when experiencing clogging during drilling. Because some of the flushing-/return channels get clogged, the cross section of the return flow becomes smaller, and the return velocity may be larger than the inlet velocity.

To see if and how the Venturi effect will work, a small demonstration has been carried out. Figure 2-5 shows the principle sketch of the apparatus made.
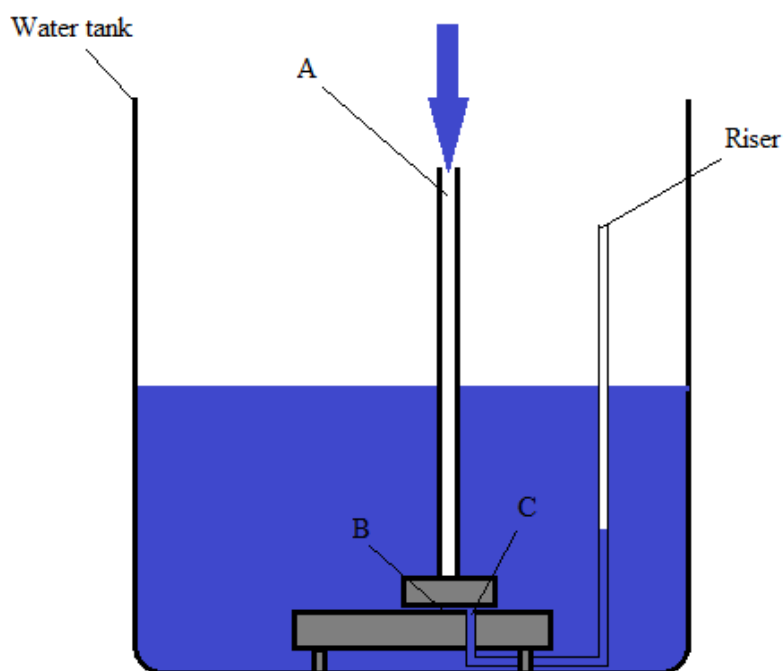

Figure 2-5: A sketch of the apparatus created to show the Venturi effect

What Figure 2-5 shows is that the water level (which represents the water pressure) is lower at a level of water flow with higher velocity. The water flow is applied in a tube with a known cross-section at point A. The outlet is at point B, where the cross-section of the flow is smaller such that the velocity becomes higher. This creates an under pressure, which is evident at point C with the lower water level in an attached riser. Due to limited resources and time, the model was not quantified in a proper way, but a suction did occur which could be seen with the lower water level. Hence, the possibility of a Venturi effect at a drill bit could not be discarded.

Both ideas for the application of the Venturi effect/Bernoulli principle can also be assessed as more complex than just relating the velocity and pressure. How the water, air, and soil mixture is composed will be a complex situation to comprehend. There should be a possibility to estimate this when applying air and/or water, but after hitting the soil in the bore hole this becomes more difficult. The density ρ will now vary as well as the velocity and pressure, which makes it even more difficult to quantify the magnitude of a possible suction. There will also probably be an energy loss along the way during flushing, which complicates the situation further.

Companies designing drill bits are aware of the risks related to high flushing pressures, and especially the consequence when flushing with air. Therefore, they make thorough dynamic analyses of the fluid flows to see how different designs will influence the flow velocity. The drilling company Robit has created a new Flow Control System to prevent air to escape into the ground. This is in relation to preventing the more known and more critical effect which is uncontrolled blowout of pressured air (as seen in the introductory example at *Helland bruer*). The Robit Flow Control System has been substantiated with computational flow dynamics in cooperation with CFD partner Etteplan (Etteplan and Robit 2017). Figure 2-6 shows the difference between the Robit Flow Control System and a conventional flushing design.
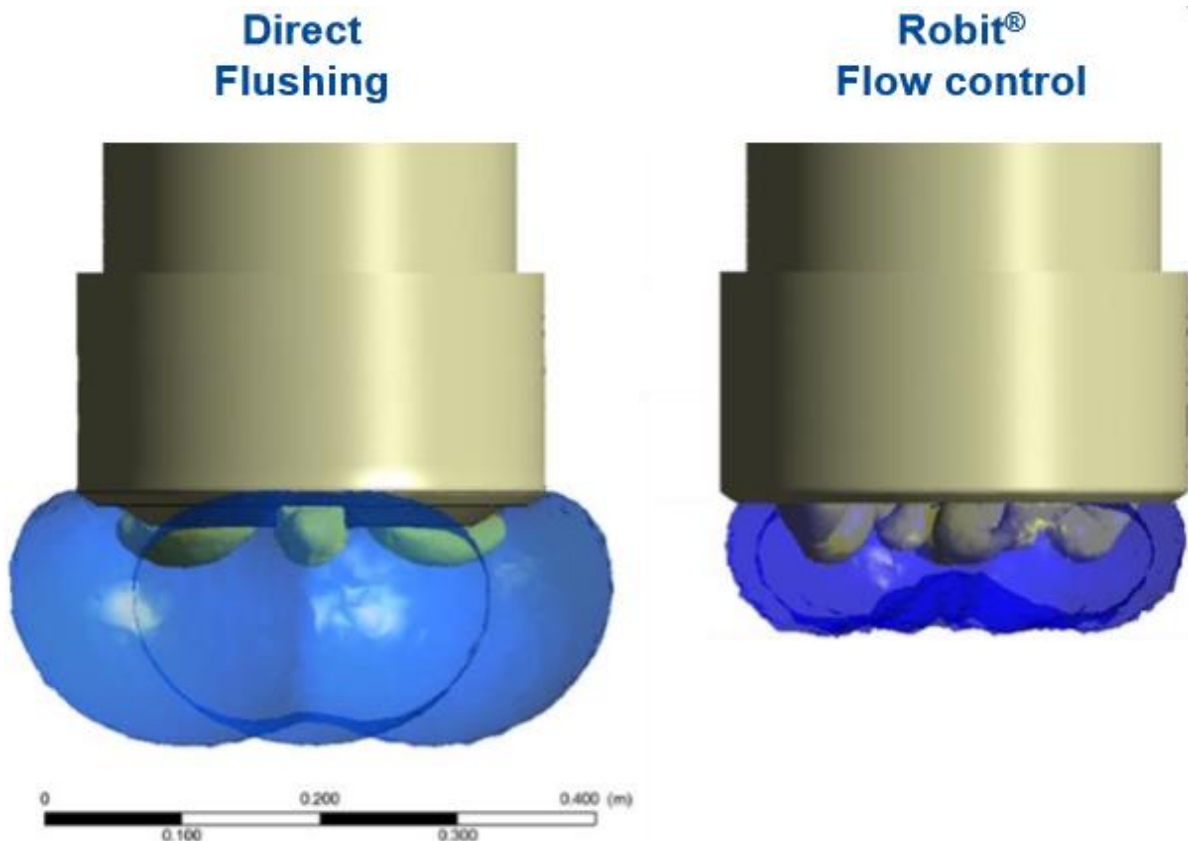
Figure 2-6: Robit Flow Control System vs Direct Flushing (Etteplan and Robit 2017)

What it shows is that the direct flushing system have a larger cavern size around the drill bit. This gives larger risk for uncontrolled blow outs, because the air-soil boundary is a lot larger. The different colors show the different air velocity isosurfaces. The yellow ones are of speed 25 m/s and the blue of 0.1 m/s. There can occur different velocities between these.

A CFD analysis on this subject is not an easy approach, and the theory behind it extends a lot further than the writers knowledge. The approach is a mathematical analogy, and is a modification from paper pulp mixing simulations, high viscosity shear thinning non-Newtonian flows (Moilanen 2017). Due to the selected analogy, the model is only applicable to relatively loose sandy soils. With the permeable moraine being of interest for the Venturi effect, the model fits relatively well, even though the exact classification of the moraine layers in question has not been done.

The question here however is to which degree the Venturi effect can occur at the drill bit, independent of the cavern size. If clogging occurs, then the cavern will probably be deformed, and most likely become smaller. The proposed idea presented in Figure 2-4 could then be applicable. This can also be the case if the soil mixture changes and becomes harder, also

making the cavern size smaller than previous. With the same air/water inlet, the velocity will then be higher than before Figure 2-7 shows the air velocity contours for the drill bit created by Robit, together with the key boundary conditions for the analysis. The visualization of the air velocity clarifies the drilling process, and hence helps understanding how the Venturi effect can occur.
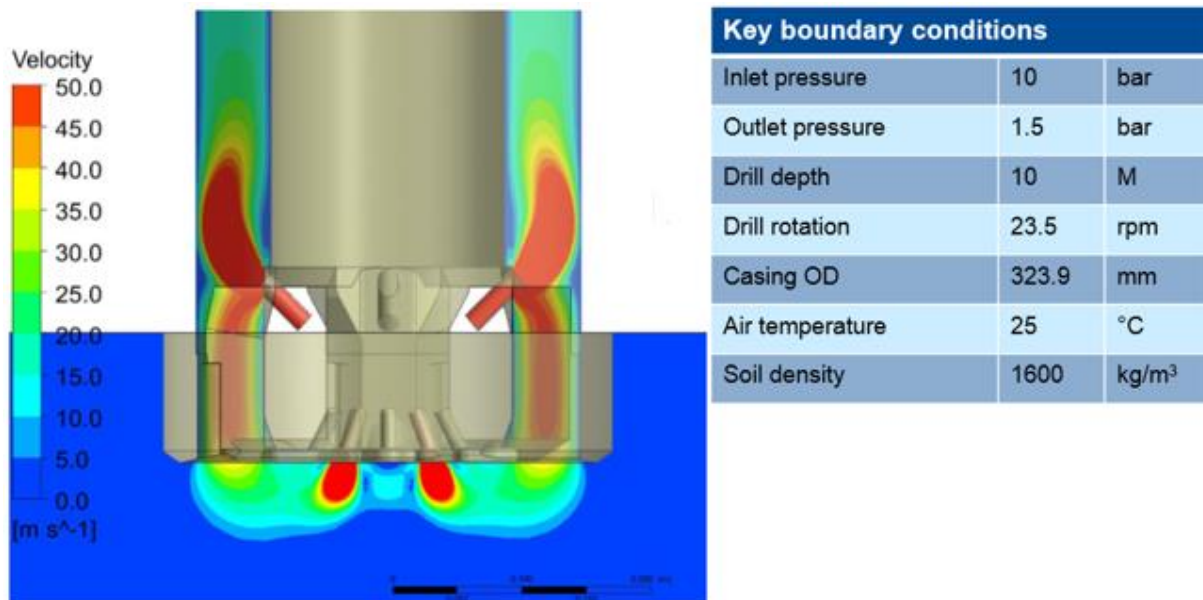

Key boundary conditions		
Inlet pressure	10	bar
Outlet pressure	1.5	bar
Drill depth	10	M
Drill rotation	23.5	rpm
Casing OD	323.9	mm
Air temperature	25	°C
Soil density	1600	kg/m³

Figure 2-7: Air velocity contours, together with key parameters for the CFD analysis (Etteplan and Robit 2017)

The comparison between what is assumed to happen around the drill bit during drilling and the Venturi effect/Bernoulli principle is very simplified, but the main idea with the pressure difference may be a good theory to the reason for settlements. As will be shown later when looking in to different projects, one of the common factors is that there has been a layer of some moraine material just above the rock bottom. This further substantiates the theory that a suction can be critical in a permeable soil layer.

If a suction is created due to reasons discussed above, the next question is then what consequence this will have for the adjacent soil. The Venturi effect is considered as most critical in non-cohesive soil layers due to both the permeability and the soil particle size. These non-cohesive layers are often found in between bedrock and clay layer, and a more common term for these layers are aquifers. Understanding the groundwater flow arising from a created suction and what this does to the soil skeleton is key to understanding the consequence the Venturi effect will have in the ground. This requires knowledge from several fields of science, like groundwater science, hydrogeology and of course soil mechanics. In this thesis, the

fundamental concept of groundwater flow from Fitts (2013) will be presented, which is later referred to and used as discussion foundation.

Rate of water flow through soil is based on Darcy's law, elaborated by the French engineer Henry Darcy in 1856. Darcy's law implies that the discharge rate Q is proportional to the head difference Δh (later denoted ΔH) and inversely proportional to the flow distance Δs. The discharge rate is also proportional to the cross-sectional area A, and the hydraulic conductivity K. Permeability k has historically been synonymous with hydraulic conductivity, and in this thesis, these are not treated differently, thus using the term permeability instead of hydraulic conductivity. Presenting an equation in differential form gives Darcy's law for one-dimensional flow:

$$Q_s = -K_s \frac{dh}{ds} A \qquad\qquad eq.(2)$$

The index s indicates the direction of the flow. Figure 2-8 shows a schematic illustration of steady flow through a sand sample.
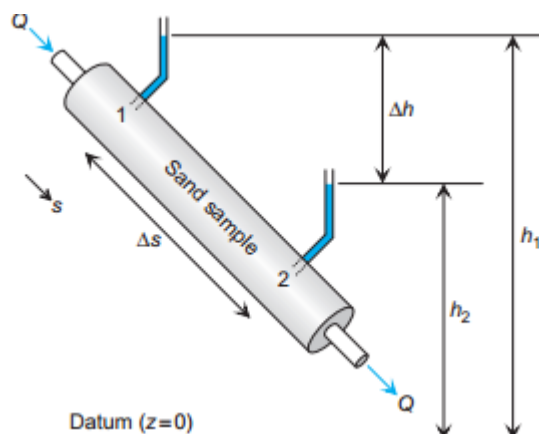

Figure 2-8: Schematic illustration of steady state flow through a sand sample from Fitts (2013)

The flow equation presented here is the simplest one, valid for a steady state one-dimensional flow in a fully saturated medium. When evaluating the groundwater flow arising from the Venturi effect, lots of considerations must be made. Perhaps the most interesting feature of aquifer layers is the amount of groundwater available. Baardvik (2015) has in relation with BegrensSkade gathered pore pressure measurements from several projects, and identified that there was large spread in the measurements, depending on the geological and hydrogeological conditions amongst other things. These geological and hydrogeological conditions could be thickness of the permeable layer, extent in horizontal direction of the permeable layer, and recharge rate in this aquifer layer, e.g. possibility of water infiltration from bedrock. Common

for these are that they are hard to determine for a large area, if not impossible (especially the recharge rate).

There are evidently a lot of factors and parameters related to the assumed Venturi effect. The aim is to evaluate and discuss several of these, if not all, and make some considerations on what will be most influential.

2.2.2 Mechanical disturbance and following reconsolidation

Direct impact from the drilling process on the surrounding clay is a known problem for driving piles, but when drilling piles this is mainly based on assumptions. When driving piles, the soil is supplanted and disturbance is a natural consequence. The disturbance can be measured in terms of strain on a soil volume around the pile, and there are several methods to quantify this impact, which are summarized by Langford and Sandene (2015). As these methods are based on the volume being supplanted they are not valid for drilled piles, and thus payed no attention in this thesis. However, the idea of a disturbed zone around the pile is continued to drilled piles.

As for the assumed suction effect described in section 2.2.1, the disturbance and reconsolidation has not been proven or quantified in any way. In addition to the question of the existence of this effect, this also leads to the speculation on how the drilling process directly effects the surrounding zone. One previous addressed theory is presented here.

The theory implies that the soil is directly influenced in the same way as when driving piles. This can be due to poor execution of drilling, with the consequence that some of the soil will be supplanted and thus creating strains in the same way as for driving piles. It is reasonable to assume that this can happen if the feeding force and penetration velocity is too high, such that the soil is supplanted instead of being flushed out because the drilling is executed too fast. Another option is that the flushing return has been forced up outside the casing instead of inside. What consequence this will have to the surrounding soil is not easy to assume, and it will also depend on the soil situation and condition around the casing. It is however reasonable to believe that also this case will create strains in the nearby soil.

In connection to these assumed effects to pile drilling, Borchtchev, Eiksund et al. (2015) tested the effects of pre-straining and reconsolidation of clay samples in a previous MSc thesis at NTNU. Through applying different degree of pre-straining, and then reconsolidating to different stresses they observed varying change in water content and accompanying volume change. The tests were done on clay from two different sites, Tiller and Stjørdal, where the samples were taken from different depths at the two sites. Table 2-1 and Table 2-2 summarizes the result from these tests, and Figure 2-9 shows the trends dependent on the degree of pre-straining.

Table 2-1: Impact of pre-straining and reconsolidation on water content and volume for Tiller clay (Borchtchev, Eiksund et al. 2015)

Sample ID	Applied shear strain γ_s, %	Re-consolidation stress p'_{re}, kPa	Water content prior to re-consolidation w, %	Water content after re-consolidation w_{re}, %	Decrease in water content due to the re-consolidation Δw, %	Volumetric strain due to the re-consolidation ε_{vol}, %
T1-H12	117	42.2	37.46	31.28	-6.18	8.48
		100	33.99	26.40	-7.59	11.49
		150	34.21	26.23	-7.98	13.91
		200	35.96	26.54	-9.42	16.61
T2-H15	66	35.6	34.61	31.29	-3.32	6.67
		100	26.61	22.20	-4.41	7.17
		150	31.67	26.61	-5.06	8.14
		200	33.14	27.36	-5.78	11.37
T3-H14	18	36.1	35.82	34.21	-1.61	5.04
		100	28.91	25.62	-3.29	8.85
		150	28.60	24.77	-3.83	9.45
		200	34.15	28.11	-7.03	13.24

Table 2-2: Impact of pre-straining and reconsolidation on water content and volume for Stjørdal clay (Borchtchev, Eiksund et al. 2015)

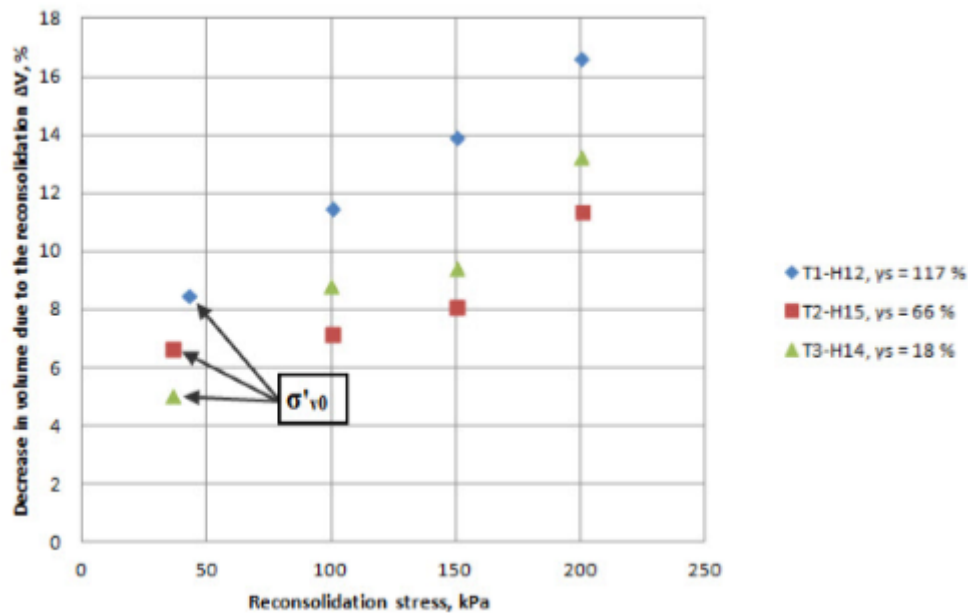| Sample ID	Applied shear strain γ_s, %	Re-consolidation stress p'_{re}, kPa	Water content prior to re-consolidation w, %	Water content after re-consolidation w_{re}, %	Decrease in water content due to the re-consolidation Δw, %	Volumetric strain due to the re-consolidation ε_{vol}, %
S1-H1	117	89.5	38.16	32.91	-5.25	9.59
		100	42.47	35.70	-6.77	11.46
		150	41.31	33.57	-7.74	13.16
		200	40.82	32.68	-8.14	14.39
S2-H1	66	75.1	30.74	27.53	-3.21	7.94
		100	30.01	26.39	-3.62	8.31
		150	30.47	25.97	-4.50	9.85
		200	32.56	26.22	-6.34	11.17
S3-H1	18	79.2	45.06	42.69	-2.37	6.61
		100	43.69	38.48	-5.21	8.92
		150	42.16	36.78	-5.38	8.66
		200	46.26	38.48	-7.78	11.32

Figure 2-9: Tiller clay: Volume change vs. reconsolidation stress (Borchtchev, Eiksund et al. 2015)

Borchtchev, Eiksund et al. (2015) identifies a linear trend for the volume reduction with increasing reconsolidation stress, which again is connected to increasing depth in an active soil pressure state. These results are probably as expected, as an applied stress state larger than the pre-consolidation stress should give volume reduction, thus questioning the theory. However, comparing the tests from the two different sites where the clay samples were taken from different depths, one can see that the volume reduction is larger for the samples at larger depths. This substantiates the theory on increasing volume reduction with increasing depths, with the trend also evident when reconsolidating to the in-situ stresses. Figure 2-10 displays a plot of the in-situ stresses and the accompanying volume strains observed to highlight this, where the Stjørdal clay is sampled from a depth of 6 - 8 m and the Tiller clay from a depth of 3 – 4 m.
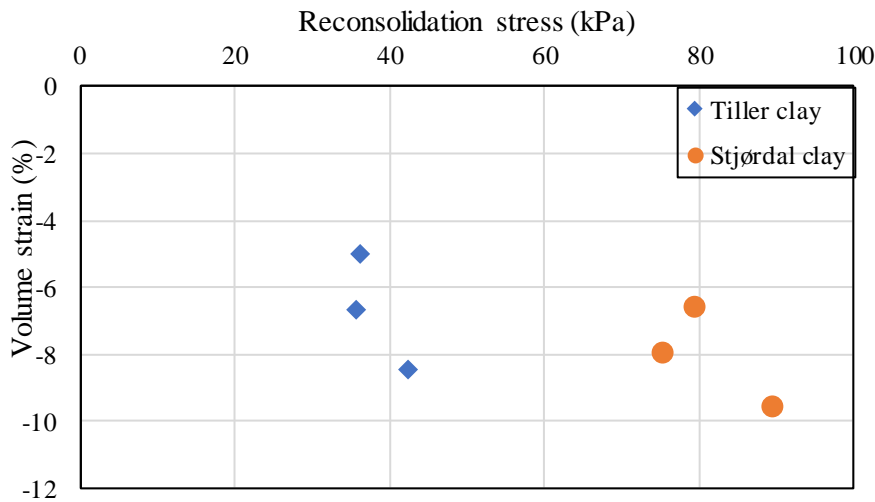
Figure 2-10: Volume strain for the tests reconsolidated to in-situ stresses

Seeing how reconsolidating to in-situ stress yields the observed volume strain even for a relatively small strain of 18%, the possibility of disturbance and following reconsolidation when drilling piles should be considered. The next question will then be to what extent a disturbed zone can be assumed. Methods have been suggested for evaluating this when driving piles, but with the drilling operation being less influential on the surrounding soil, this will for now be based on assumptions.

3 Cases

3.1 The Hobøl river bridge

The presentation of the Hobøl river bridge is a summary of the investigation done in relation to BegrensSkade, and for the full report reference is given to Haugen, Ahmed et al. (2015). The emphasis here is on the ground conditions, drilling method, instrumentation, and measurement results. The foundation work was done in 2014 – 2015. The Hobøl river bridge is a 290 m long concrete bridge with 9 bridge axes and 8 bridge spans, built with concrete foundations on casted steel piles drilled to rock. The dimension of the piles is 711x12.5 mm. Figure 3-1 shows drawings of the bridge.
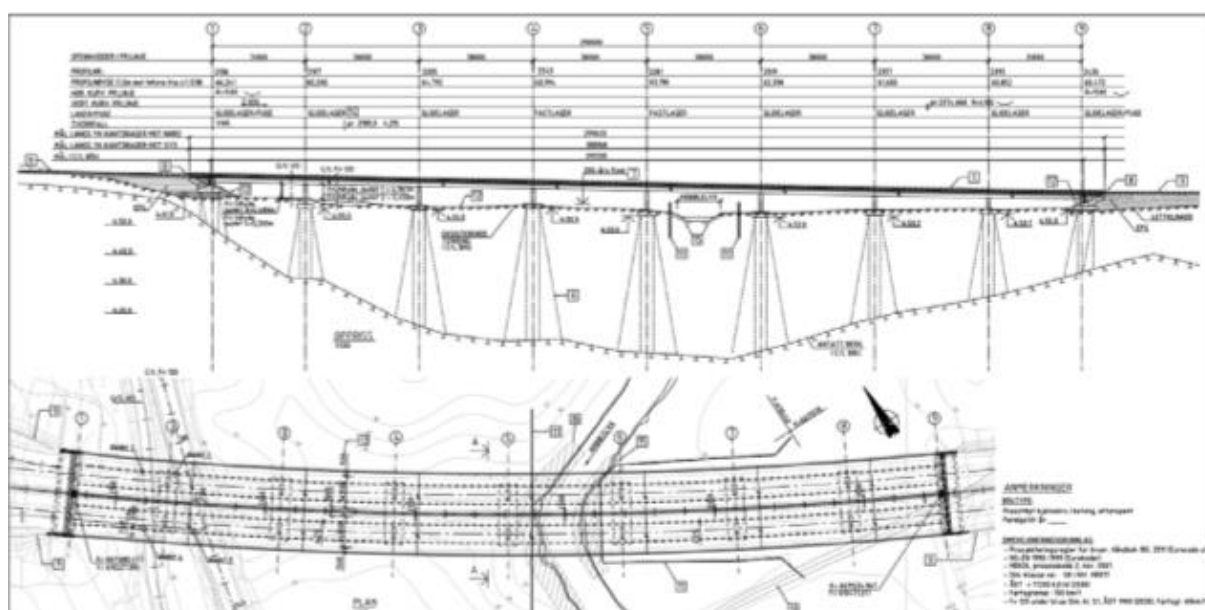

Figure 3-1: Longitudinal- and plan section of the Hobøl river bridge (Haugen, Ahmed et al. 2015)

3.1.1 Ground conditions

From Figure 3-1 it is evident that the depth to rock varies from west to east direction (left to right in the figure), with magnitude of 5-15 m in axis 1 to approximately 55 m in axis 5. At axis 9 the depth to rock varies between 20-30 m. This is found through interpretation of soundings, but during the pile works the depth to rock has partly been found to be larger.

The normal water level of the Hobøl river in the area is at level +53,5. The terrain level at the surrounding flood plains and slopes reaches from level +54,5 by the river, to approximately +60 at axis 1 in west and +57 at axis 9 in east. The groundwater level in the area varies between

1-4 m under terrain level, and is at its most shallow near the river. Pore pressure measurements reveal some over pressure, approximately 15% above hydrostatic pressure.

The ground layering consists of 2-4 m of dry crust and medium firm clay over soft and sensitive/quick clay. Down towards the bedrock a layer of firm moraine is detected, with the maximum magnitude of 5-6 m.

3.1.2 Foundation and pile drilling

Because of the difficult ground conditions with the soft clay there were need for pile installation methods which resulted in no mass displacement, pore pressure increase or erosion outside of the steel casing. However, at the time being, there were no available drilling equipment which could control the exact quantity of masses being flushed out. The requirement was thus changed from the strict description of "no mass displacement, pore pressure increase or erosion" to a description of "a minimum of disturbance, erosion and pore pressure change". Another important aspect to this case was that the piles were inclined piles and should be drilled with an inclination of either 6:1 or 10:1. Prior to this project, the construction of inclined piles with a length up to 50 m had never been done before, thus the entrepreneur had to be extra conscious.

The entrepreneur chose drilling equipment of the type DTH-ROX from Robit. This is a DTH system with a single use ring bit and a robust solution for the coupling of the pilot bit and ring bit. This system was considered as easier to handle for the drilling operator than other systems. Figure 3-2 shows the drill bit used together with the technical details from the supplier, while Figure 3-3 shows the concept of the flushing. The idea is that the flushing return in this way will find the way between the drilling string and steel casing.

DTH-ROX+ 711/16

Pilot, Down the Hole Hammer, Single-use
Piling, micropiling and underpinning

Casing OD mm	711
Casing OD inch	28
Casing Max Wall mm	16
Casing Max Wall inch	0.630
Ring bit ID mm	640
Ring bit ID inch	25.197
Ring bit OD mm	740
Ring bit OD inch	29.134
Pilot bit OD mm	672
Pilot bit OD inch	26.457
Hammer	20-24"
Product number	03-522-711-3995

Figure 3-2: Robit drilling system with technical details (Downloaded from Robit 22.03.2017)

Figure 3-3: Picture of the pilot bit with flushing and return channels (Haugen, Ahmed et al. 2015).

When drilling in clay there is not a great need for energy to drive the pile/casing. In clay, there is not the same need for water as in firmer masses, since the clay will easier be disturbed and flushing with air is sufficient. When reaching the moraine masses however, it is important to increase the use of water to make sure that the hammer and pilot do not get clogged. The challenges in this project was the areas where the moraine masses had large deposits of silt

fragments. These fragments will easily flow into the channels in the drilling equipment and the chances of clogging increases. The drilling resistance was also larger in these areas due to pore under pressure. The penetration rate has been as low as 3-4 cm/min here, as opposed to a normal rate of 10-15 cm/min in firm masses and rock. The solution here was to use a lot of water and minimum of air, and rotate slowly, so that the drill bit avoided getting clogged.

Some key numbers when drilling in this project are given under:

- Water was usually pressurized to 10 bar in the machine.
- During "normal drilling" in clay, the amount of water was approximately 250 l/min.
- During drilling in silt and silty sand, the amount of water was approximately 350 l/min and the use of air was minimized.
- During drilling in rock, the amount of water was approximately 300-350 l/min.
- The rotation of the drill bit was 3-4 rotations per minute during drilling in rock.
- Feeding force of 15-16 kN when drilling in rock.

3.1.3 Instrumentation

In cooperation with BegrensSkade there were installed 4 electric piezometers and 5 settlement anchors at axis 4. This was the deepest axis, and the potential unwanted drilling effects were probably considered to be largest here. The bridge foundation in axis 4 consist of 11 casted steel piles. The piezometers were installed in slightly different distance from one of the piles, and in two different ground layers, respectively clay and moraine. The settlement anchors were installed in different depths to see if there were any local difference with the depth, or if they all settled the same amount. The different instruments and depths are summed up in Table 3-1 and the plan view of the instruments relative to the piles are presented in Figure 3-4.

Table 3-1: Overview of piezometers and settlement anchors

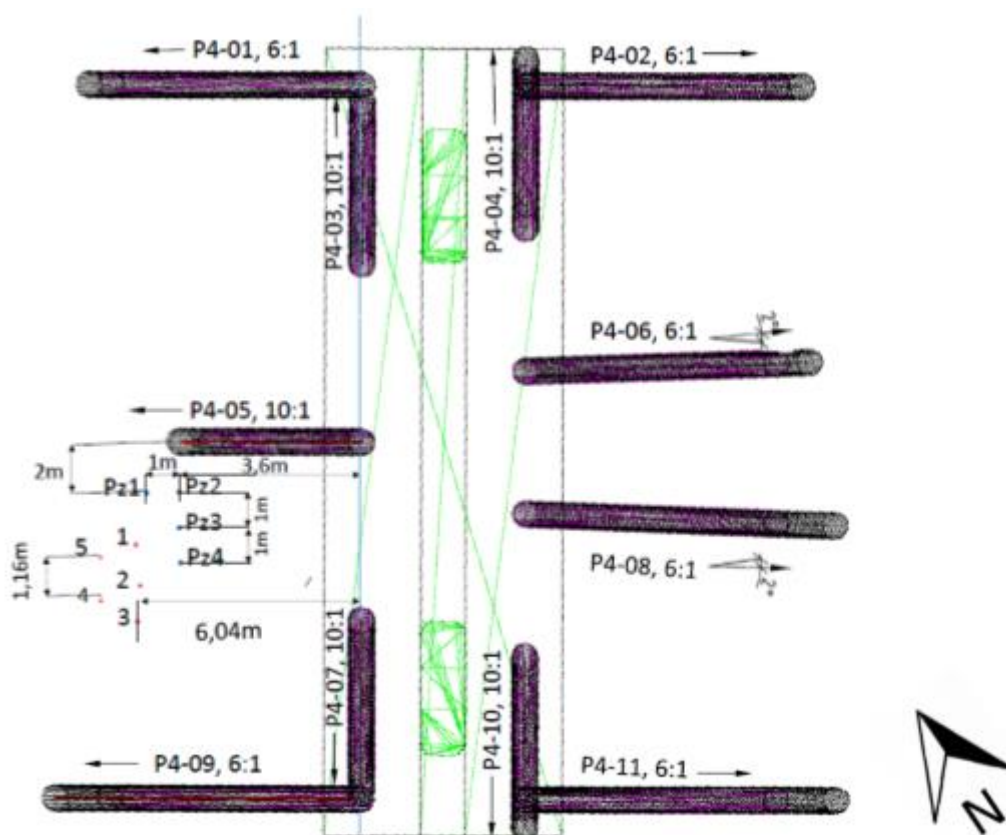| Meter	Depth (m)
PZ 1	43
PZ 2	36
PZ 3	36
PZ 4	42
S1	16
S2	26
S3	31
S4	36
S5	41

Figure 3-4: Plan for piles and instrumentation (Haugen, Ahmed et al. 2015)

In Figure 3-4 the piles are denoted with numbers P4-01, P4-02 etc., and the number next to it shows the inclination. As one can see, the instrumentation meters were closest to P4-05, and P4-09, and there was reason to believe that the drilling for these would have the largest effect on the measurements.

The pore pressure measurements were logged every hour in the period when they drilled in axis 5, 3 and 4. When drilling in axis 4, PZ1 was logged every half hour to better capture the

immediate effects. The settlement measures were performed almost every day in the period from 01.09.2014 – 19.11.2014 when drilling in axis 5, 3 and 4.

The order of the pile works was as follows (referring to the number of the axes): 1 → 5 → 3 → 4 → 2 → 6 → 7 → 8 → 9, with the more detailed dates and order around the time of pile drilling in axis 4 given in Table 3-2.

Table 3-2: Dates for drilling in axis 1- 5

Axis	Pile number	Date of drilling
1		16.06 – 01.07.2014
5		06.08 – 18.09.2014
3		23.09 – 14.10.2014
4	4-01	14 – 15.10.2014
	4-05	15 – 16.10.2014
	4-09	16 – 20.10.2014
	4-10	21 – 22.10.2014
	4-04	22.10.2014
	4-11	27 – 28.10.2014
	4-08	28 – 29.10.2014
	4-06	30.10 – 03.11.2014
	4-02	03 – 04.11.2014
	4-07	04.11.2014
	4-03	05.11.2014
2		11.11 – 24.11.2014

3.1.4 Results from measurements

Figure 3-5 shows the pore pressure development during drilling in axis 5. The piles in this axis is located approximately 40-50 m from the measurement equipment, and hence it is only natural that the impact should be small. However, some small temporary changes in the piezometers in the moraine layer can be observed.
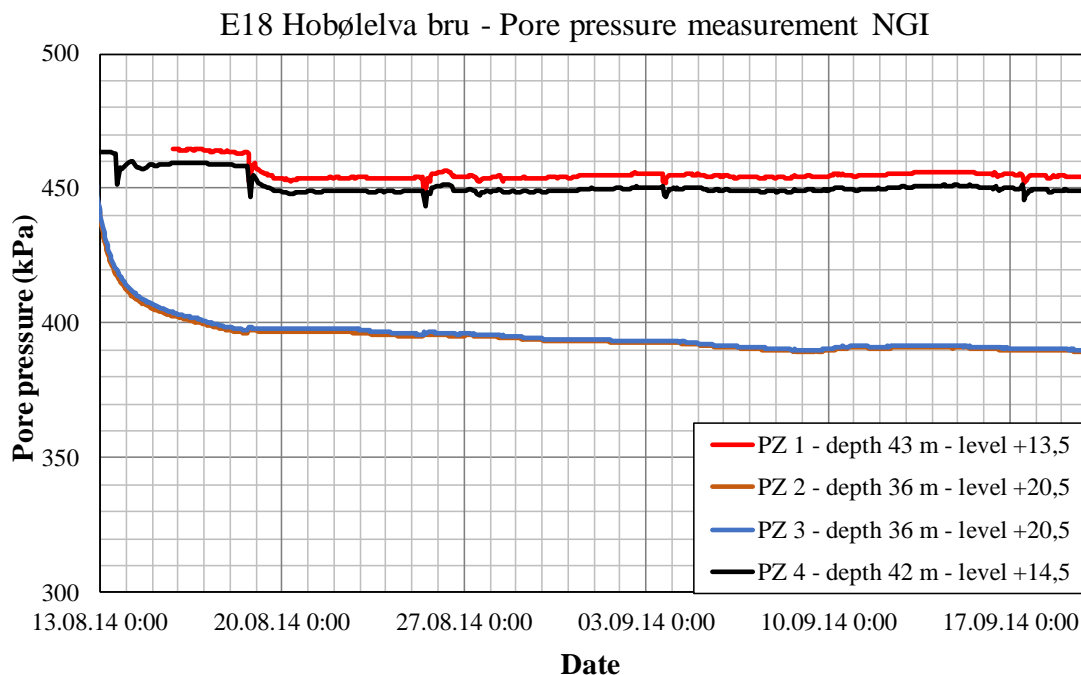

Figure 3-5: Pore pressure measurement during drilling in axis 5

There can evidently be disturbance in as large distances as 40 m, and this is substantiated when comparing to the results during drilling in axis 3. Figure 3-6 shows the pore pressure development during drilling in axis 3. This is also drilling conducted at 30-40 m from the piezometers, and the measurement shows as much as 65 kPa in pore pressure change. It is also important to remember that these measurements were logged every one-hour, and thus there could be even larger changes as well as more frequent occurrence.
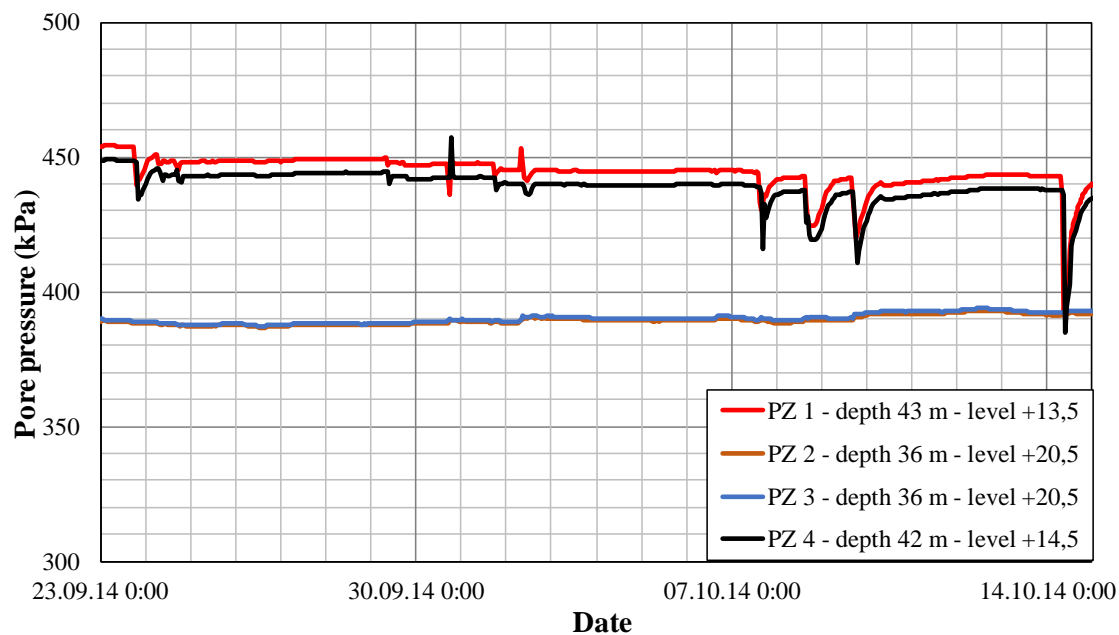
Figure 3-6: Pore pressure measurement during drilling in axis 3

Further, the effect of drilling on pore pressure stands quite clear when looking at the closer drilling operations. Figure 3-7 presents the pore pressure development when drilling in axis 4, and here one can see that the pore pressure fluctuates in both layers. Especially when drilling P-05 (around 16.10.14), which is the pile closest to the instruments, there is a large decrease in the clay layer (PZ 2 and PZ 3), and both a large decrease and increase in the moraine layer. The pore pressure change in the moraine layer assumes to be because of the effects presented in section 2.2. The rapid and temporary decrease which can be seen several times during drilling may be due to a suction around the drill bit, and thus a transient groundwater flow may occur in the moraine layer. This will be further discussed in relation to the PLAXIS analyses later. The pore pressure decrease in the clay is thought to be because of settlement in the moraine layer, creating settlement in the clay layer which again yields a suction in the clay.
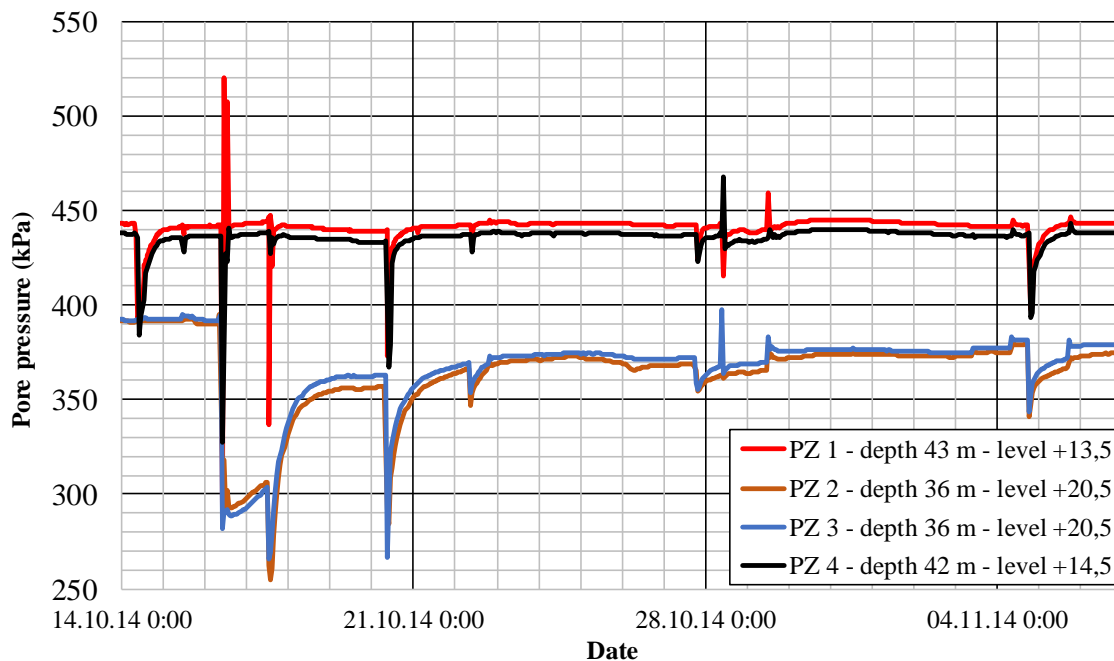
Figure 3-7: Pore pressure measurement during drilling in axis 4

Figure 3-8 presents the settlement in axis 4 during drilling for the closest axes as well as in axis 4 itself. In resemblance to the pore pressure measurements, the largest influence is during drilling for piles P-01, P-05 and P-09 in the period from 14.10 – 20.10, where the anchor at level 41 m has settled as much as 50 mm when reaching 20.10.14.
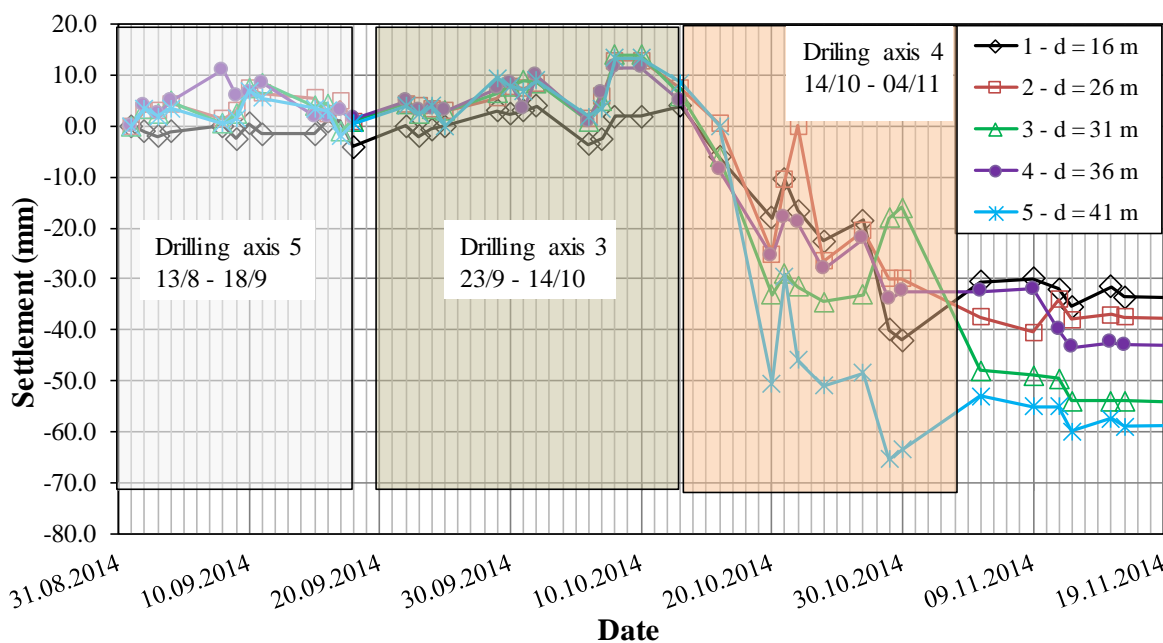

Figure 3-8: Settlement measurement for axis 4

It is especially the large pore pressure decrease during drilling for P-05 which is interesting. The operator commented that the pore pressure probably had been affected by stop and restarting of drilling from one day to the other. When stop in the drilling process at large depths, one should take the necessary measures for creating disturbance of the surrounding soil. The casing should be filled with water to create a counter pressure, and the pilot bit and hammer should be released from the ring bit to avoid that silt/clay clogs the channels for air- and water supply. At the Hobøl river the stop and restarting of the drilling took place in the clay layer at approximately depth 36 m, and it was during restarting that the large pore pressure reductions were observed. Figure 3-9 zooms in on these days when drilling for P-05. One interesting observation here is that PZ1 shows both decrease and increase, while PZ4 which is one meter shallower only shows decrease. It is especially interesting because PZ1 and PZ4 shows approximately the same measurement at 10:08 and 11:08, while the pore pressure increase in PZ1 occurs in between these as this is logged every half-hour. The same is also the case for the second pore pressure increase between 12:08 and 13:08. This could imply that the disturbance due to irregularities in drilling operations is more complex than expected, with even larger fluctuations.
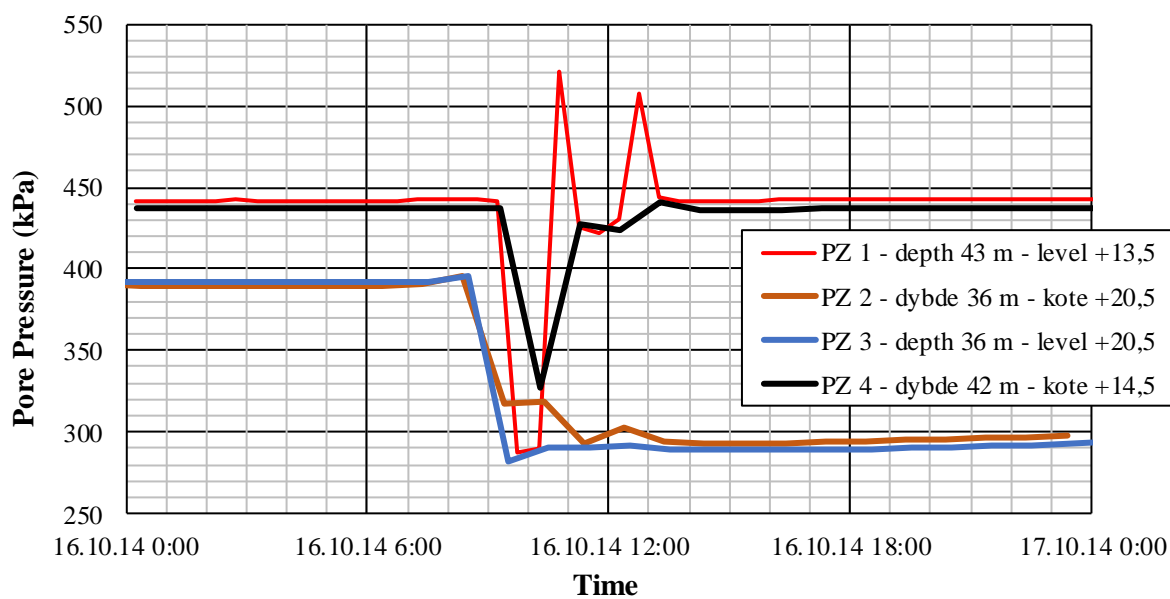

Figure 3-9: Pore pressure development at Hobbøl River project during drilling for P-05

3.2 Gladengveien 10

Gladengveien 10 is a housing complex consisting of 56 apartments over 7 floors, in addition to a parking basement. Like the Hobøl river bridge project, this project is only briefly summed up here, with weight on the settlement and pore pressure development during the drilling operations. For the full report, reference is made to Helgason (2015). During the project, there was conducted both drilling for anchors and for piles. The total area is approximately 1500 m^2 and the housing complex itself amounts to an area of 700 m^2. The building pit was excavated between 1 to 4.2 m, depending on the previous ground level and the wanted excavation level (the parking basement varies from one to two basement levels). The excavation should be conducted to respectively terrain level +59.95 and + 57.3 from the original level between +61 and +64.5 varying along the outskirt of the excavation pit. Figure 3-10 shows air photo of the construction area.

Figure 3-10: Air photo of the construction area (in yellow) and the building (blue) at Gladengveien 10 (Helgason 2015)

3.2.1 Ground conditions

Depth to bedrock varies between 8 and 40 m, with increasing depth from east to west. The top layer is 3 – 3.5 m deep and consists of gravel, rock, and dry crust clays. Beneath this layer, the soil consists of silty clay down to a depth of 9 m above bedrock. The clay is characterized as soft to medium firm.

The pore pressure measurement shows a groundwater level at +60.3 m, which is approximately 3 m below the terrain level. A hydrostatic pore pressure with this groundwater level is therefore assumed.

3.2.2 Foundation and pile drilling

The project consists of drilling for both anchors and piles. The anchors function as bracing for the sheet pile wall which is installed in part of the construction site. The plan for where the sheet pile walls are installed and where there is an open excavation is shown in Figure 3-11. The building rests on a total of 68 piles, with diameter dimensions 90, 100, 120, 130, 150 and 180. All the piles are drilled with ODEX air driven DTH-hammer.

Figure 3-11: Plan for the installed sheet pile walls at Gladengveien 10 (Helgason 2015)

The construction work started at the end of March 2014. Installation of sheet pile wall was done in April 2014 after excavation to first strut level. Drilling of casing for anchors was done in this phase. Then excavation to the wanted level as well as drilling casing for steel core piles was done gradually in May 2014. More detailed time history for events will be presented together with the measurements of interest.

3.2.3 Instrumentation

The instrumentation program for monitoring deformations and pore pressures were altered through the construction process, and consisted in the end of:

1) Deformation/settlement measurement in strategic positioned points outside the construction pit.
2) Pore pressure measurement in different depths in- and outside the construction pit.
3) Tilt measurement of the sheet pile wall

The interest in this thesis is mainly the two first points, and hence it is only these measurements that will be presented in the following. Three pore pressure meters (piezometers) were installed, with the location shown in Figure 3-12. Two of the meters were installed in the north end at depth 20 m (in clay masses) and depth 34 m (moraine layer) from terrain level. The last one was installed near the south end at depth 12 m. These were checked in the period from May 2014 to September 2014, which covers the foundation work, including drilling of piles. The settlement has been measured in a total of 21 locations, which are show in Figure 3-13.
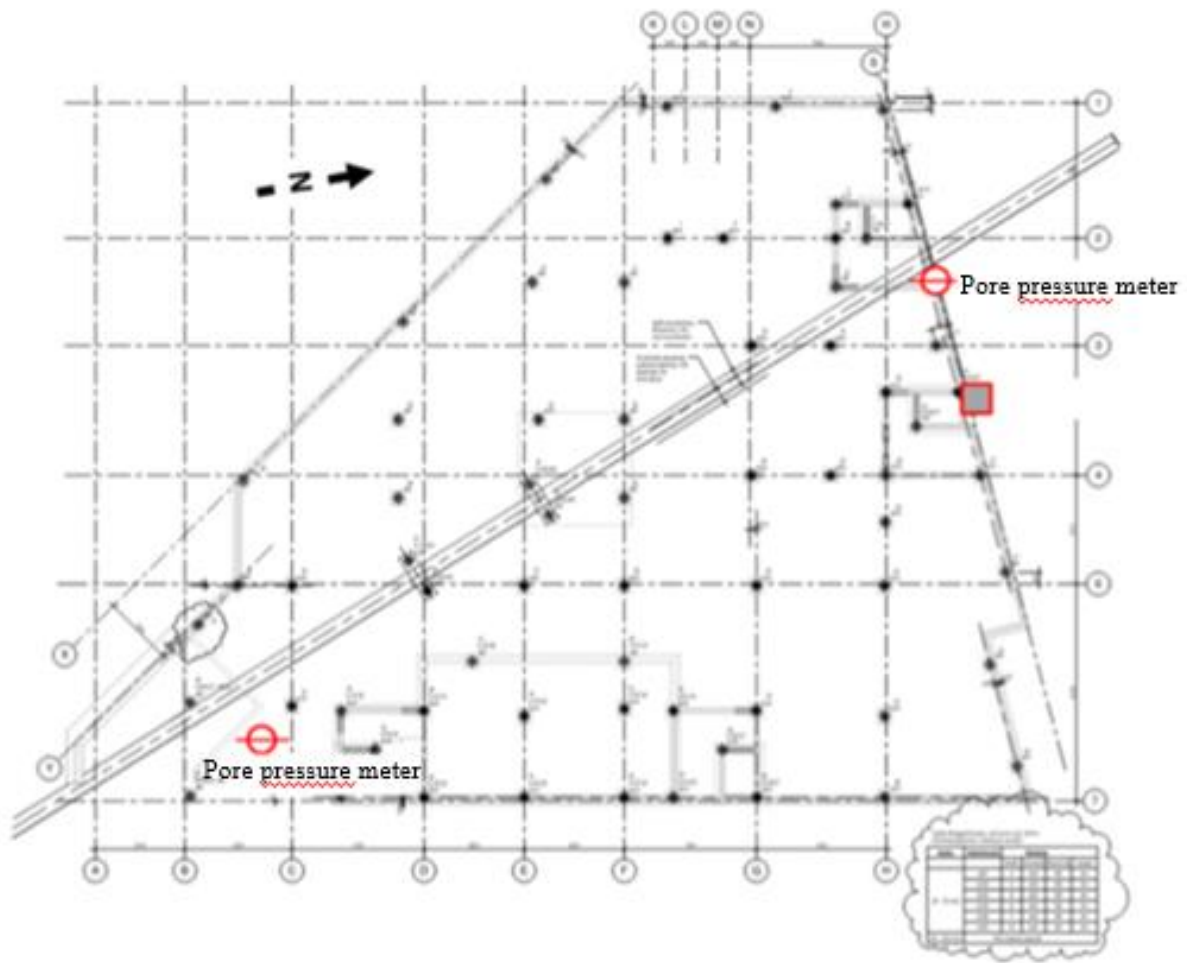
Figure 3-12: Indication of where the pore pressure measurements have been done (Helgason 2015)

Figure 3-13: Overview of the settlement measuring points (Helgason 2015)

3.2.4 Results from measurements

Figure 3-14 shows the pore pressure measurement for the piezometers installed at Gladengveien 10. It is evident that there are several temporary pore pressure reductions during the drilling operations when studying the pore pressure development for PZ2 and PZ3. Especially in PZ3, which is located in the layer between bedrock and clay, the pore pressure drop is as big as 70 kPa at most.
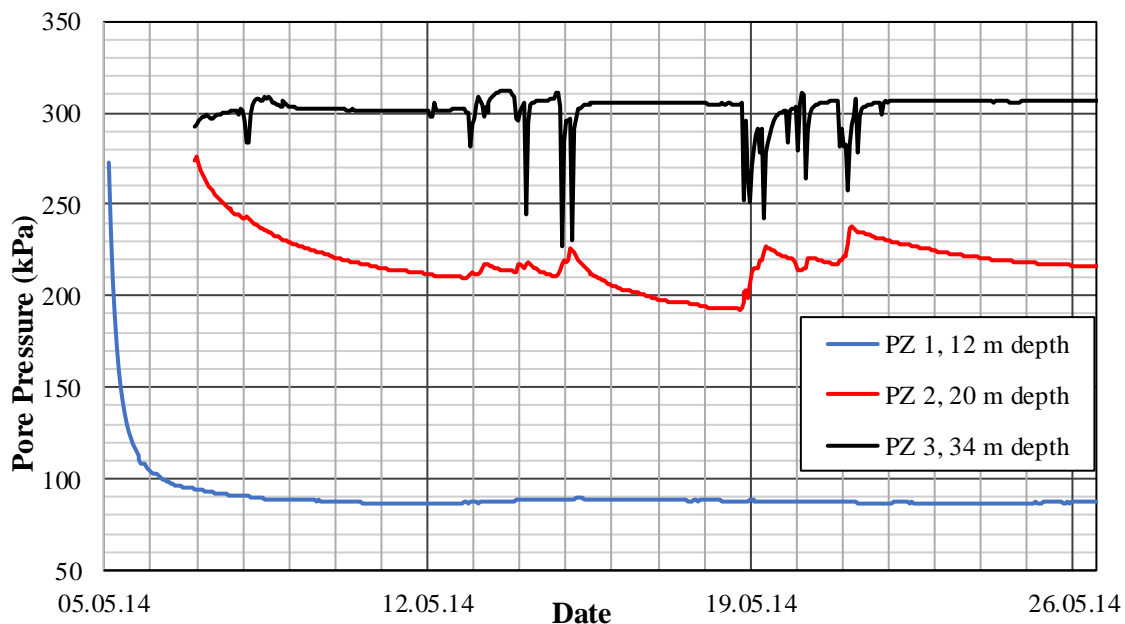
Figure 3-14: Pore pressure measurement during drilling

Comparing the results from the pore pressure measurements to the settlement measurements, the observations are even more interesting. Figure 3-15 shows the settlement development for the measuring points in Bertrand Narvesens vei.
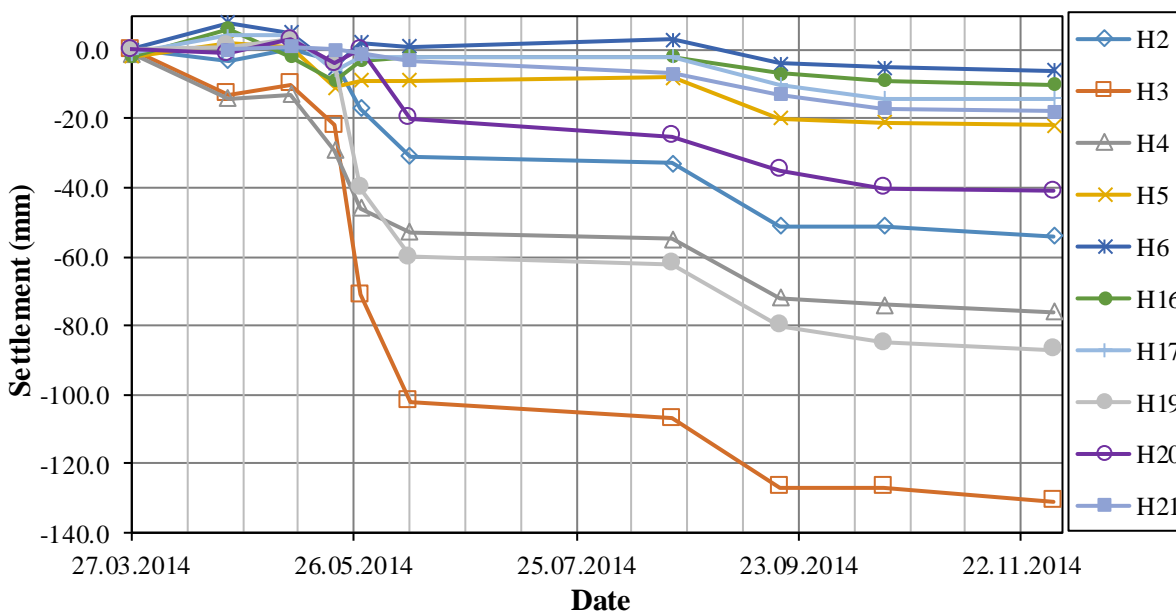

Figure 3-15: Settlements for the measuring points located in Bertrand Narvesens vei

The reason for not plotting all the settlement measuring points is that there is no registered settlement in point H7 – H15. H18 is not included as well because of stop in measurements in June 2014. It is also worth mentioning that the pore pressure meters PZ2 and PZ3 are located

close to settlement point H3 and H4, as seen when comparing Figure 3-12 and Figure 3-13. When evaluating this closer, the correlation between pore pressure drop and settlements are quite clear. There are especially two periods where the pore pressure fluctuated a lot: around 13.05.2014 – 15.05.2014 and around 19.05.2014 – 21.05.2014. According to the pile protocols (Holt-Risa and Østlandet 2014), there was no drilling in the period between 15.05.2014 – 19.05.2014. This could explain the steady pore pressure in this period. According to the protocols, pile number 6, 13, 17 and 19 were all drilled in the two periods with registered large fluctuations. It is also worth mentioning that the drilling operator experienced casing fracture when drilling pile number 17, and hence had to start over with this operation. The location of these piles is in the area where PZ2 and PZ3 are located. Figure 3-16 shows this with a close-up of the piling plan first shown in Figure 3-12.
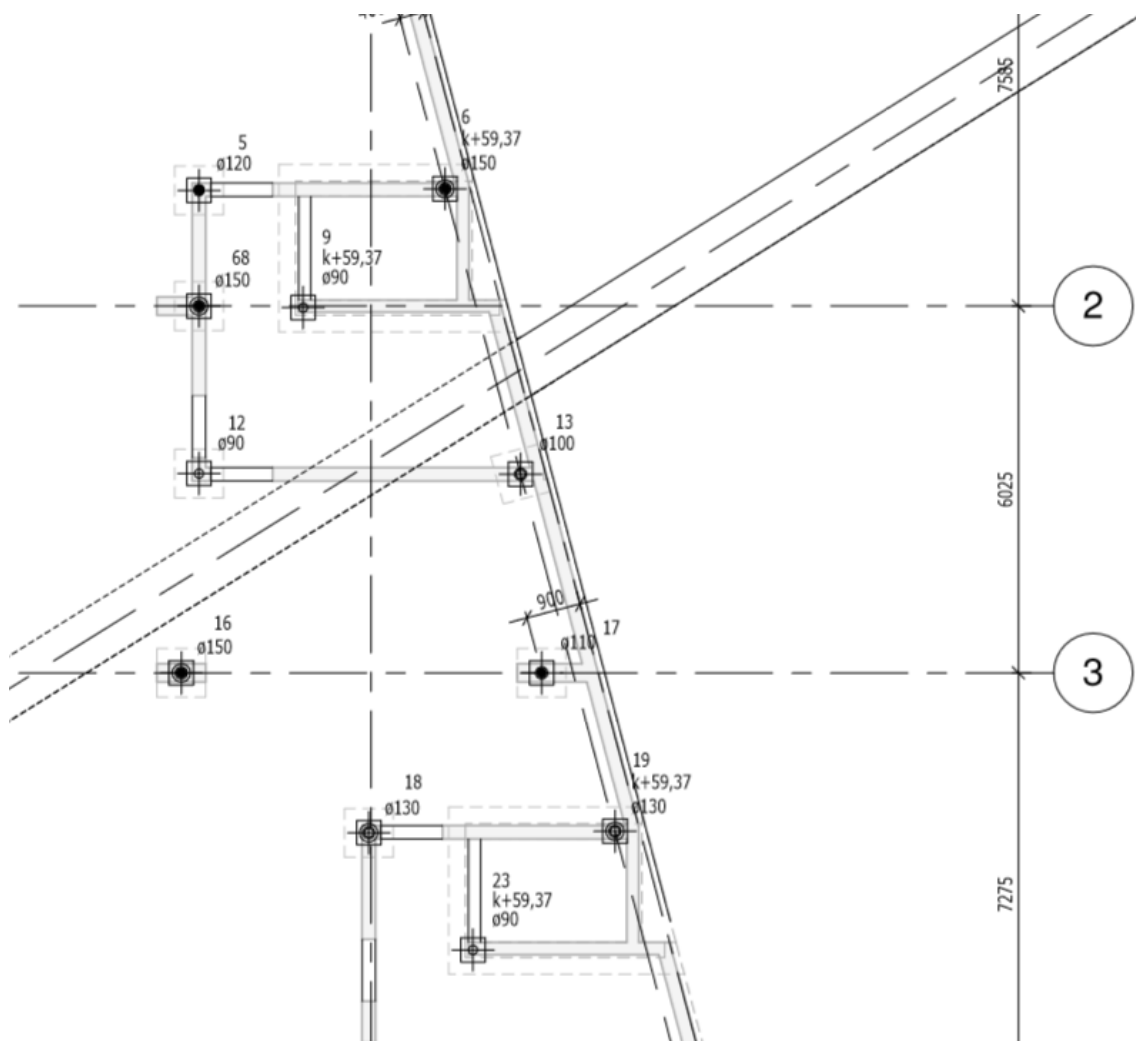

Figure 3-16: Close-up of the piling plan for Gladengveien 10, displaying the location of some selected piles

The pile drilling was executed during the period from April – May 2014 as mentioned above, and more precisely between 28.04.2014 and 22.05.2014. The settlement meters were in this period measured approximately every second week, with one assumption between 21.05.2014 and 28.05.2014. Figure 3-17 shows the settlement in the period around the drilling, where the dates for measuring is evident. Between 22.04.2014 and 09.05.2014 there was not registered any significant settlement, and that is why this period is not included. There is reason to believe that there is some correlation between the pore pressure fluctuations and the settlement response. From Figure 3-17 it seems like the settlement response is a bit delayed, evident with the change in especially H3 between 21.05.2014 and 28.05.2014, and further on to 10.06.2014. This same tendency is also observed for H4 and H19.
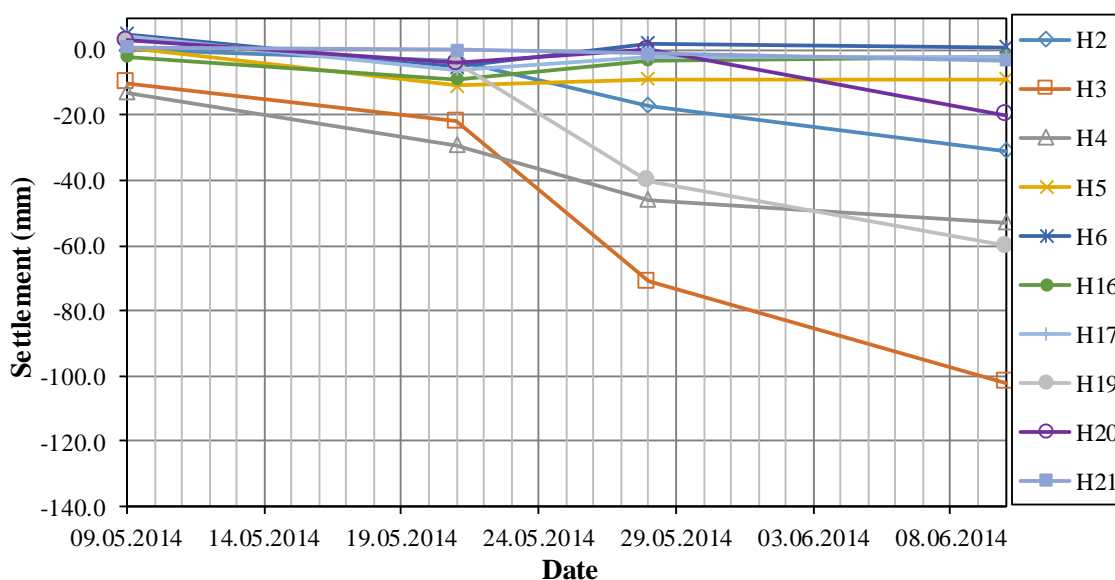

Figure 3-17: Settlement during pile drilling at Gladengveien 10

The settlements are larger than expected with a maximum of 130 mm in November 2014. One reason pointed out in the report is deviation when installing the sheet pile wall. There were registered large vertical deformations of the sheet pile wall, which probably was caused due to lack of contact between sheet pile wall and bedrock. This caused the sheet pile wall to settle a lot when tensioning the anchors. This again would lead to buckling of the sheet pile wall, which again will create settlements outside the excavation pit.

Another reason is related to irregularities during drilling. The drilling operator sometimes discovered that the drill bit was clogged, and thus used higher air pressure to prevent this. This can explain the fluctuations in pore pressure, which again yields settlements. This theory substantiates the idea about drilling effects, and will serve as discussion foundation later.

4 Single pile analyses

Conducting FEA to resemble the effects presented here requires a lot of simplifications. The use of PLAXIS to predict soil response needs good knowledge about the soil layering and soil parameters. The interest for these single pile analyses is not to recreate an exact in-situ case, but to observe what will happen theoretically when manipulating some parameters in a somewhat similar situation as in the cases presented. Therefore, the soil layering is simplified with one clay layer and one moraine layer to represent the ground conditions of interest. Parameters for both the soil layers are chosen arbitrary, and are only remotely representative for actual clay and moraine properties. With simplified cases and arbitrary soil parameters, a simple soil model is desired. In PLAXIS, this will be the Mohr Coulomb model, with only several parameters determining the soil response. For further interest in the software and soil models available, reference is made to PLAXIS Manuals (PLAXIS 2017).

To create the two different effects presented in this thesis, a lot of introductory analyses have been conducted to find out what is most reasonable. Finally, two approaches were chosen; one for each effect. General for both is that they are implemented with an axis-symmetric model, with the installed casing/pile being in the center and observations are made in radial direction.

4.1 Suction around the drill bit

The most interesting idea explaining the temporary pore pressure reductions is the Venturi effect. The main idea, as explained in section 2.2.1, is that during drilling with high flushing pressure a suction can occur around the drill bit. In the case of the Hobøl River bridge and Gladengveien 10 this possible suction could be observed through a large drop in the pore pressure which quickly rose back to the previous pore pressure. This suction is here attempted created with a pore pressure difference yielding groundwater flow.

The PLAXIS model is presented in Figure 4-1. It is built with two layers of the same depth 5 m, with a line load on top representing the weight from overlying soil. To create a similar situation as the observed cases, the moraine layer should be under a deep clay layer. Thus, the level starting at y-coordinate = 0 m in the model is thought to be at 30 m depth, hence the line load is of size 300 kPa. The data for the two layers is presented in Table 4-1, and the parameters are chosen arbitrarily, as it is only groundwater flow which is of interest for this case. The casing has been modelled as a plate with the parameters equivalent for a steel material.
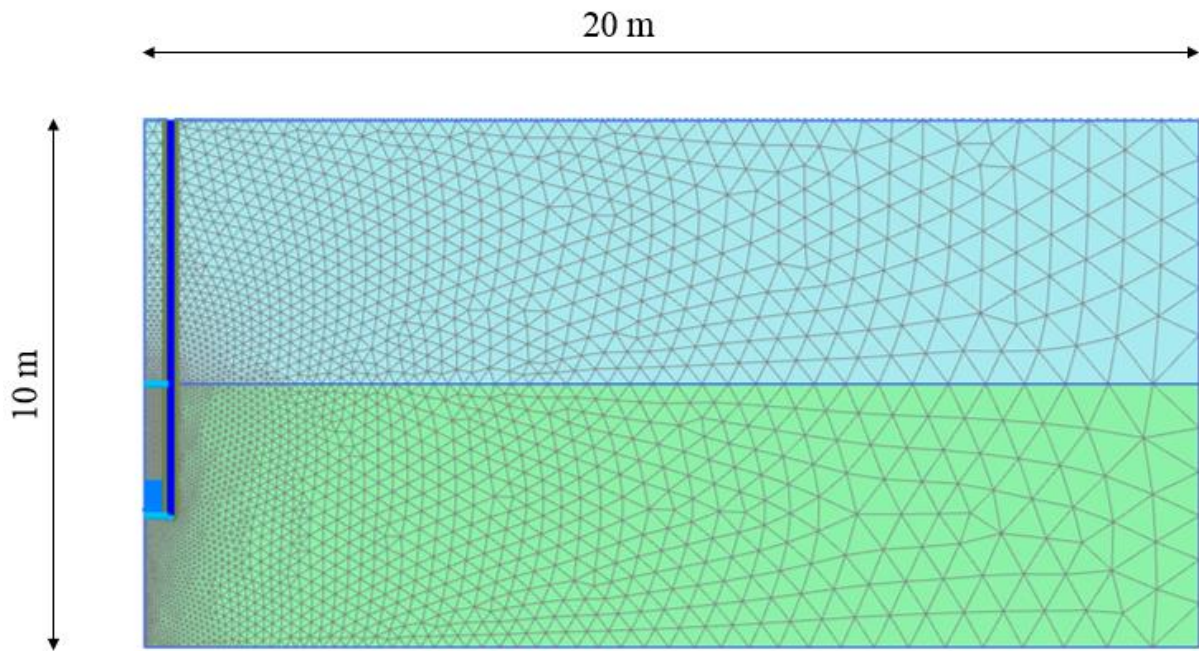
Figure 4-1: Plaxis model with the element contours

Table 4-1: Parameters for the two soil layers

Material	Clay	Moraine
Model	Mohr Coulomb	Linear Elastic
Drainage type	Drained	Drained
γ (kN/m^3)	20	20
E' (MPa)	3	30
ν	0.33	0.2
c' (kPa)	10	1
ϕ (rad)	25	40
k (m/day)	0.0001	-

The drilling process is modelled as an empty casing which is installed to a depth of 37.5 m, with a line-load inside representing the feeding force on the drill bit. The analyses are run with different phases presented in Figure 4-2.
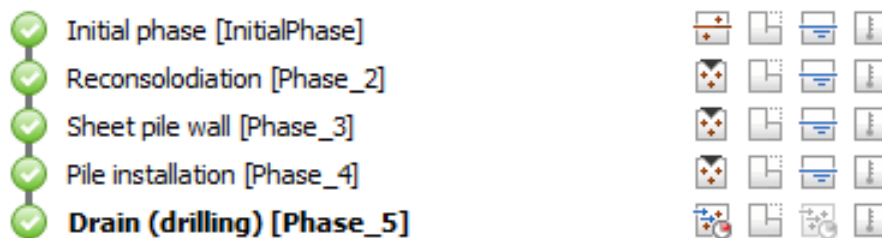

Figure 4-2: Showing the different phases during the analyses

In the initial phase, the ground load of 300 kPa is applied, and the reconsolidation phase after this resets the deformation. The sheet pile wall installs the pile casing, and the pile installation phase removes the soil inside the casing (the soil to the left for the pile wall) and activates the drilling load. The drain (drilling) phase is then either run as a plastic phase with steady state groundwater flow or as a fully coupled flow-deformation with given time interval to create the transient groundwater flow.

With the suction resulting from flushing with air/water being of interest in this analysis, the moraine is modelled as an elastic material, such that there can be "unlimited" groundwater flow without any fracturing. The interest here is to see what happens with the water flow, and the goal is to make some observations that can be of interest in a discussion on pore pressure observations and volume loss. To represent such a suction, a drain is installed in the bottom of the casing at level -7.5. This level is chosen because it is in the middle of the moraine layer. The drain is modelled with a vacuum behavior with a head of -7.5 m. The boundary at the right in the model has a boundary condition with head $= 30$ m, making the potential difference $= 37.5$ m. The left boundary and the bottom boundary are both closed. The distance from the center of the pile to the right boundary, denoted R, is 20 m as indicated in the figure. This is the default model, which is being used for most of the analyses, but also these parameters will be varied to see the influence of them.

The parametric analyses conducted here is done to see the effects of different key conditions related to groundwater flow. These parameters are the time span, size of potential difference, and influence of permeability and boundary conditions.

When plotting the results from these analyses, the groundwater head is chosen as the pore pressure indicator because of its direct connection to the applied potential difference. When looking at the development in the radial(horizontal) direction, the relative distance r/R from the pile wall is used. This meaning that $R = 20$ in the model, is manipulated to $R = 19.5$, such that the distance 0.5 m from the left boundary gives $r = r/R = 0$, and the right boundary $r = 19.5$ m gives $r/R = 1$. The relationship is explained in Figure 4-3.
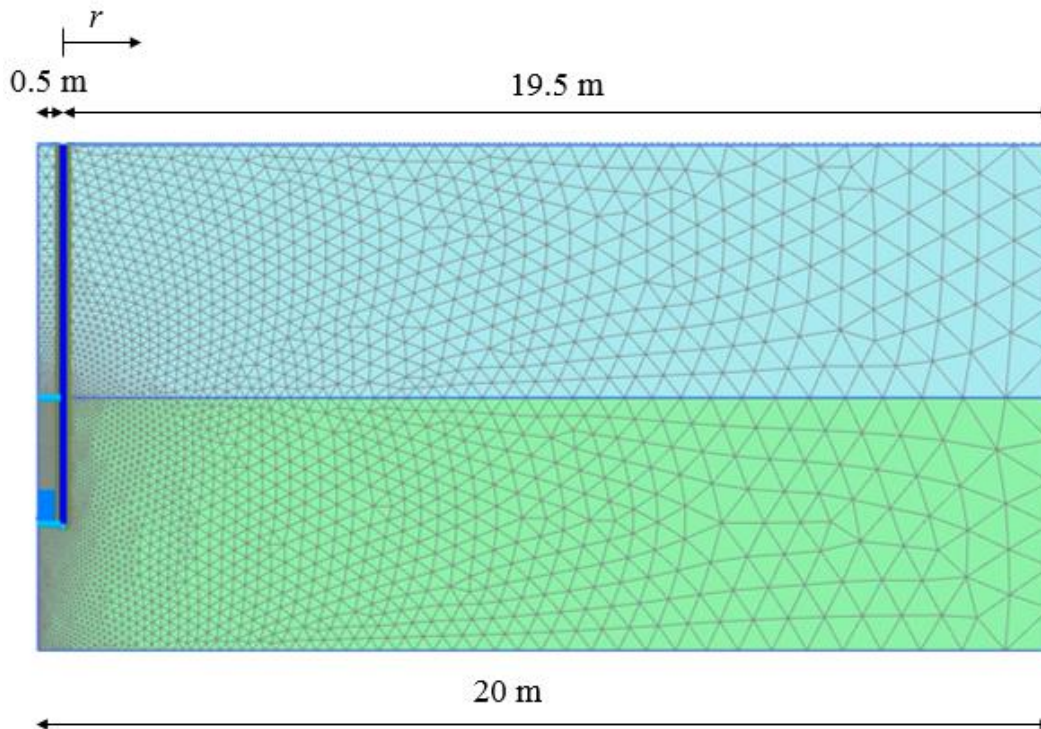
Figure 4-3: Model explaining the r/R relationship

4.1.1 Time comparison

The first analysis of interest is the influence of the duration of the suction. Depending on the permeability of the moraine layer, the pore pressure distribution will be different depending on the time duration of the analysis. The reason for checking the time influence is that the possible suction during drilling will be temporary, such that a steady state solution probably will give a wrong understanding of what really happens. To evaluate the influence of the duration, a permeability of 1 m/day is applied, which resembles a relatively fine sand. For this analysis, the default model with $R = 20$ m is used. Figure 4-4 shows the groundwater head distribution with the relative radial distance from the pile wall, dependent on the time duration of the analysis. One can observe that the 2 hours coupled flow-deformation analysis resembles the steady state solution, which means that this is nearing the time for the solution to converge to the steady state solution.
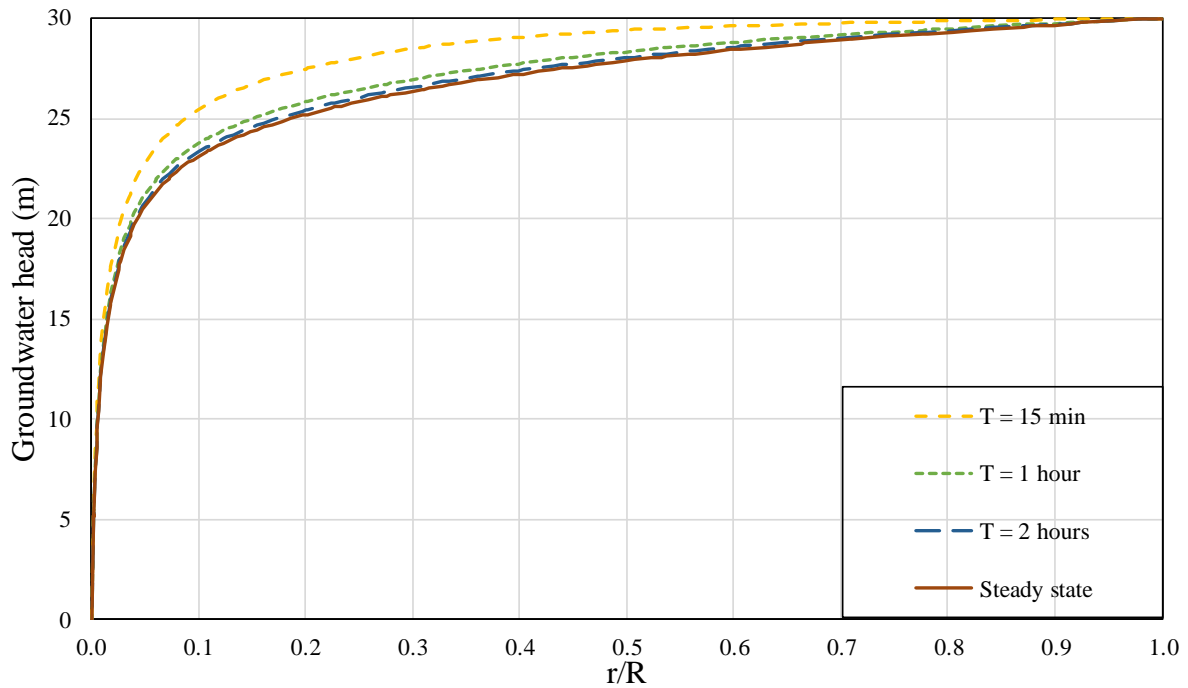
Figure 4-4: Time comparison of the groundwater head distribution

Two important notes here are 1) this is dependent on the permeability (high permeability yields a fast convergence to steady state), and 2) this is also highly dependent on the boundary condition. When resembling the in-situ cases, the left and bottom boundary is supposed to be closed, because the model is axis-symmetric and there is an assumed impermeable bedrock at the bottom. What the boundary condition should be at the right boundary and how large the influence area *R* should be is more difficult to decide. This will be further discussed in section 4.1.4.

4.1.2 Potential comparison

The suction is modelled as a drain where a potential difference is the driving mechanism for the groundwater flow. How representative this is considering the real-life situation will be discussed. To get a significant reaction the smallest potential difference will be 37.5 m, and this will be compared against two larger potentials. The analysis is done with fully coupled flow-deformation and run for approximately 15 min (0.01 days). The resulting pore pressure change is plotted for two points, one point L at the coordinates (1.01, -7.50) and one point K with coordinates (4.01, -7.50). This gives a radial distance from the pile wall of approximately 0.5 m for point L and 3.5 m for point K.

Figure 4-5 and Figure 4-6 shows the pore pressure development for point L and K, depending on the potential difference, denoted ΔH. The influence is not that big at point K, and thus the difference when changing the potential difference is not that big either. There seems to be a linear trend for the increase in potential difference, when comparing the difference between 57.5 m and 47.5 m to the difference between 47.5 m and 37.5 m. The trend is evident in both points. This is reasonable considering the proportionality of the potential *H* in eq. (2) presented in section 2.2.1, and gives a good indication that PLAXIS calculates the flow as expected.
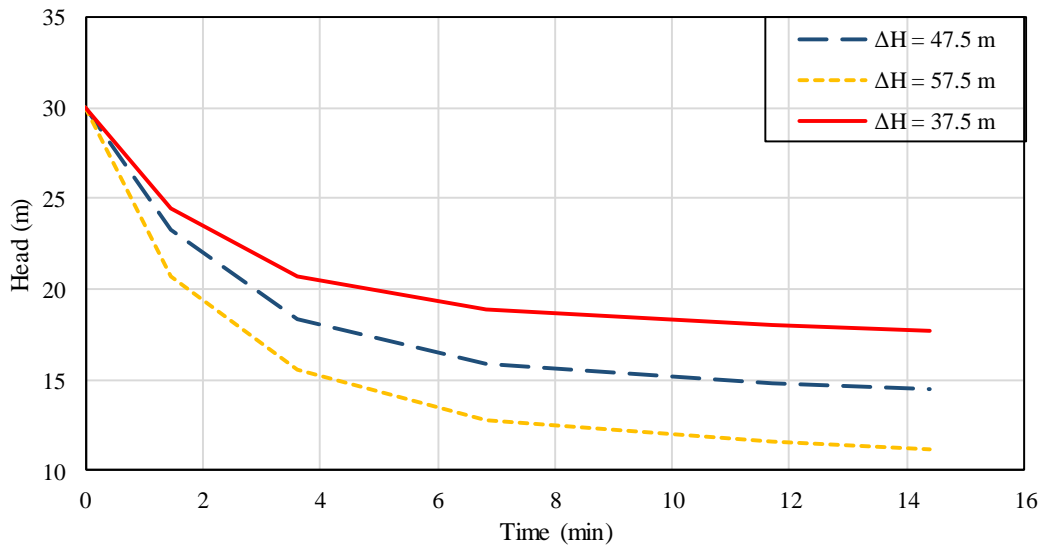
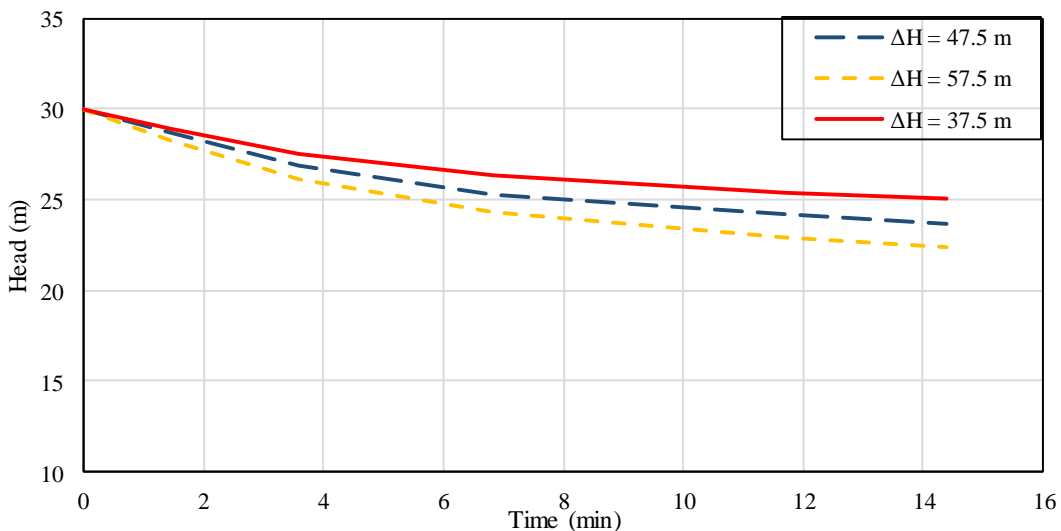*Figure 4-5: Pore pressure development for point L (1.01, -7.50)*

Figure 4-6: Pore pressure development for point K (4.01, -7.50)

4.1.3 Permeability comparison

For the permeability comparison four arbitrary isotropic permeabilities are chosen. The different permeabilities are thought to be representative for varying coarseness of sand/silty sand. Figure 4-7 - Figure 4-9 displays the groundwater head distribution for different time durations. This is done with the other parameters being default as mentioned in the introduction to this section. The values of *k* are in m/day.
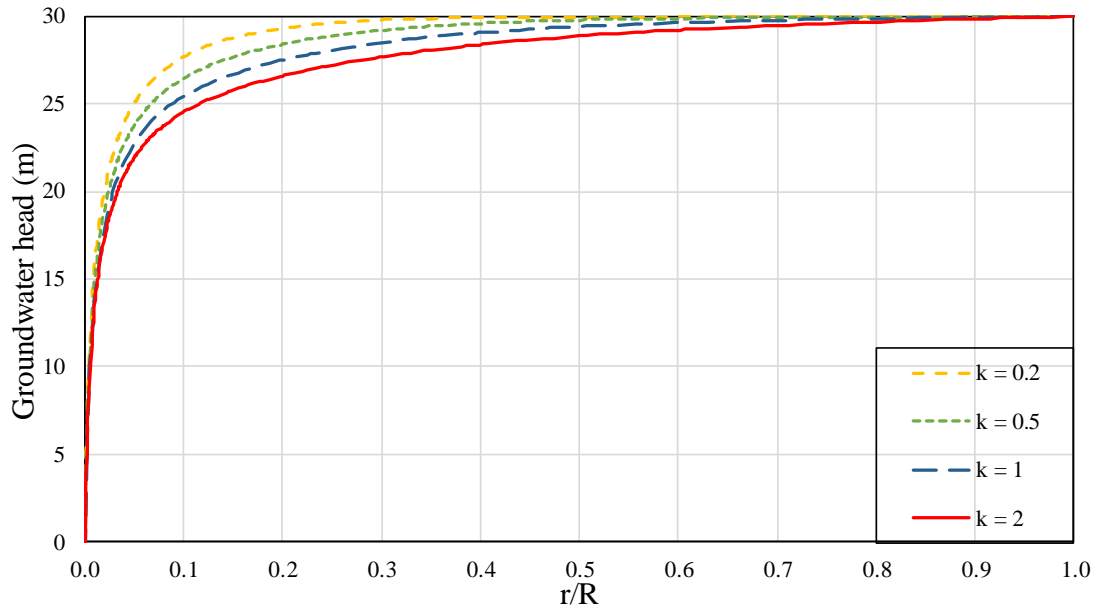

Figure 4-7: Groundwater head distribution for 15 min analysis
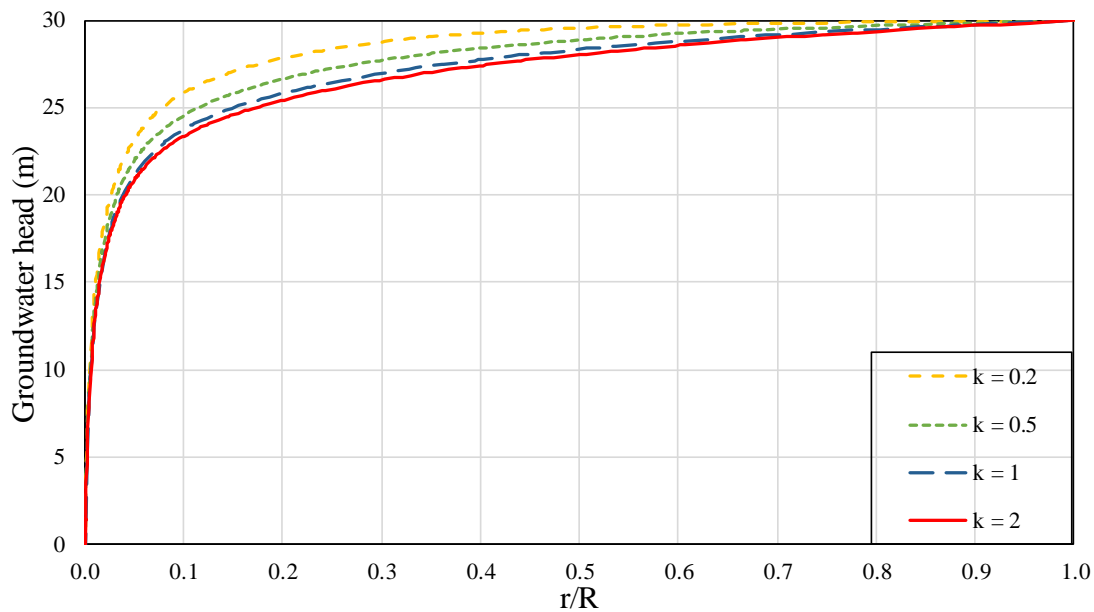

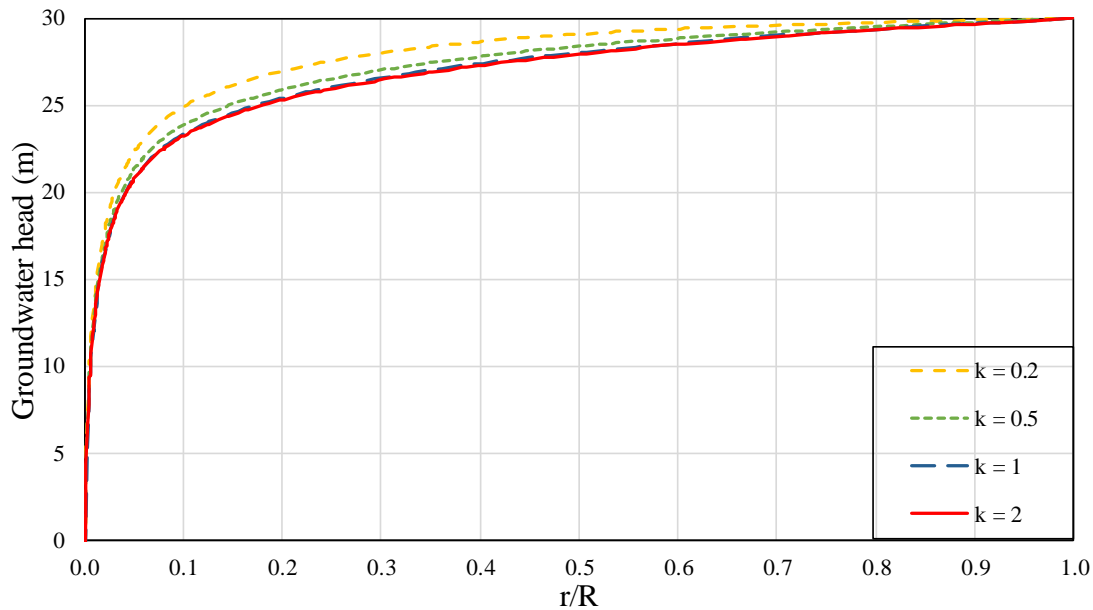*Figure 4-8: Groundwater head distribution for 1-hour analysis*

Figure 4-9: Groundwater head distribution for 2-hour analysis

As previously mentioned, a small permeability will give a slow convergence towards a steady state solution. That is why the comparisons are plotted for different time durations. At two hours, one can see that $k = 1$ and $k = 2$ are overlapping which indicates that they are nearing the steady state solution. For the 15-min analysis, there is a noticeable difference in pore pressures in radial direction. However, as eq. (2) implies, the theoretical groundwater flow is proportional to the permeability, so the resulting pore pressure distribution cannot be evaluated without regarding the resulting groundwater flow as well. This will be addressed in section 4.1.5, and the results from these analyses will be relevant for the duration of the possible Venturi effect.

4.1.4 Boundary comparison

As mentioned in section 4.1.1, the time analysis is greatly dependent on the boundary, both how great the range/influence area is (the magnitude of R) and whether the boundary is closed or open (with a steady pore pressure head). A constant permeability of $k = 1$ m/day is chosen unless else is specified. The first analysis to be done is to investigate the influence of the range R. Figure 4-10 and Figure 4-11 displays the groundwater head distribution with the relative distance r/R to comprehend how the shape of the distribution is for the different values of R, for 15 minutes' analysis and the steady state solution respectively.
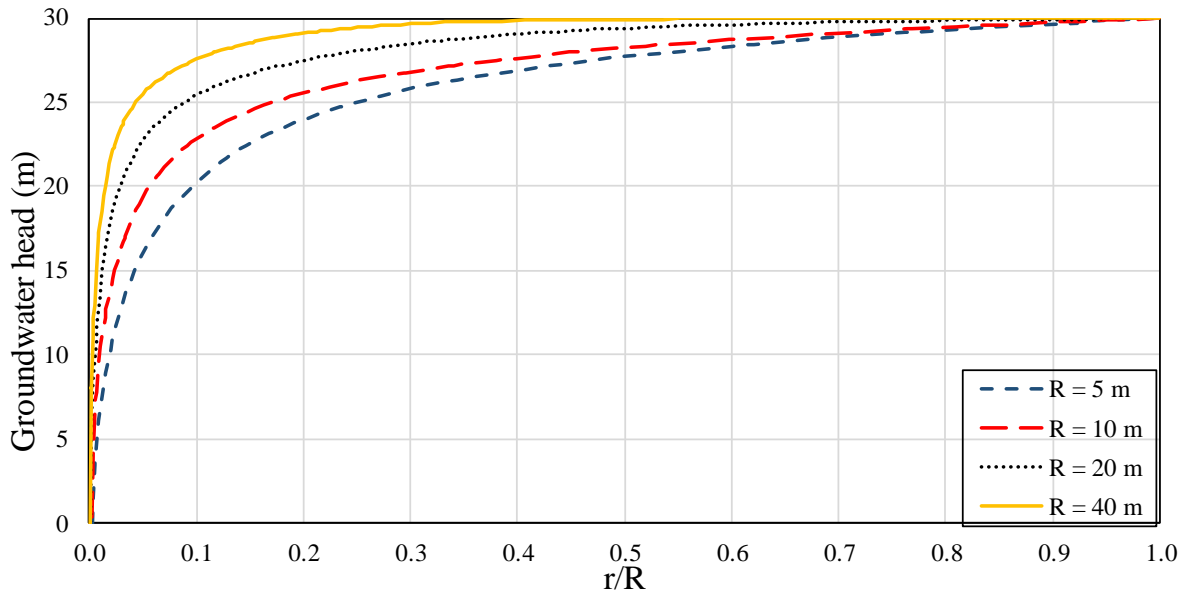
Figure 4-10: Fully coupled flow-deformation with 15 minutes' duration for different ranges R
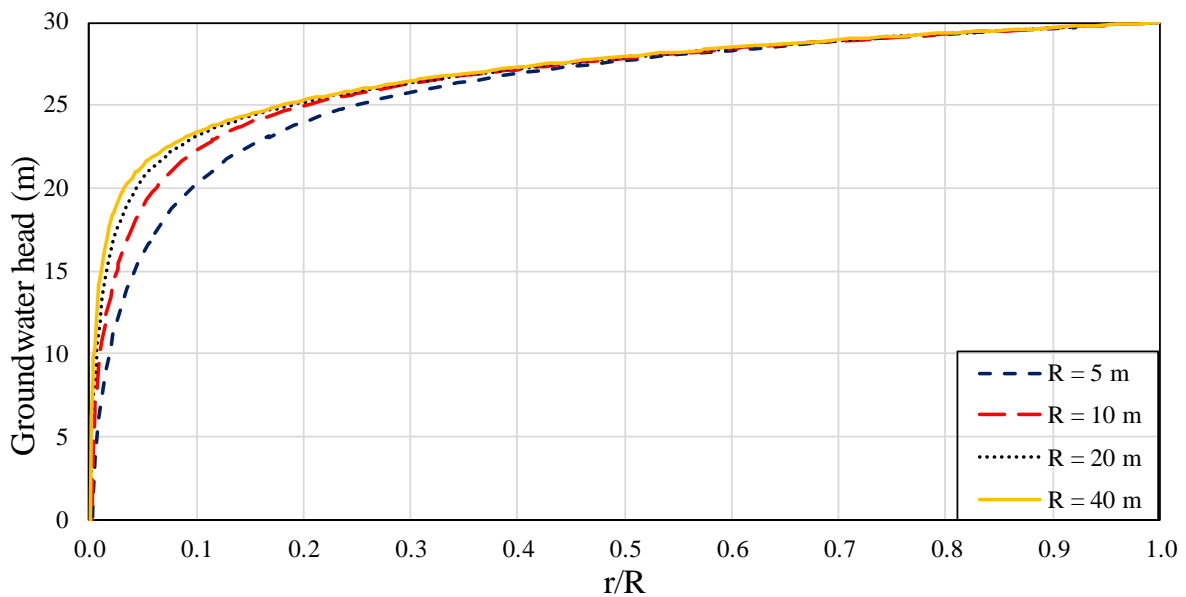

Figure 4-11: Steady state solution for different ranges R

An interesting observation here is that the steady state solution does not give the same shape for the different values of *R*. This is probably because the model is axis symmetric, and this makes the groundwater head - distance relation more complex than for a plane strain model. There could also be some inconsistency in the way of calculation depending on the distance. For the coupled flow-deformation analysis with a duration of 15 minutes, the differences are larger. This is as expected, with the larger values of *R* needing more time to dissipate. Regardless of how obvious the results are, it points to the importance of recognizing the boundary ranges.

When varying the range *R,* it is perhaps more relevant to see the distribution with the actual distance from the pile. Figure 4-12 displays this for a steady state solution. The effect of the varying range *R* is now more evident, with the lower values of *R* giving much steeper distributions. It is also possible to see a linear trend in the increase/decrease of the groundwater head. The difference between the different ranges are approximately the same, which is evident if measuring the values at for instance 2- and 4-meter distance from the pile. The increase in the value of *R* however is a doubling, which probably relates to the fact that the cross section which the groundwater flows through is proportional to 2*R*.
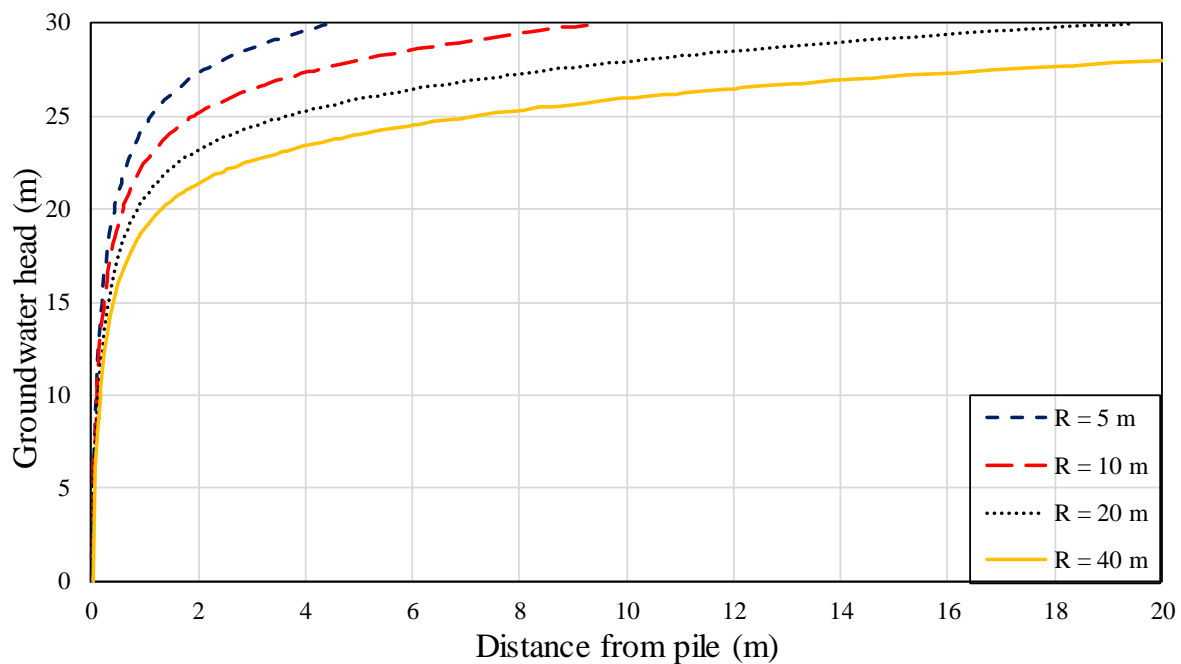

Figure 4-12: Groundwater head distribution against distance from pile for steady state solution with different values of R

Interpreting these results alone, one can see that the distance *R* has great influence on the pore pressure distribution. At a distance 4 m in Figure 4-12, the difference between *R* = 5 m and *R* = 40 m is approximately 7 m groundwater head, meaning a pore pressure difference of 70 kPa. Varying the *R* is the same as deciding where the pore pressure distribution with debt is fixed. This lead to the next boundary study: what happens when manipulating the quantity of water available.

It is obvious that running an analysis of indefinite duration with a closed boundary will just drain the whole model. Therefore, a time dependent analysis must be conducted. Here both a duration of 15 min and one hour will be tested, and the results are displayed in Figure 4-13 - Figure 4-15 with different values of *R*, to see these effects coupled.
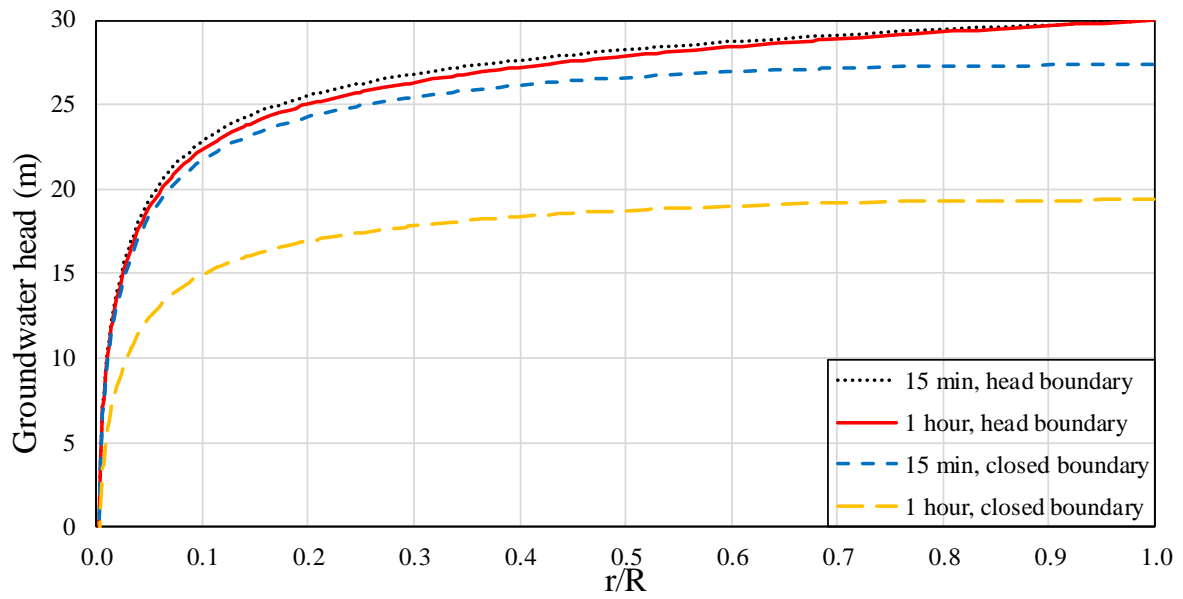
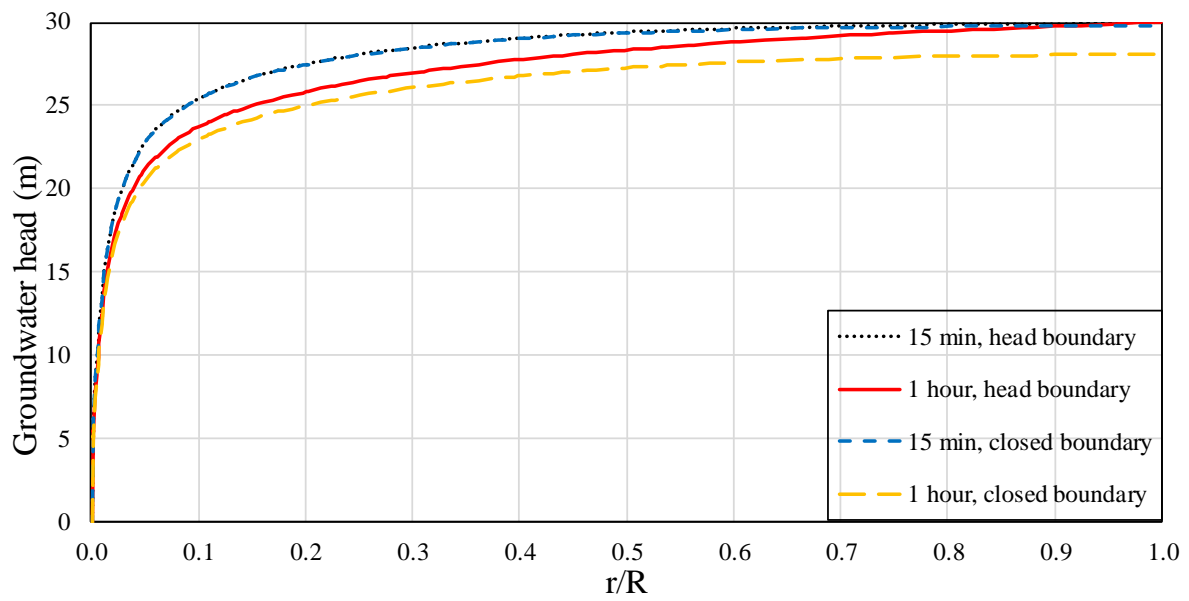*Figure 4-13: Groundwater head distribution for R = 10 m*

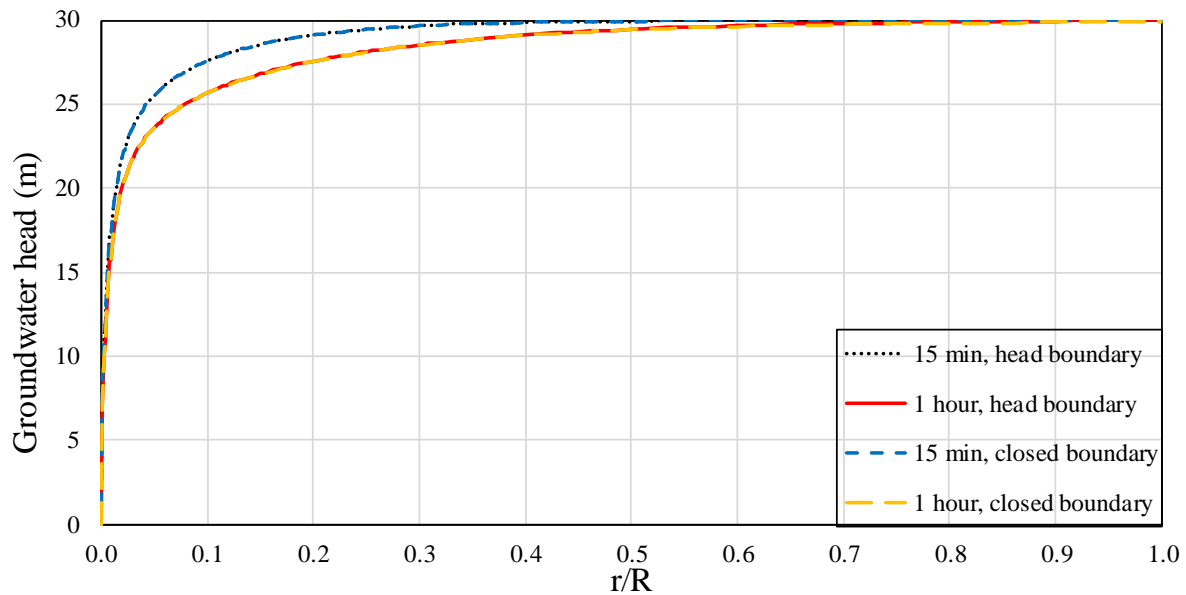*Figure 4-14: Groundwater head distribution for R = 20 m*

Figure 4-15: Groundwater head distribution for R = 40 m

Figure 4-13 - Figure 4-15 shows the importance of the range/influence area when seen in relation with boundary conditions and time duration. A small value of R will be more dependent on whether there is unlimited inflow of water (head boundary), no extra water available (closed boundary), or somewhere in between these two by roughly interpolating them. This is evident when comparing the results from $R = 10$ m to the results from $R = 20$ and $R = 40$ m. For $R = 40$ m, there is no difference between the closed and open boundary, which is probably due to a combination of the time not being sufficient and the permeability not being higher. For $R = 10$ however, there is a significant difference between the two boundary alternatives. There is also a smaller difference between the 15-min and one-hour with head boundary than there is between the head and closed for 15-min analysis. This indicates that for small values of R, the boundary condition is more important than the time duration of the analysis.

4.1.5 Groundwater flow evaluation

The different parameter analyses are all interesting to evaluate, but when comparing to a real-life situation the quantity of groundwater flow flushing out through the pile casing must be considered. This is hard to quantify, but a large excess flow would be noticed visually. The question is how large this excess flow could be before realizing that extra masses are being flushed out. Hence, the key to relating the parametric analyses to real-life situations is to evaluate the groundwater flow. Table 4-2 and Table 4-3 summarizes the values for the groundwater flow when varying the parameters which are most influential.

Table 4-2: Groundwater flow for a potential difference of 37.5 m

k (m/day)	Time (days)	q (m^3/rad/day)	q (l/min)
	ΔH = 37.5 m		
0.2	0.01	1.336	5.8294
	0.04	2.523	11.00866
	0.08	2.829	12.34384
	Steady state	3.064	13.36922
0.5	0.01	5.542	24.18154
	0.04	7.272	31.73009
	0.08	7.607	33.1918
	Steady state	7.67	33.46669
1	0.01	13.32	58.11946
	0.04	15.24	66.49704
	0.08	15.57	67.93694
	Steady state	15.32	66.84611
2	0.01	29.21	127.4527
	0.04	31.17	136.0048
	0.08	31.44	137.1829
	Steady state	30.66	133.7795

Table 4-3: Groundwater flow for k = 1 m/day

ΔH (m)	Time (days)	q (m^3/rad/day)	q (l/min)
	k = 1 m/day		
17.5	0.01	6.381	27.84236
	0.04	7.203	31.42902
	0.08	7.324	31.95698
	Steady state	7.389	32.24059
27.5	0.01	9.768	42.62094
	0.04	11.18	48.78195
	0.08	11.42	49.82915
	Steady state	11.24	49.04375
37.5	0.01	13.32	58.11946
	0.04	15.24	66.49704
	0.08	15.57	67.93694
	Steady state	15.32	66.84611
47.5	0.01	16.87	73.60926
	0.04	19.31	84.25577
	0.08	19.72	86.04473
	Steady state	19.41	84.6921

It can seem like the change in groundwater flow is proportional to both the change in permeability k and the change in potential difference ΔH, which is reasonable considering eq. (2) presented in section 2.2.1. Also observe that the flow changes depending on the duration of

the analysis. A transient flow of limited duration will probably not have established the same degree of flow, thus making the flow velocity and flow quantity smaller than the steady-state solution.

Knowing the groundwater flow in l/min is helpful to make some qualitative assumptions regarding the on-location drilling. One can assume an extra percentage of water flushing out without witnessing that it is more than pumped in. This will be further discussed when comparing the case with the FE analyses, but an example will be given here. If the water/air compressor connected to the drill rig delivers water at a rate of 350 l/min, then a 10% extra flushing return could be fair to assume. This means that the suction around the drill bit is of such a size that a water flow of 35 l/min can be assumed. Further on, when knowing the speed of drilling in m/min with depth, one can assume how much soil material that will be removed with time. Comparing this to the rate of water flow from the drill rig gives a relationship between the amount of water and amount of soil being flushed out at the same time. With the assumed excess flow due to suction, one can then assume a magnitude of excess soil being flushed out.

This idea is however a bit farfetched, but can perhaps serve as a maximum magnitude of extra soil removed depending on the observed groundwater flow. The force that the water suction creates depends on the velocity, which again depends on the cross-section of the water flow. This force is the driving mechanism for flushing out excess soil, and the question will be how large this force must be to impact the soil particles. Evaluating the force of the water flow on the soil particles can be done through a gradient assessment.

4.1.6 Gradient assessment

This section, together with the previous one on groundwater flow evaluation will serve as the evaluation basis for the potential extra soil volume flushed out. The groundwater flow gradient, denoted i, is the same as the part dh/ds in eq. (2) described in section 2.2.1. For an upward groundwater flow, the critical gradient is approximately 1 (Harr 2012). However, for horizontal flow, there does not exist a specified critical gradient. An assessment of what will be critical is then based on assumptions. When plotting the groundwater head distribution with the distance from the pile wall/model center, there is evidently very large gradients in both vertical and horizontal direction close to the center. Figure 4-16 displays the groundwater head distribution in horizontal direction with three different tangent lines which represents different gradients.

For this analysis, a permeability of 1 m/day and a time interval of 15 minutes has been applied, as well as the default case with $R = 20$ m and $\varDelta H = 37.5$ m. Figure 4-17 displays the groundwater head distribution in vertical direction, and Table 4-4 gives the gradient for each tangent line.
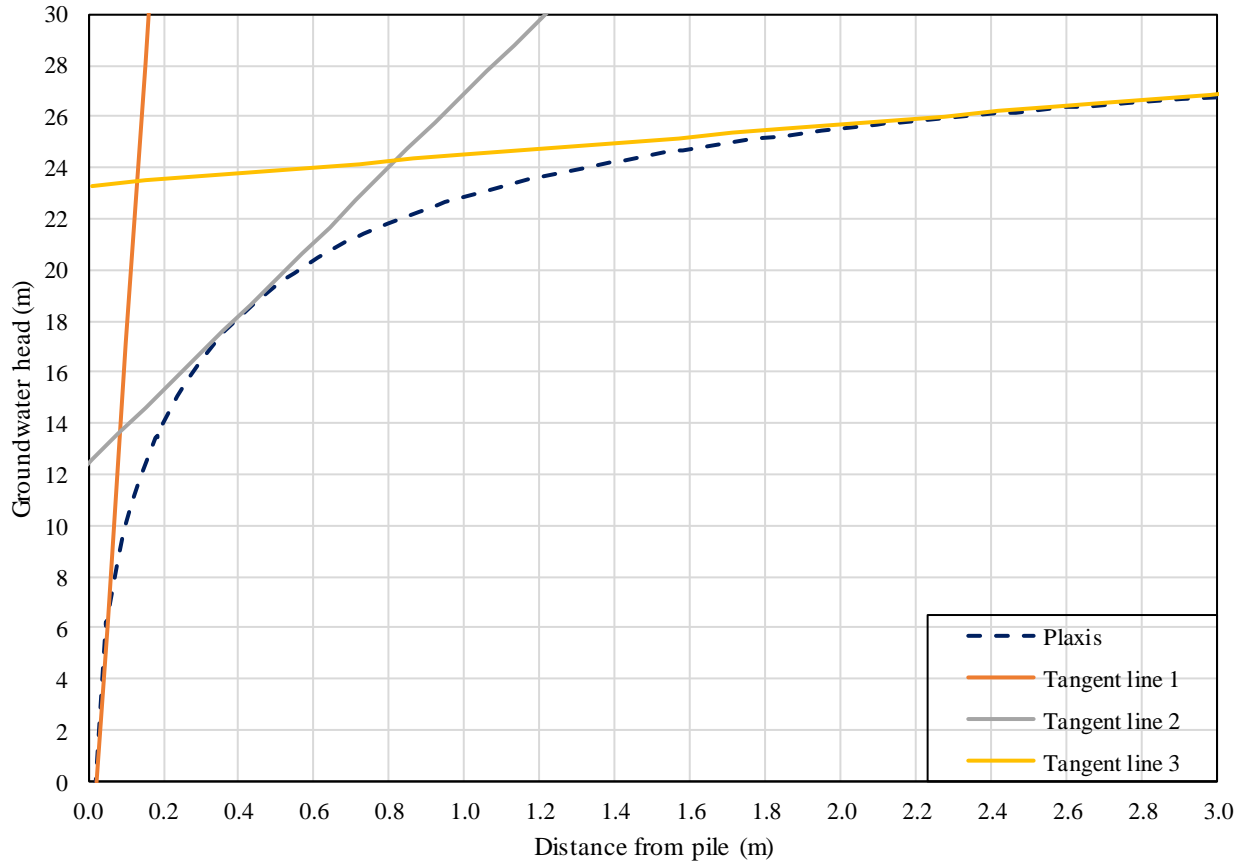

Figure 4-16: Groundwater head distribution in horizontal direction together with gradient lines
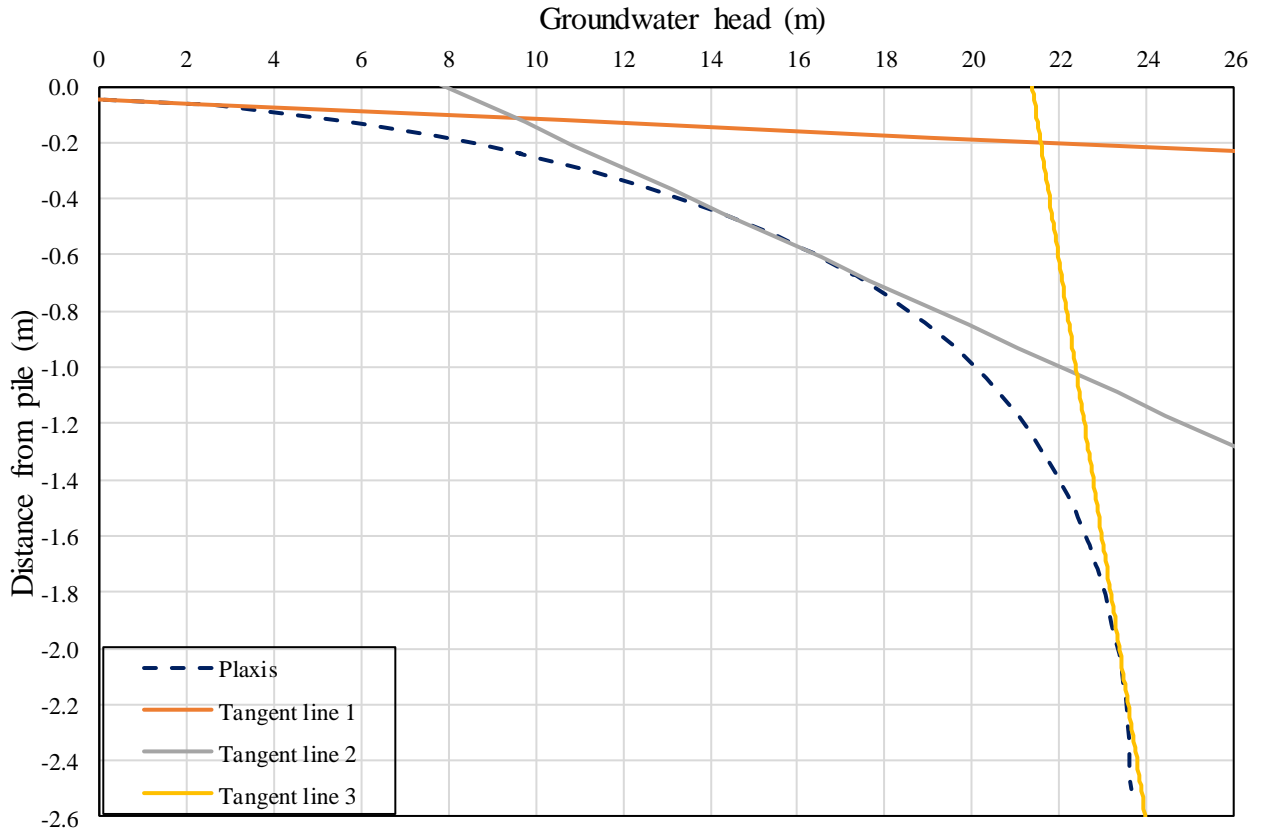
Groundwater head (m)

Figure 4-17: Groundwater head distribution in vertical direction together with gradient lines

Table 4-4: Gradients for the tangent lines corresponding to the groundwater head distributions

	Tangent line	Gradient i
Horizontal direction	1	218.6
	2	14.5
	3	1.2
Vertical direction	1	140.5
	2	14.1
	3	1

There are evidently very large gradients near the pile casing, which probably will yield some sort of fracturing in this area. Theoretically for a vertical flow, a gradient $i > 1$ will give hydraulic fracturing. In this model, the vertical gradient just below the created drain is a lot larger than this for some small distances. The same is observed for the horizontal direction, and the distance from the casing is almost as much as 3 m before the gradient is equal to 1. Seeing how important effect the water pressure have on the effective stresses in the soil, a large flow gradient will most likely exercise a fracturing force on the soil particles. The question however, is how to relate the gradient to an entrainment force.

4.1.7 Discussion

To simplify what is thought to happen with a Venturi effect during drilling, it is now listed in following order:

1. The drilling process reaches a permeable layer with silty/sandy grains.
2. As these grains can be easier removed with flushing fluids and are larger than clay grains, the probability of clogging in the drill bit increases.
3. This again may lead to a change in cross-section for the flow, which will create a higher velocity and thus a lower pressure, eventually creating a suction.
4. This suction will then collect groundwater from the permeable soil.
5. If the flow velocity of this water is high enough, this can create an entrainment force which can drag the soil particles along with the water.
6. This will then create settlement due to volume reduction in the permeable layer.

There is evidently a lot of "may" and "if" regarding the Venturi effect in drilling operations, and a lot of fields of science and factors involved to further complicate the effect. The aim here is not to give any definite answers, but to see how the different soil- and analysis parameters will affect the idea of the Venturi effect. Doing this in the FE program PLAXIS is helpful because the software is based on the theory of soil mechanics. Due to modelling limitations in the program, many simplifications have been done, and these will be discussed here.

The first question arises when trying to create a suction. Representing this through a drain based on difference in groundwater head makes the suction only influence the groundwater, and not the soil grains. An assessment on the excess soil being sucked out will therefore be based on assumptions. As observed in section 4.1.6, the large gradient could imply a large entrainment force, meaning that the force from the water flow on the soil grains is large enough to drag particles along the flow direction and thus also removing soil. This could in turn give some sort of fracturing in the surrounding soil and create even larger disturbance.

Discussing the applicability of a drain to create a suction leads to the next question: how large should this suction/potential difference be? In introductory analyses, it was observed that a small potential difference, ΔH, gave little effect on the pore pressure distribution for even small values of r. In this phase, the results were compared to observed pore pressure changes for the Hobøl River bridge and in Gladengveien 10, where the pore pressure reductions were severe for larger distances from the drilling hole. It was thus chosen to apply considerable large ΔH to see effects on larger radial distances r. However, denoting the chosen ΔH as considerable large

is done without any reference to observed in-situ suction. Seeing how the assumed effect itself is uncertain, measurement of a possible suction is not available to the writer and the magnitude is thus unknown. The comparison between the observed PLAXIS results and in-situ cases will be further addressed in section 5.

In section 4.1.5, a groundwater flow evaluation was presented, where the groundwater flow q was related to ΔH and k, which both are decisive for the theoretical flow. A low permeability yields less groundwater flow, even though the driving mechanism ΔH (representing the suction) is of the same size. Picturing a real-life situation, one would think that a suction would affect the soil particles directly, such that the relationship between observed groundwater flow and soil being sucked into the casing is harder to anticipate. It can also be fair to assume that the soil permeability and the soil skeleton's stiffness/strength is correlated, such that the possibility of fracturing is indirectly permeability dependent. Evaluating how large amount of soil volume being flushed out is therefore considered as a too complex operation to make with FEA alone.

The most influential soil property on the pore pressure distribution seems to be the boundary conditions. How large the area/range R is and what kind of groundwater boundary is applied has great influence, as evident in section 4.1.4. This is probably the most interesting observation, as the boundary condition often is the most complex situation interpreting in-situ. This is further complicated when drilling more piles. The issue is discussed when doing comparison to the presented cases.

4.2 Disturbed zone around pile

Several ideas to represent a disturbed zone in PLAXIS has been considered, and the final choice is a model where a volume reduction is applied on an adjacent zone around the pile. The idea of volume reduction to a soil volume is based on the possible strain and reconsolidation of adjacent soil due to pile drilling, as presented in section 2.2.2. This is a secondary effect, because the soil needs to be reconsolidated, and is thus not related to the temporary pore pressure drop and sudden settlements observed in the cases of Hobøl River Bridge and Gladengveien. However, the idea is of interest due to the visual observations of disturbed clay when drilling, and the analytical implementation in FEA can serve as a good discussion foundation.

As the effect is secondary it is interesting to observe what happens in both an undrained and drained model of the clay, which is representative for the short-term and long-term effect in the adjacent zone. The model with the applied mesh is presented in Figure 4-18, where the soil volume next to the pile is always modelled as drained to allow for volume reduction, and the soil next to this is modelled both as drained and undrained. As for the analyses in section 4.1, the Mohr Coulomb model is applied for the clay layer, and since the moraine layer only serves as a boundary layer between the clay and bedrock, the Mohr Coulomb is applied here as well. However, since the analyses are interested in settlement, it could be interesting to choose a more advanced soil model and do parametric analyses on the stiffness and strength as well as the volume reduction. This is done with the Hardening Soil model in PLAXIS, further presented in the strength and stiffness variation section (section 4.2.3 - 4.2.5).
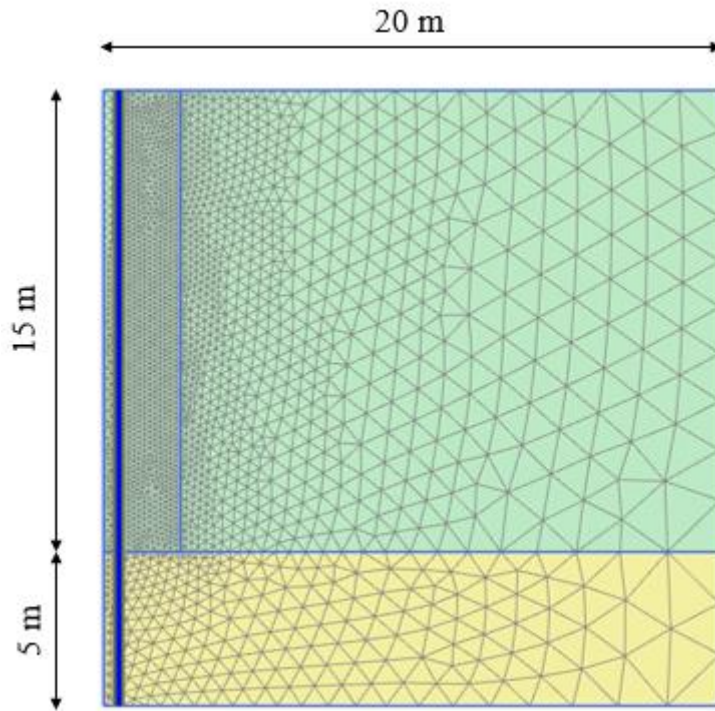
Figure 4-18: The PLAXIS model of volume reduction analyses

4.2.1 Drained analyses

The drained plastic analyses in PLAXIS will give the long-term effects of volume reduction. This will then give the analytical solution for what settlements that is observed when the soil can be deformed. The whole clay layer is thus modelled as drained. The soil properties are approximately the same as for the model in section 4.1, and are as listed in Table 4-5.

Table 4-5: Soil parameters for the drained analyses

Material	Clay	Moraine
Model	Mohr Coulomb	Mohr Coulomb
Drainage type	Drained	Drained
γ (kN/m^3)	20	20
E' (MPa)	3	30
ν	0.33	0.2
c' (kPa)	10	1
ϕ (rad)	25	40

Through this parameter analyses the size of the soil cluster and the degree of volume strain will be varied, and two variants of strain with depth will be tested: both one with two zones of

different volume strain, and one model with a constant volume strain with the entire depth. The different cluster sizes and volume strains applied are presented in Table 4-6.

Table 4-6: The different parameters used in Plaxis analyses

Cluster size (m)	εV, two zones (%)		εV, one zone (%)
0.5	-1.0	-1.5	-1.25
1.0	-2.0	-3.0	-2.5
1.5	-4.0	-6.0	-5.0
2.0	-6.0	-9.0	-7.5
3.0			
4.0			

The different strain rates are applied for all the different cluster sizes. For the model with two clusters, each cluster is 7.5 m deep. Some of the results which highlights the different trends and observations are included in the following.

Figure 4-19 - Figure 4-21 displays the results for three different cluster sizes; 0.5 m, 2.0 m, and 4.0 m. The model is created with fixed boundaries in the right side, meaning that it can't be moved in either vertical or horizontal direction. When the model is 20 m in the x-direction, this may be enough to distort the settlements.
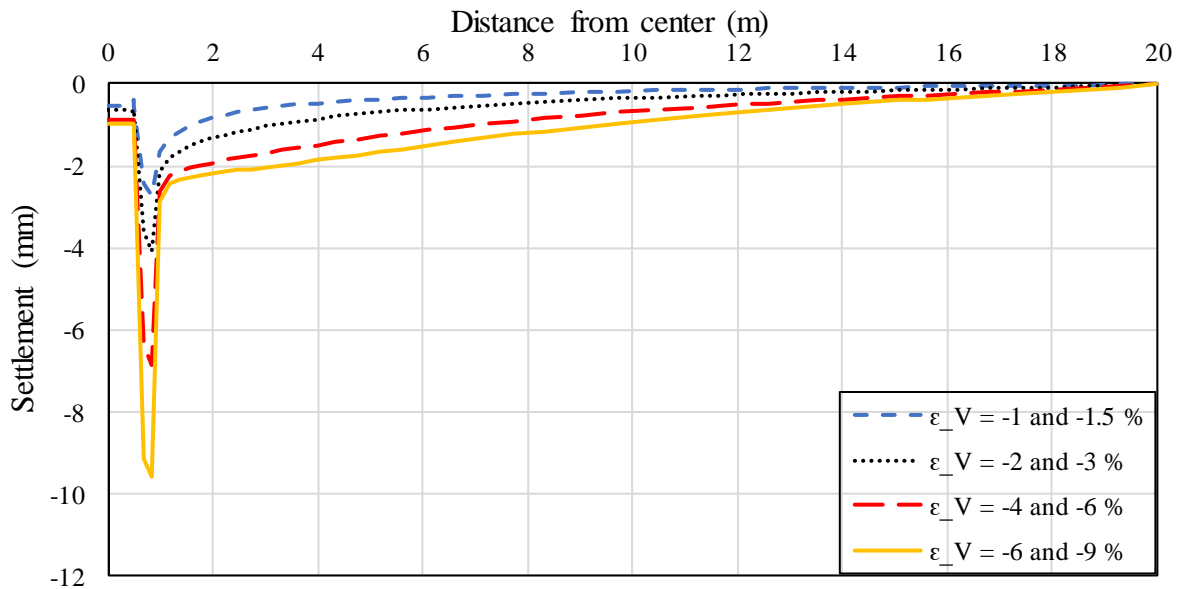

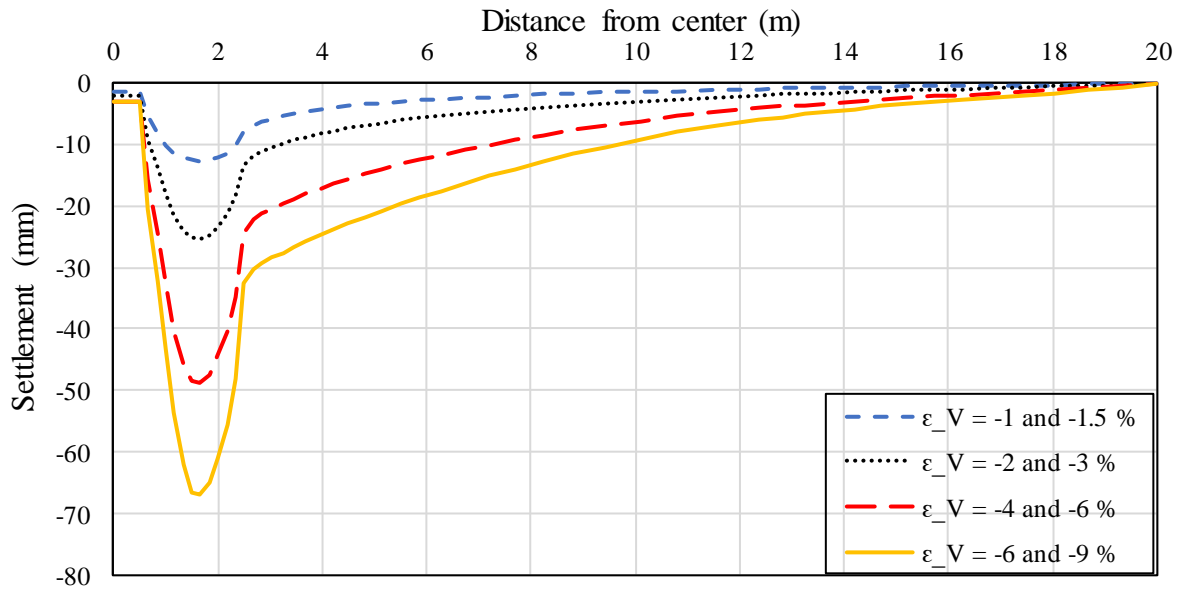*Figure 4-19: Settlements for a two-zone model with cluster size 0.5 m*

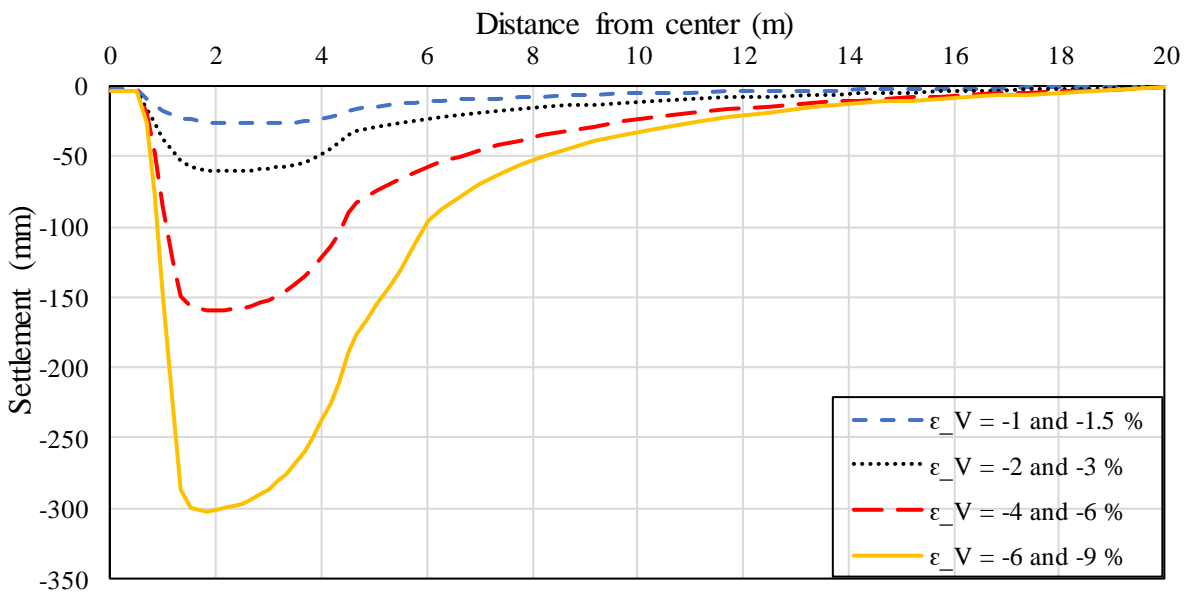*Figure 4-20: Settlements for a two-zone model with cluster size 2.0 m*

Figure 4-21: Settlements for a two-zone model with cluster size 4.0 m

Due to the small spread of the cluster in Figure 4-19, the shape of the deformation is quite similar, with a very local and small peak just in the applied reduction zone. The shape of the settlement changes more when increasing the cluster size. In Figure 4-21, one can see that the settlement curvature is more concave, which is rather obvious considering the difference in size of the cluster. The resulting deformation pattern is not directly applicable to the in-situ cases, but the interesting thing to see is how large the applied strain must be and the size of soil volume that needs to be affected to get settlements of importance. If the increasing volume strain with depth as described by Borchtchev, Eiksund et al. (2015) is valid, then the comparison between

such an applied volume strain against a constant one with depth is also of interest in the theoretical analyses. Figure 4-22 shows the two different models to see what effect the increasing volume strain with depth has. The strain applied is of such a magnitude that the average strain is the same for both models.
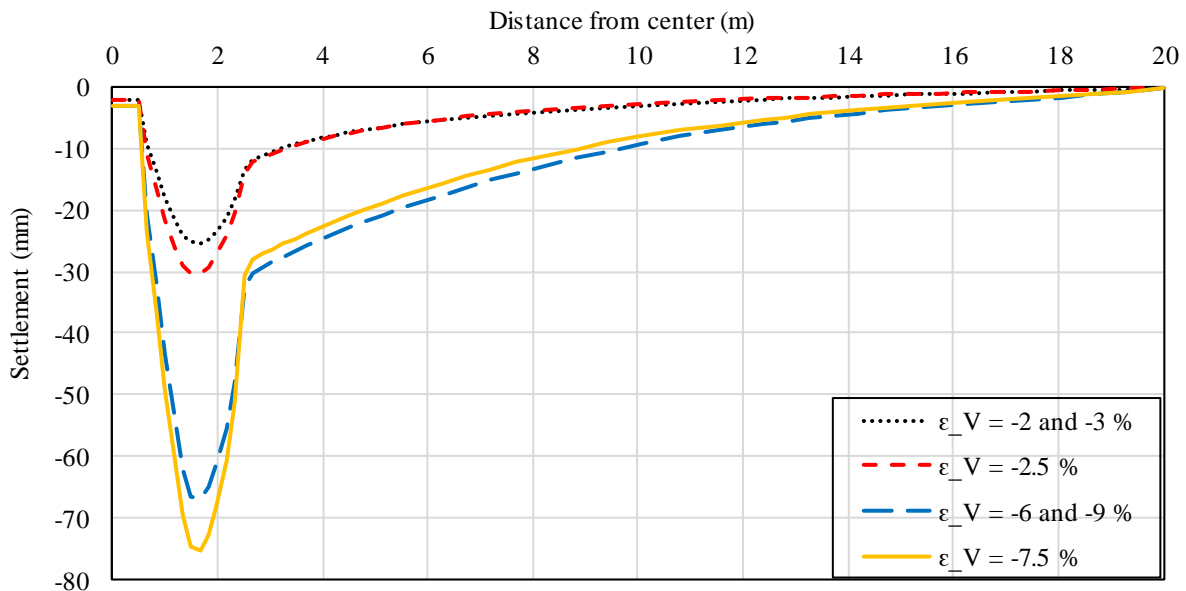

Figure 4-22: Comparison of one-zone and two-zone model for a cluster size of 2.0 m

Figure 4-22 shows that the difference between the two models is not significant, but there is a visual trend that the constant volume strain has a larger maximum deformation in the strained zone. With the large strains and cluster size needed to see this trend it is questionable how much this observation should be considered. What it may prove however, is that the deformation created at large depths have less influence in the top of the soil, but have greater influence on the neighboring settlements.

For the most extreme case, where the soil cluster size is 4.0 m and the average strain is -7.5%, the model in PLAXIS starts to reach its limit. This can be seen with the irregular deformation shape in Figure 4-21. When doing the analysis for a constant strain of -7.5%, the soil body collapses at 86% finished analysis, thus leaving the results without comparison foundation for this extreme case.

When evaluating the settlements created by a volume reduction, an interesting observation is made. It turns out that the size of the volume reduction and the size of the total volume of settlements does not fit. A hand calculation of the most extreme case supports this:

$$\Delta V = \sum A_i \bar{u}_{yi} \qquad\qquad eq.(\,3\,)$$

$$= \sum 2\pi \bar{r}_i \, \Delta r_i \bar{u}_{yi}$$

ΔV = Volume change (m^3)

$\bar{u}_{yi}$ = Average settlement for increment i (m)

$\bar{r}_i$ = Average distance to increment i (m)

Δr_i = The thickness in radial direction for increment i (m)

Figure 4-23 shows how the different sizes in eq. (3) is decided.
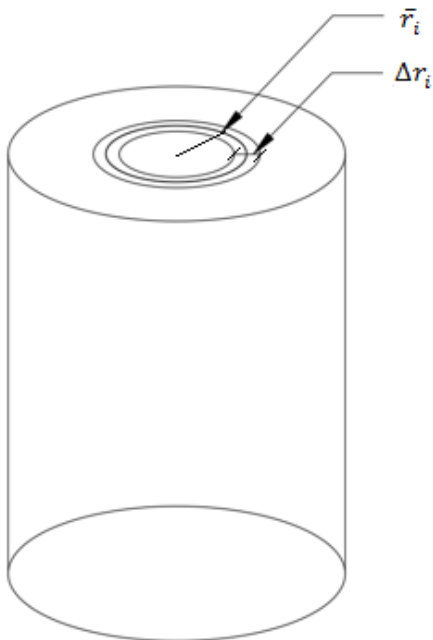
Figure 4-23: Calculation model for the total volume loss in PLAXIS

Summing the settlement increments will then give a total volume loss of 44.4 m^3, calculated in the Excel Spreadsheet with the data from PLAXIS. The volume loss from the volume strain is calculated from:

$$\Delta V = \varepsilon_V V \qquad\qquad eq.(\,4\,)$$

$$= \varepsilon_V L \pi (r_o{}^2 - r_{in}{}^2)$$

ε_v = Volume strain (-)

V = Volume of the soil cluster (m^3)

L = Length/depth of the soil cluster (m)

r_o = Radius of the outer border of the soil cluster (m)

r_{in} = Radius of the inner border of the soil cluster (m)

Doing this calculation for the extreme case of an average volume strain of -7.5% ($\varepsilon_v = 0.075$), and the soil cluster size of 4 m ($r_o = 4.5$ m and $r_{in} = 0.5$ m) gives a volume loss of 70.7 m^3. In other words, the size of the applied volume reduction and the total volume of the measured settlements are far from the same.

The reason for this large difference is how PLAXIS copes with applying strains on a soil volume/area next to an unstrained soil volume/area. The applied strains will yield a resulting force, which is partly taken by the strained soil volume and the neighboring soil volume. The adjacent soil contributes to the strained soils stiffness such that the observed deformation becomes smaller than the applied deformation.

When applying a negative volume strain on a soil volume (V1) connected to another soil volume (V2), V1 will try to pull V2 in the strained direction. This operation will create tension forces, relaxing the previous compressed soil. This is evident in an effective stress contour plot. Figure 4-24 displays the effective stress contour plot for -7.5% volume strain on a 3.0 m large soil volume.
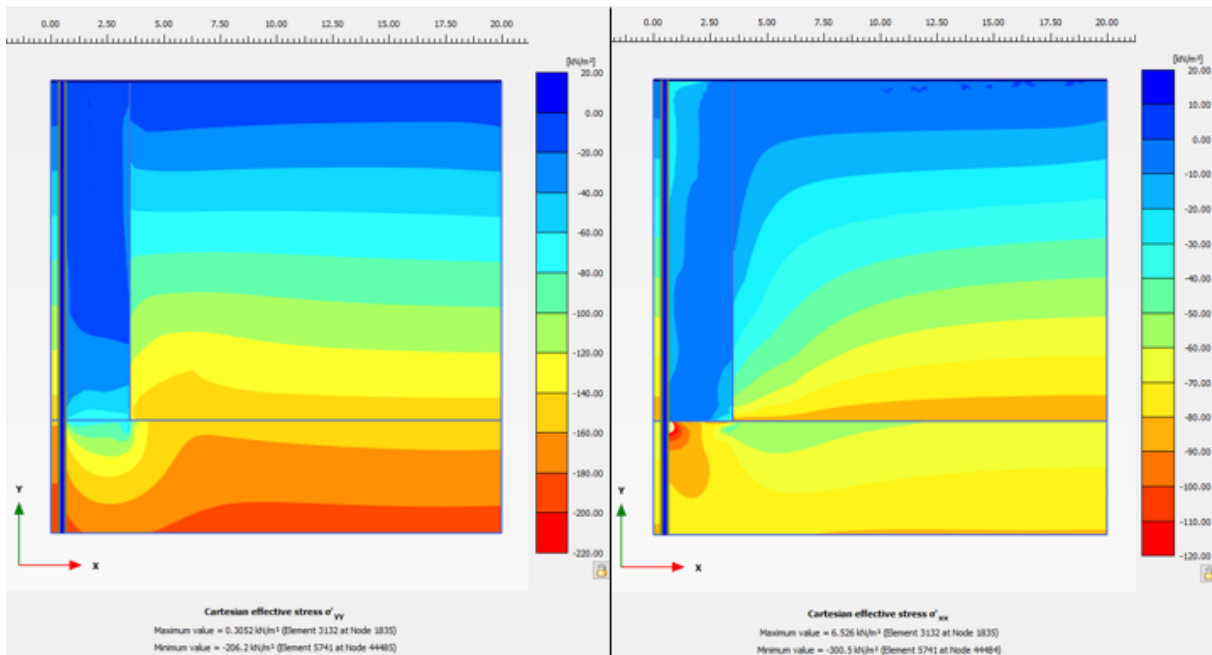

Figure 4-24: Vertical (to the left) and horizontal (to the right) effective stresses for the case with -7.5% volume strain on a 3.0 m soil cluster

The values of the colors are not so easy to see, but the point is that the effective stresses are close to zero (blue color) in the strained soil volume when applying such large strains. From the horizontal effective stress plot, the stresses in the unstrained zone is also clearly affected by this. These observations give reason to believe that the applied strains unload the initial stress state, which is uniform along the radial direction.

4.2.2 Undrained analyses

When doing an undrained analysis, the soil volume is restricted from being changed. That is why the volume strain will be applied on a drained soil volume, while the adjacent soil will be modelled as undrained. The reason for seeing the undrained response in PLAXIS is to grasp the short-term situation (albeit after the reconsolidation of the strained zone). The model is the same as the previous one, but the material for the surrounding clay has an undrained behavior instead of drained. The soil parameters are given in Table 4-7. Note that the shear strength s_u increases with 1 kPa per meter depth (increasing depth is here referred to as positive z-value). This alteration was made due to failure in the initial phase with a low constant value of s_u.

Table 4-7: Soil parameters used for the undrained analyses

Material	Clay	Clay2	Moraine
Model	Mohr Coulomb	Mohr Coulomb	Mohr Coulomb
Drainage type	Drained	Undrained (B)	Drained
γ (kN/m^3)	20	20	20
E' (MPa)	3	3	30
ν	0.33	0.33	0.2
c' (kPa)	10	-	1
su (kPa)	-	30 + z	-
ϕ (rad)	25	-	35

The analyses are done with only the one strained zone this time, with the same level of strain as for the drained analyses, as presented in Table 4-6. The volume strain analyses show approximately the same deformation pattern as for the drained ones. Figure 4-25 displays the settlement observed when doing the undrained analyses (on the right side), compared to the settlement observed when doing the drained analyses (to the left). As one can see, the deformation pattern resembles a great deal, but the magnitude of the settlements is a bit larger in the drained case. Figure 4-26 further substantiates this, with the direct comparison for one of the applied volume strains. The tendency is the same for all the drained-undrained analyses.
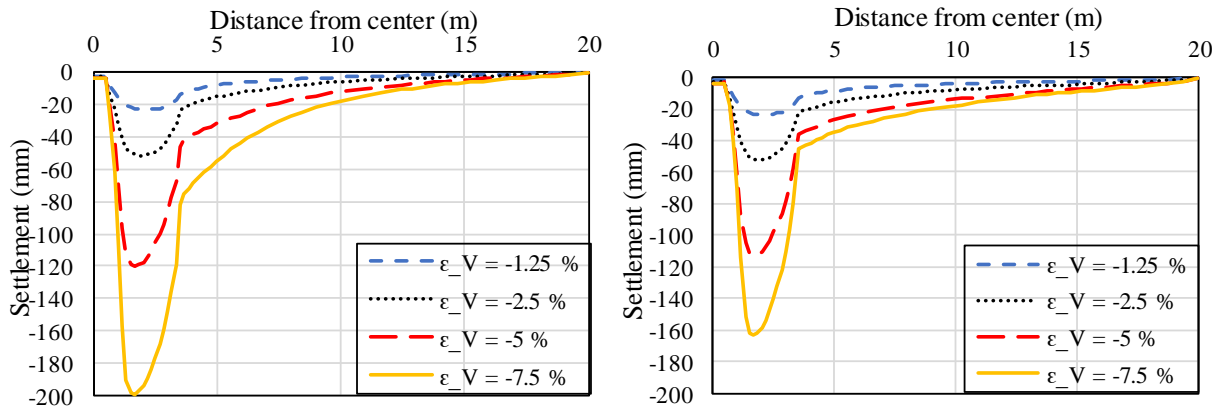
Figure 4-25: Settlements for both drained (to the left) and undrained (to the right) analyses for a cluster size of 3 m.
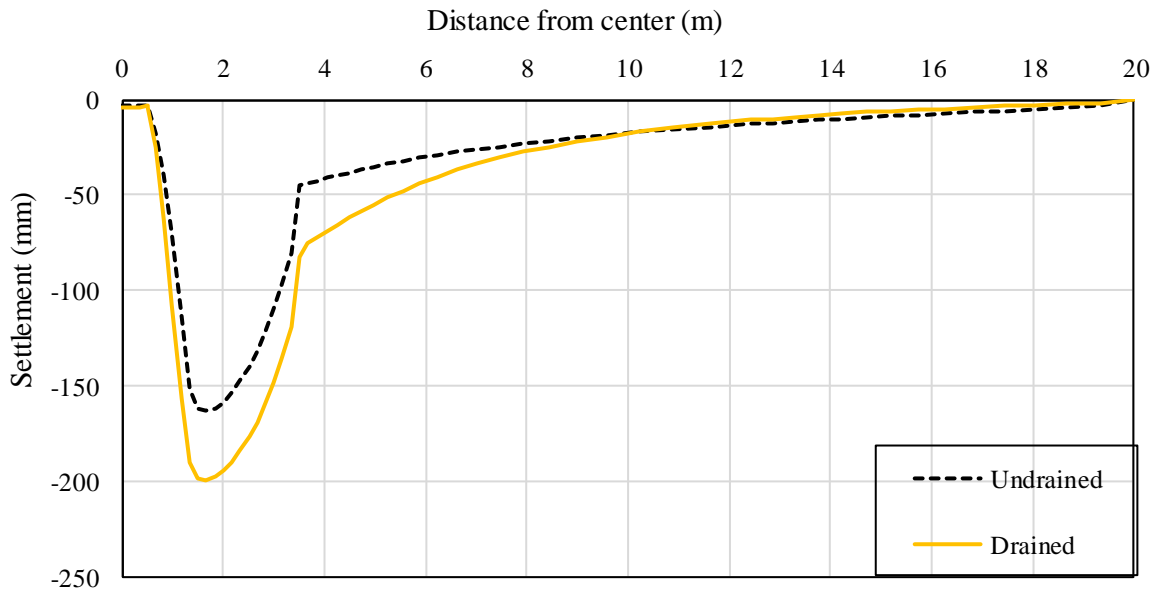

Figure 4-26: Comparison between undrained and drained analysis for the case with a 3 m large cluster and -7.5% volume strain.

Observe that the maximum deformation (which is found in the drained zone) is considerably smaller in the undrained analyses. This implies further that PLAXIS couples the two zones such that the volume reduced zone becomes stiffer when the adjacent zone is stiffer.

4.2.3 Hardening Soil comparison

The Hardening Soil model in PLAXIS is a more advanced soil model which accounts for the plastic behavior of soils, as opposed to the simple Mohr Coulomb that is dominated by the theory of elasticity, and is thus better suited for dealing with soil deformations. The aim here is not to do a comprehensive study on the soil models, but merely to see what the consequence of choosing one or the other is. Further description is therefore referred to the PLAXIS manual (PLAXIS 2017). The parameters used are presented in Table 4-8.

Table 4-8: Parameters chosen for the Hardening Soil model

Material	Clay	Clay2	Moraine
Model	Hardening Soil	Hardening Soil	Mohr Coulomb
Drainage type	Drained	Undrained (B)	Drained
γ (kN/m^3)	20	20	20
E' (MPa)	-	-	30
E_{50} (MPa)	3	3	-
E_{oed} (MPa)	3	3	-
E_{ur} (MPa)	9	9	-
ν_{ur}	0.2	0.2	
ν	0.33	0.33	0.2
c (kPa)	10	-	1
su (kPa)	-	30 + z	-
ϕ (rad)	25	-	35

Figure 4-27 shows the result for the case with a 2.0 m cluster size, when comparing the Hardening Soil to the Mohr Coulomb. The difference in maximum displacement in the strained soil volume is larger than a doubling, which is surprisingly large. Second, the deformation pattern is a lot more curved between the strained zone and the adjacent unstrained zone, yielding larger settlement for distances away from the strained zone. At 5 m distance, the Hardening Soil model gives approximately three times the settlement observed in the Mohr Coulomb model.
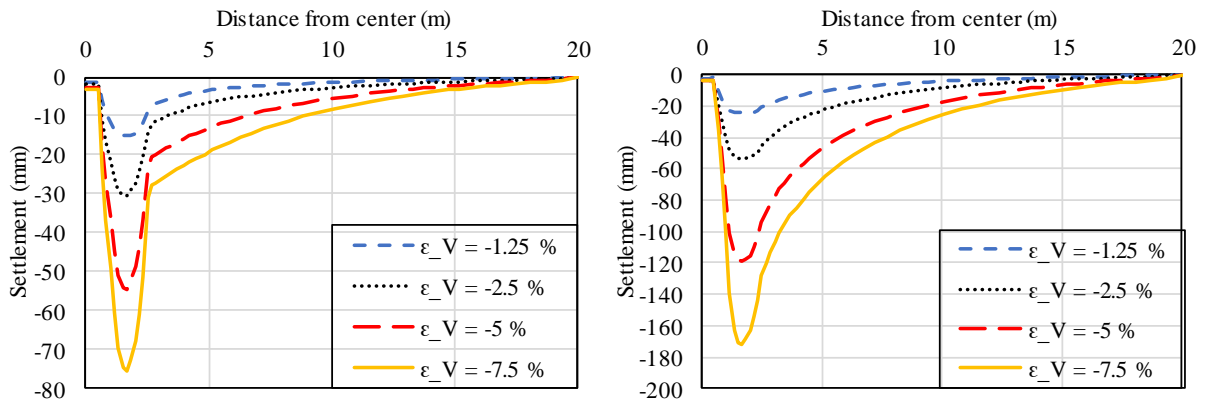
Figure 4-27: Comparison between Mohr Coulomb (to the left) and Hardening Soil model (to the right) in drained situation with a 2.0 m cluster size.

As for the drained – undrained comparison this can be highlighted when plotting the deformation pattern together. Figure 4-28 shows the direct comparison for two different strain levels for the drained situation with a 2.0 m cluster size, where the large difference is quite clear.
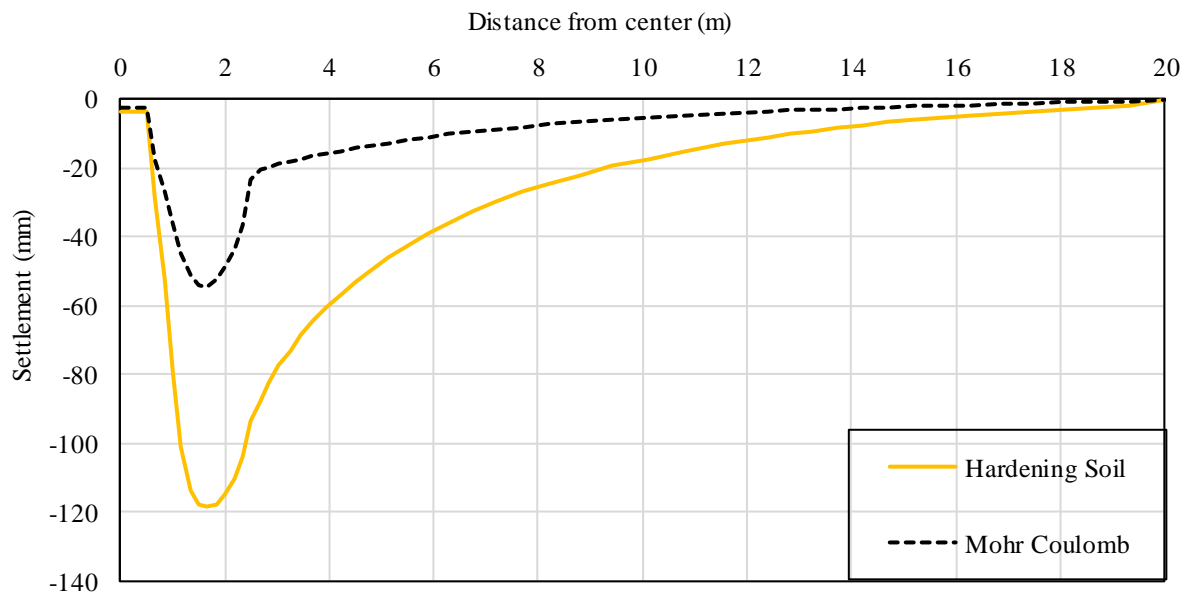

Figure 4-28: Comparison between the Hardening Soil and Mohr Coulomb model for a drained case with -5% volume strain and 2.0 m cluster size

However, this tendency is not as evident when applying the strain to a 3.0 m cluster. Figure 4-29 shows the result from the analyses with a 3.0 m large cluster size, where the difference in settlement is not nearly as large as for 2.0 m. The curved settlement pattern is still evident, and this is an interesting observation when evaluating and discussing the impact of assumed straining and reconsolidation.
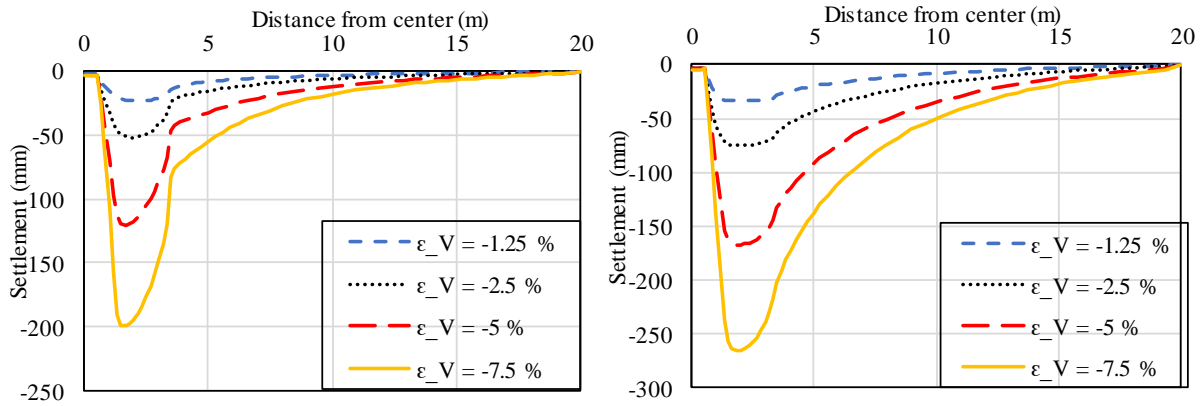
Figure 4-29: Comparison between the Mohr Coulomb (to the left) and the Hardening Soil model (to the right) for a drained case with a 3.0 m cluster size

It is evident that the choice of soil model is of importance when comparing the volume reduction effect, and this will be further discussed in section 4.2.7. Considering these results, the Hardening Soil model will be the basis for the further investigation on strength and stiffness variation.

4.2.4 Strength variation

For both the drained and undrained situations it could be interesting to see the effect of varying the strength of the soil. The strength parameters chosen for the clay layer resembles a loose – medium dense clay, but what if it were to be looser/softer or firmer/stiffer? By varying the cohesion c' and the friction angle φ for the drained soil model and the shear strength s_u for the undrained model, this can be evaluated.

First, the drained case is evaluated, as this is only dependent on c' and φ. The cluster size is chosen to 2.0 m and volume strain of -5% as this is significant enough to observe deformations, but not so large that the model is in danger of collapsing. Figure 4-30 shows the resulting deformations for some arbitrary chosen drained shear strengths. The interest of the thesis is primary for soft soils; therefore, the cohesion and friction angle are not chosen larger as presented in Figure 4-30.
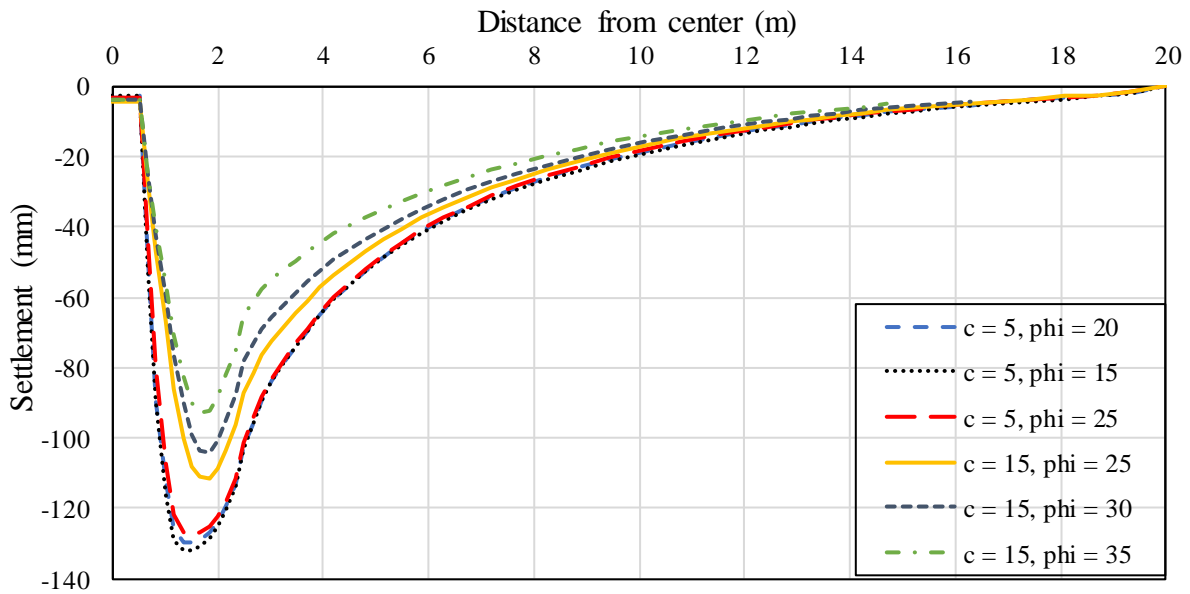
Figure 4-30: Strength variation for a soil cluster of 2.0 m and -5% volume strain for a drained case

The strength influence on the settlements seems to be most dependent on the cohesion. However, considering a $\tau - \sigma'$ plot, the cohesion is dependent on the attraction a and friction $\tan \varphi$ which makes the friction angle φ (denoted phi in Figure 4-30) more influential than credited here.

Undrained strength variation can also be of interest, for the same reason as the undrained soil has been evaluated. This is modelled as in section 4.2.2, with a drained soil volume where the strains are applied, and the rest of the model as undrained with the shear strength s_u as determinative for the strength. Thus, it is this parameter that is changed, while the rest are kept constant. Figure 4-31 shows the resulting settlement when varying the undrained shear strength. Before evaluating the results, two important notes should be made: 1) the undrained shear strength s_u is increasing with 1 kPa per meter depth for the same reason as presented in section 4.2.2, and 2) the analysis for $s_u = 10$ kPa reached maximum capacity already in the initial phase, meaning that the soil is too weak/soft for the analysis. The resulting settlement for $s_u = 10$ kPa should not be considered when discussing the undrained strength influence.
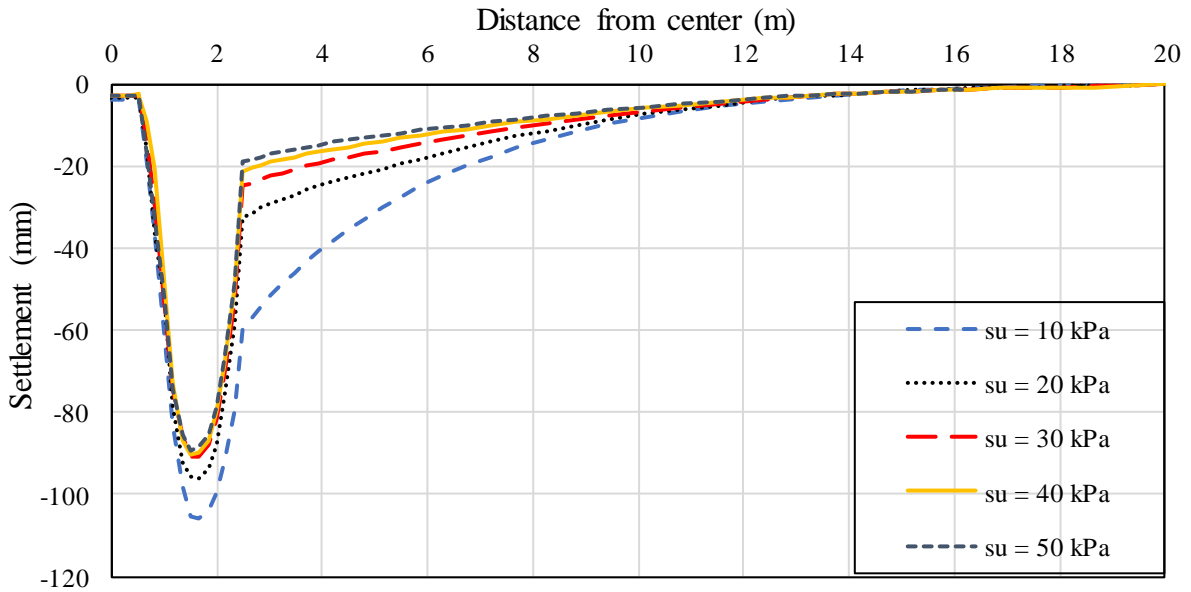
Figure 4-31: Undrained strength variation for a soil cluster of 2.0 m and -5% volume strain

4.2.5 Stiffness variation

A key soil property related to deformation is the stiffness, tested and decided through a stiffness modulus *M* or *E*. In PLAXIS, the stiffness of the soil is decided with different variants of the modulus *E*, as seen when presenting the parameters chosen in the Mohr Coulomb or Hardening Soil model. For these analyses as well, the Hardening Soil has been chosen. As in section 4.2.3, the relationship between the different stiffness parameters is $E_{50}^{ref} = E_{oed}^{ref}$, and $E_{ur}^{ref} = 3E_{50}^{ref}$.

The analyses are chosen to be done drained, at the same case as in section 4.2.4: volume strain of -5% and cluster size of 2.0 m. Figure 4-32 shows the result with the different applied stiffness. The other parameters are as presented in section 4.2.3.
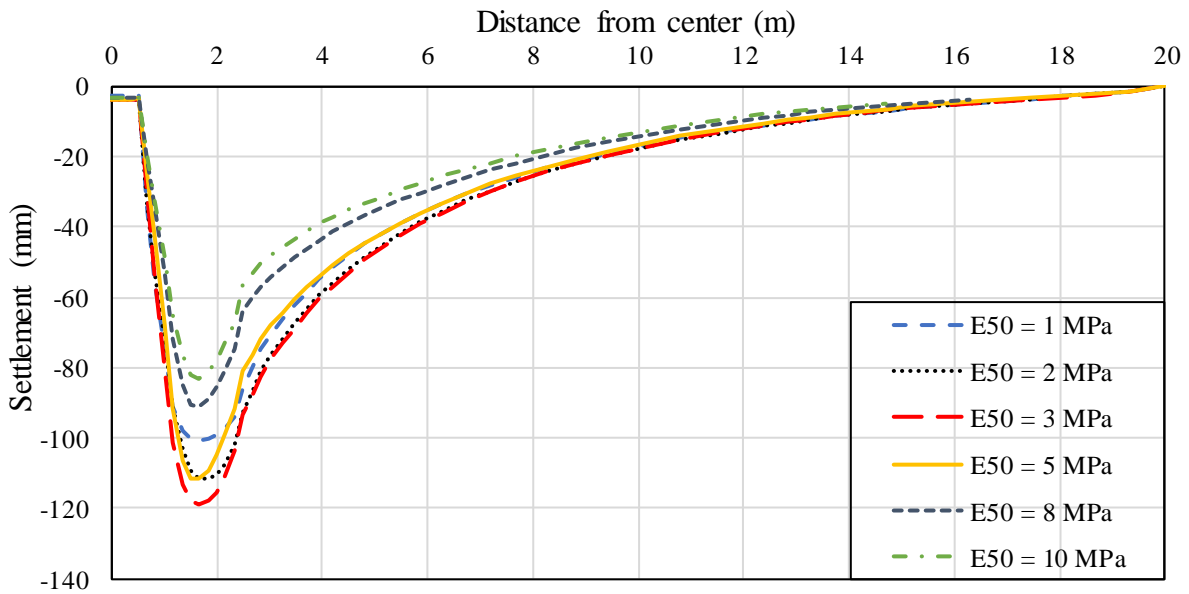
Figure 4-32: Drained stiffness comparison in the Hardening Soil model, -5% volume strain and 2.0 m cluster size

The difference in settlement with the varying stiffness is not surprisingly large, but what it is interesting is that the softest soil does not give the softest response. It is when applying a stiffness of E_{50}^{ref} = 3 MPa that the largest settlements are observed. From lowest to highest stiffness it seems that this is that response becomes softer until this level, and then stiffer when applying larger stiffness than E_{50}^{ref} = 3 MPa again. The reason for this is not easy to answer, and will not tried to be answered here, as it is not what is the main interest in the thesis. The observations done for both the strength and stiffness analyses should however be considered when discussing the relevance to in-situ situation.

4.2.6 Pore pressure observations

When running the PLAXIS analyses in an undrained model, the pore pressure development is of interest. This is because an undrained situation will not let water dissipate, and load actions will then instead affect the water pressure. Evaluating the pore excess pore pressure will help explain the difference in settlements, as well as being interesting to the discussion on pore pressure changes regarding drilling.

When running the first volume reduction analyses, some pore pressure distributions were gathered. The soil model used for the following results is thus the Mohr Coulomb model, with parameters presented in section 4.2.2. Figure 4-33 shows the excess pore pressure distribution

with depth for a case with 0.5 m large soil volume and varying volume strain. The cross-section where the excess pore pressure is registered is right next to the boundary between the drained and undrained soil volume. There are some disturbances to the plot, partly due to the mesh, and in the depth close to 15 m there is some inconsistency between the three neighboring soils (undrained clay, drained clay, and moraine).
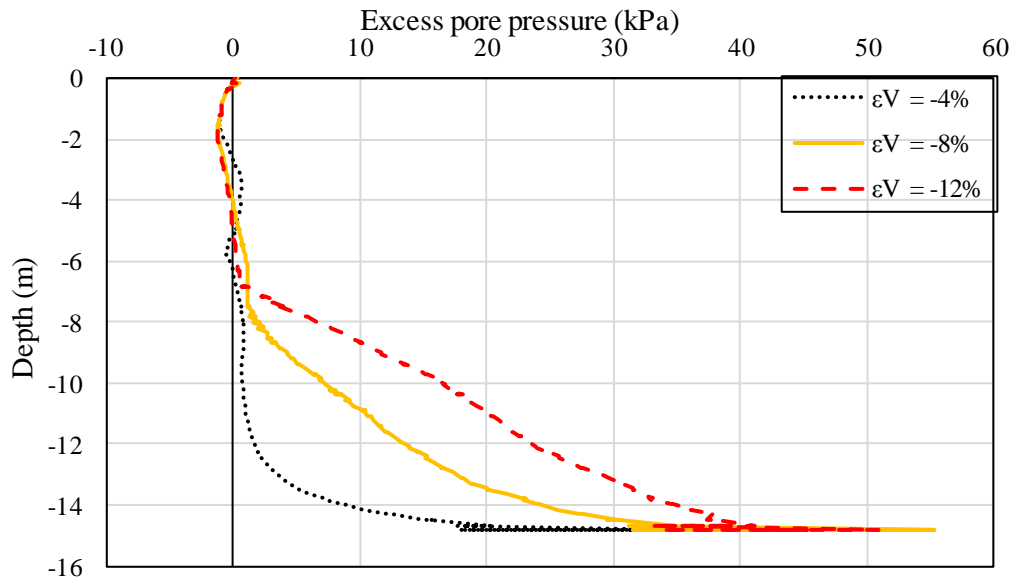
Figure 4-33: Excess pore pressure with depth from undrained analysis, for soil cluster size 0.5 m. Positive values represent suction

As evident, the volume reduction operation yields relatively large suction when reaching the largest depths. This may be related to the observed deformation pattern in the undrained analyses. Figure 4-34 displays a contour plot of the deformations in x-direction. The negative values imply that the deformation is in left direction in the figure. The deformations can be interpreted as trying to expand in the radial direction, which in turn will try to expand the void between soil grains, which will create a pore under pressure (suction).

Figure 4-34: Deformation contours for the deformation in x-direction for an undrained soil

4.2.7 Discussion

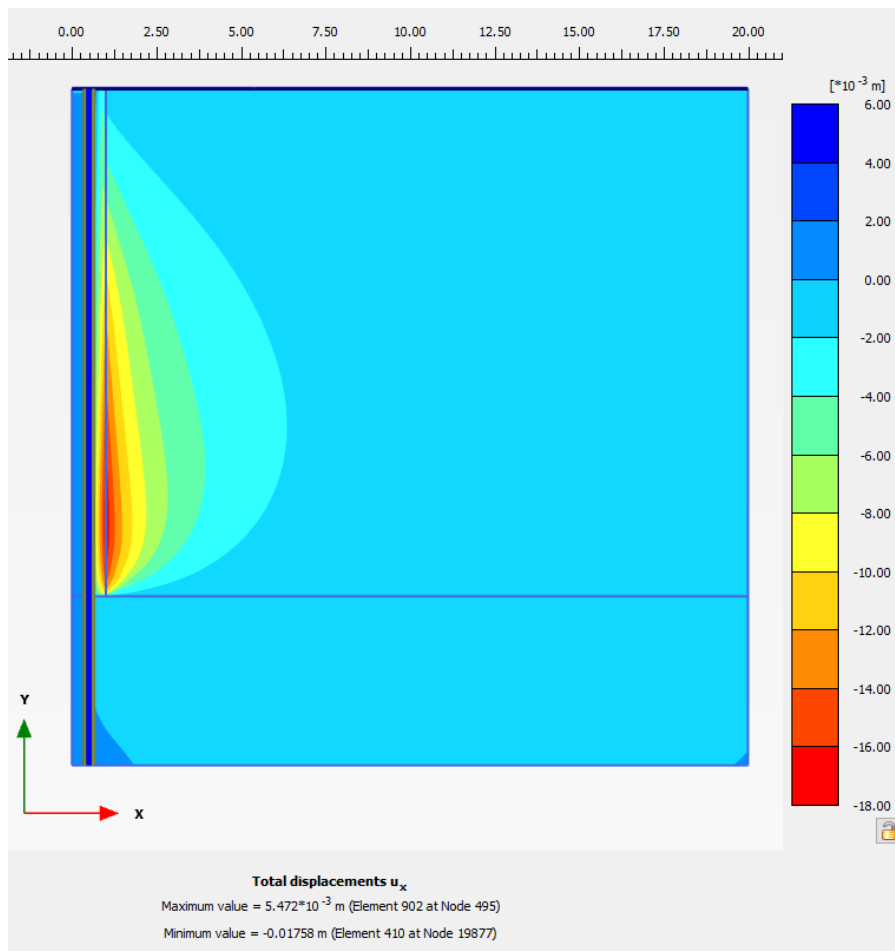Denoting these FEA as parametric analyses should be done with caution. When changing the stress situation in the soil, there is a whole lot of different parameters that can be controlled, and an assessment of all of them is a lot more time consuming than the time span of this thesis. The main interest has been on the variation of strain rate and influence area, and the last sections on the soil model and strength and stiffness variation has been conducted to show what influence this can have. Further on, the soil parameters chosen in these analyses are arbitrary and should not be mistaken as representative for real soil conditions. Field and laboratory testing of soil properties will probably show a larger correlation between stiffness, strength, density, water content etc. The aim in this thesis has been to lock the parameters for each analysis, and see what effect the variation of only one parameter have in FEA.

The idea of volume reduction is based on observations from straining and reconsolidating clay in oedometer tests, and is thus a secondary effect which can occur after some time. The time

span for the possible straining and reconsolidation will be dependent on the soil properties, which makes it hard to predict. Modelling this effect is most reasonably done in a drained situation such that the soil can deform. Picturing the case as partly undrained is therefore maybe a poor approach. It is unlikely that two neighboring soil volumes can react so differently.

However, modelling the soil as undrained will give excess pore pressure when applying strains/stresses. As seen in section 4.2.6, the volume reduction yields quite high suction at large depths.

It has been interesting to see that the total deformation and the applied volume reduction does not comply with each other. As mentioned in section 4.2.1, the reason for this is how PLAXIS deals with applied strains. This is further evident when observing that stiffness and strength will have a significant influence on the deformation pattern from applied volume strains. With large applied strains, it is also observed that the effective stresses reach zero, meaning that there is nearly no contact between the soil grains. In practical sense, this means that the only stabilizing force in this area is the hydrostatic pressure from the water, and the soil grains are only floating in this water. The occurrence of this for a large zone in-situ is considered unlikely, as this effective stress state would imply a great more disturbance than what is probable.

The focus in this section has been the applied volume reduction, based on straining and re-consolidation. A question of how large strains that could occur is therefore a timely one. From laboratory tests executed by Borchtchev, Eiksund et al. (2015), a correlation between applied strains and observed volume reduction was suggested. However, the strains applied were quite large, and especially the completely remoulded test is rather unlikely when relating the laboratory tests to in-situ conditions. Nevertheless, the laboratory results did show that the smallest applied shear strain of 18% would give a volume reduction as big as $5 - 6$ %, which gives considerable settlements when applied to a significant area. Determining how large the strains could be and the extent of the potential strained zone will remain a difficult task. Generally, pile drilling is assumed to have a much smaller impact than pile driving, and the aim is that pile drilling should not give any disturbance on adjacent soil at all. With this in mind, most of the analyses presented in this section are probably exaggerating the strain and size of strained area. The remaining possibilities of small areas and modest strain yields small settlements, which gives reason to question if this effect can be influential from a theoretical perspective.

5 FEA comparison to in-situ cases

Presented in the previous sections has been a walkthrough of the reasoning around assumed drilling effects, exemplified with construction projects, and implementation in FEA. It is important to mention that the negative effects related to drilling is observed more rarely than not observed at all. However, it is the cases where measurements and physical observations of unfortunate outcomes that often are remembered. The cases presented in this thesis are examples of this, even though the resulting settlements and pore pressure reductions did not cause any danger to neighboring infrastructure.

The similarity between the two cases presented is that both have been constructed with air driven DTH drilling systems, and the soil layering has been of the same characterization. Especially the moraine layer between bedrock and clay has been of interest, as there has been observed severe temporary pore pressure reductions here. These measured reductions are of such a magnitude that they cannot be ignored when evaluating the project. There has also been observed larger settlements than foreseen, which can be related to the pore pressure reductions when comparing the point of time for the observations. In the project Gladengveien 10, the settlements were considered partly due to an error when installing the sheet pile wall. However, the observed settlements did occur after drilling the piles in the excavation pit, which points in the direction of drilling operations as a cause for this as well. Hence, drilling operations can be considered as decisive for the observed pore pressure reductions and settlements in both the projects presented.

The FEA analyses on implementation of Venturi effect have only been focusing on pore pressure influence, while the disturbed zone has been considered as volume reduction and hence focusing on settlements. As mentioned previously, the possible volume reduction effect will be a secondary one, while the pore pressure fluctuations have been temporary. This gives reason to believe that the assumed Venturi effect is a better explanation of the observed measurements than the mechanical disturbance. Next task is then to relate the results from FEA to the observed in-situ measurements.

The first parameter analysis conducted was the influence of time span of the transient groundwater flow. These analyses should be seen in relation to both the boundary conditions and permeability. Comparing the FEA to the in-situ observations should therefore be done with basis in these parameters. However, the reports from the project does not describe the soil conditions for the firm masses in the moraine layer. Haugen, Ahmed et al. (2015) do include

some details on the described drilling process at the Hobøl River bridge, where there is noted that the drilling challenges had been largest for the soil masses with high content of silt. At Gladengveien 10 however, the occurrence of firm masses beneath the clay is only partly mentioned. This makes the evaluation of water flow from suction around the drill bit harder, as there is evidently no record of soil condition for these layers. Instead of using registered parameters for flow, the FEA can rather serve as basis for fitting the observed pore pressure reductions.

As seen in section 3.1.4 and 3.2.4, the measured pore pressure drops and rises again in between two measurements. This is further highlighted in Figure 5-1 and Figure 5-2, where two of the pore pressure measurements for the cases are presented with clear marks for the point of time for the measurements.
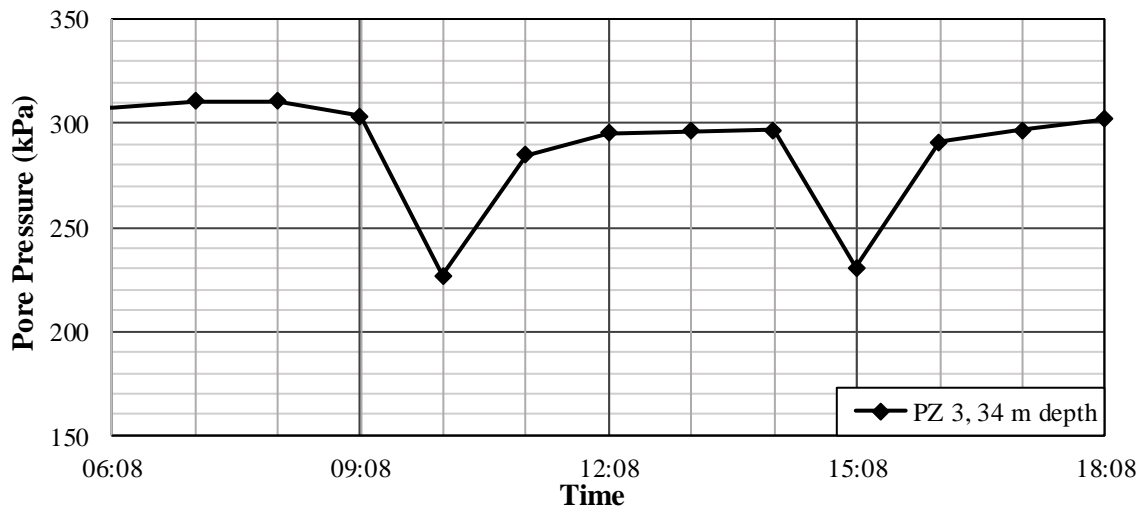

Figure 5-1: Pore pressure measurement in PZ3 for Gladengveien 10 for 15.05.2014
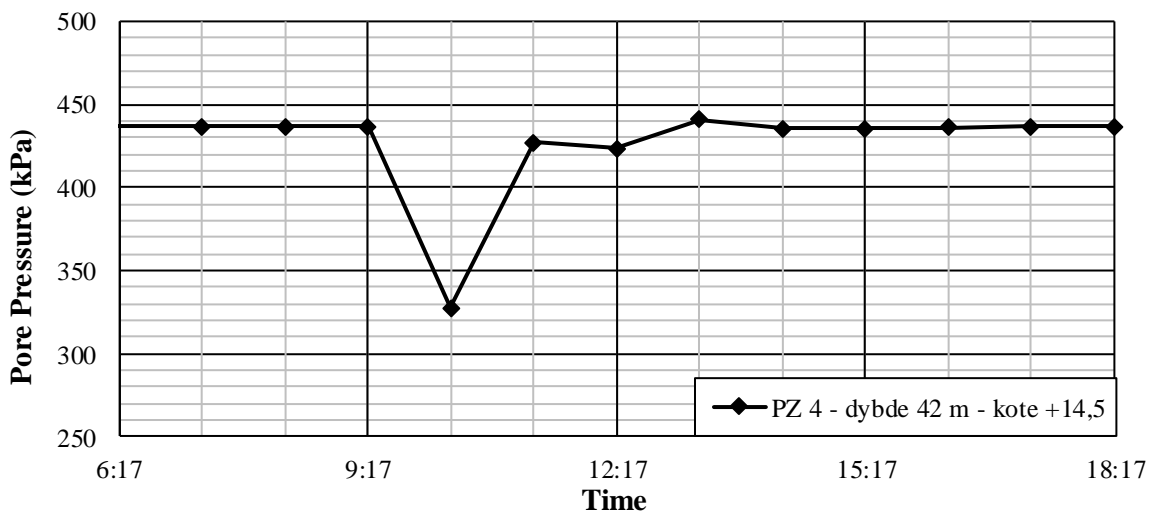

Figure 5-2: Pore pressure measurement in PZ4 for the Hobøl River bridge for 16.10.2014

The reason for choosing particularly these piezometers is that these are placed in the firm masses, and are therefore the piezometers which show the largest temporary pore pressure reductions. For both projects, it is evident that the measurements were done every one-hour for these piezometers. What happens in the piezometers in the meantime is unknown, and the duration of the reduced pore pressure is therefore unknown. There is also a possibility that the pore pressure fluctuations occur more often than measured, if the fluctuations are of short duration each time. At the same time, one can't overlook the opportunity of a persistent pore pressure reduction in between the measurements. Regardless of which of the opportunities of pore pressure development is the probable one, the observed measurements can serve as an indication of when fluctuations have occurred and the magnitude of the decrease (or increase).

The proposed Venturi effect is based on difference in fluid velocity, which can be related to changes in the cross-section of the fluid flow. These changes in cross-section is again due to unforeseen incidents during drilling, which usually are noticed and coped with in short time. The concept of the Venturi effect is therefore considered as temporary, just as the observed pore pressure reductions. This gives the possibility to directly compare observed measurements and FEA, using the transient pore pressure development seen in PLAXIS. The maximum pore pressure drop at Gladengveien 10 was approximately 70 kPa, while at the Hobøl River bridge the drop was as large as 110 kPa at most. Using the PLAXIS results as back calculation data, one could see what parameters are needed to reach the same values of change in groundwater head (1 m groundwater head = 10 kPa pore pressure). An important aspect here is distance between the piezometer with observed pore pressure drop and the triggering drilling operation. From the available protocols and drawings, the distance can be considered in the range from 2 – 4 m in the Hobøl River bridge project for the maximum measured pore pressure drop, while at Gladengveien 10 the distance is more uncertain because several piles have been installed at the same day where the maximum pore pressure reduction occurred. It is likely that it was the two closest piles that affected the measurements, and then the distance is around 4 – 6 m.

In section 4.1, several plots of groundwater head distribution with radial distance were presented, which were representative for all the results from PLAXIS. There were only a few analyses where such large pore pressure reductions as observed from the in-situ cases were observed. These were from the boundary comparison analyses in section 4.1.4, and some of the observations are shown in Figure 5-3 with the radial distance from pile in the horizontal axis.
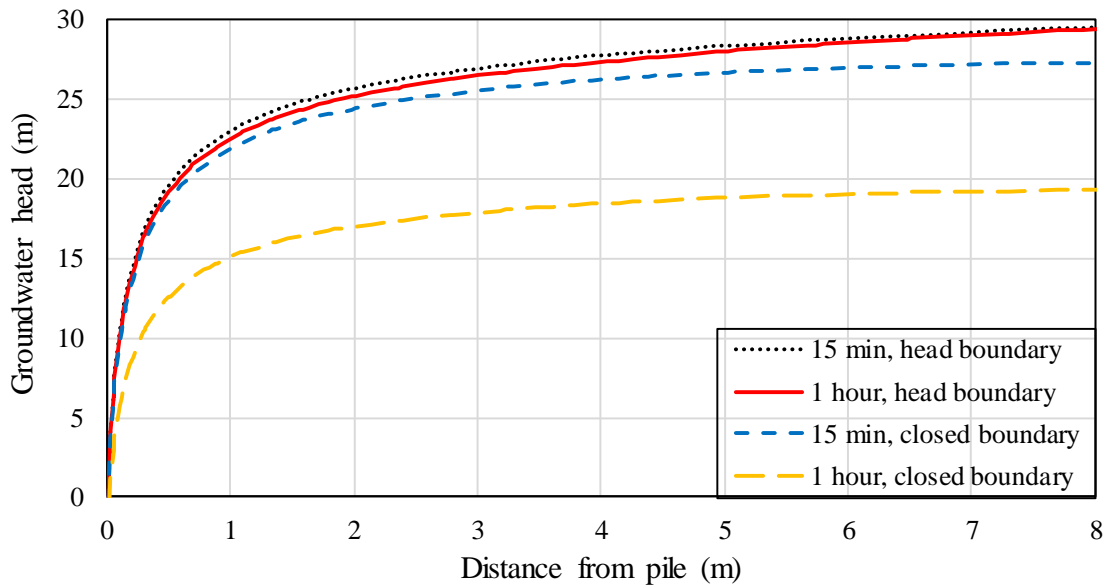
Figure 5-3: Groundwater head distribution with radial distance from pile observed in PLAXIS

Figure 5-3 is identical to Figure 4-13, but with the real radial distance instead of the relative distance at the x-axis. This is to show at which distance from the pile the different groundwater head is observed. The parameters in the analysis shown here is a range $R = 10$ m, $k = 1$ m/day, and $\Delta H = 37,5$ m. This situation shows a groundwater head decrease of 10 m all the way to the end boundary for the 1 hour analyses with closed boundary. At the closest distances the groundwater head decrease is even larger. This shows what parameters that are decisive when applying a groundwater flow in PLAXIS.

Recall from section 4.1.4 that the default case did not show the same large spread for the closed versus head boundary. This is fair when considering how much the soil volume in the axis-symmetric model increases with increasing radial distance. The observed drainage of the soil volume is interesting for several reasons, and especially the dependency on the range R. It is not unlikely to have a moraine layer that varies considerably in both thickness and composition with the radial direction from a drilling hole, such that a closed boundary can be assumed for a short time period. In-situ soil conditions will be more locally varying than what is applicable in PLAXIS, and especially in a thin moraine layer in between bedrock and thick clay masses. The evaluation of boundary conditions should also include the importance of being a 3D problem. In PLAXIS, this has been accounted for by using an axis-symmetric model. However, the limitation then is that the boundary applied is the same along the outer line. A further study should then probably be conducted in a 3D FE program.

Another feature to evaluating the boundary conditions is the effect of several piles. From the projects presented in chapter 3, the piling plans show that the distance between the piles have rarely been larger than 10 m. The question is then what an already installed pile (or pile casing) will have of effect to the drilling operation of a new one. If completely and correctly installed (with steel core pile and grouting), it should not affect the groundwater table. However, leakage along casing have been noticed at several projects (Baardvik 2015), and this could affect the groundwater situation enough to have impact on neighboring pile installation. From the projects observed here, the measurements seem to be independent of piling order. However, one cannot determine if previous drilling operations have affected the groundwater supply. Thus, the effect of several piles is a difficult task to implement in FEA, seeing how there is no indication of the impact.

The pore pressure measurements from the Hobøl River bridge and Gladengeveien 10 show that rapid draining with following slower influx of groundwater is a good explanation for the pore pressure fluctuations observed. As discussed above, the reason for the observations can be one of many or composed of several factors. The interest of the pore pressure fluctuations in this thesis has been to see them in relation to settlements, and the question is to what extent do the temporary pore pressure reductions contribute to observed settlements. In section 3.1.4 and 3.2.4, the settlement measurements and pore pressure measurements were considered correlated in some way, as the point of time for the two observations overlapped. Can the effective stress change due to pore pressure change explain the observed settlements? Probably not, as the pore pressure change is only temporary, and hence will the effective stress change be temporary as well. If strains are evolved in the short time span of the reduced pore pressures, then they will be reversed when the groundwater is stabilized again. The observed settlements should therefore be related to flushing out excess soil masses.

The quantification of excess masses being flushed out is a very difficult task. As discussed in section 4.1.5 - 4.1.7, some simplified calculation can serve as an approximation, but with no empirical data to compare with this will only be assumptions. Gathering of flushed out masses should be given more attention in future projects to further evaluate the possibility of an occurring Venturi effect.

Which fluid to use for driving the hammer piston and clearing the cuttings has not been emphasized so far. Both projects presented were drilled using air, so there is no available data for this thesis that pore pressure reductions also can be observed using water. There is a general understanding that air powered drilling is more critical than water due to compressibility of air,

and hence the capability of dissipating into cracks in the soil and creating disturbance. Several surveys support this, amongst them field tests of drilling systems done by Lande and Karlsrud (2015) and Ahlund, Ögren et al. (2016). Regarding the Venturi effect, air powered drilling can be considered more critical, because of increased risk of clogging in the flushing channels. However, the Venturi effect will occur for both fluids when constraining the cross-section of the flow, so that water powered drilling cannot be considered harmless.

In section 2.2.1 on the Venturi effect, two different ideas on how this can work were presented. Which one that is most likely as an explanation for an assumed suction has not been further discussed during the work here. To do this, field and/or lab work must be conducted to observe on micro level what happens during the drilling operation.

6 Conclusions and further work

As mentioned in the introduction, a conclusion will resemble more ta a suggestion on further work. Some useful observations have been made during the work, which can serve as preliminary conclusions on the comparison between FEA and in-situ measurements.

When evaluating the two effects presented in the thesis based on the measurements from reference cases and FEA, the assumed Venturi effect seems as the most representative one. The cases show clear temporary pore pressure reductions and a suction created around the drill bit seems as the best explanation for this.

The parametric analyses done regarding the Venturi effect have shown that the boundary conditions in the firm moraine layer have been most influential on the pore pressure distribution in radial distance from the pile. The boundary conditions in FEA should for further analyses try to cope with the hydrogeological situation experienced in-situ. The measurements from Hobøl River bridge and Gladengveien 10 also suggests that it is the drilling process when reaching the firm moraine layer that should be emphasized.

The identification and evaluation of unwanted effects from pile drilling is still an unexplored field, and to further substantiate the suggested theories more field work should be conducted. In connection to the Venturi effect, a lab-size model resembling the one presented in Figure 2-5 could be created to explore the suction created when changing the cross-section of the fluid flow, and especially to see the effect this will have on soil particles. A model like this could also indicate the differences experienced with water versus air flushing, and by using different types of sand, silt and clay the parameter effects could also be evaluated.

Further analytical work should also be conducted. The groundwater flow analysis in PLAXIS has here been done with a linear elastic material which is not affected by groundwater flow. It has been observed that the flow gradients close to the drill bit will be very large, and with other soil models, it is more likely to see some fracturing due to the effective stress change and entrainment force from the water. This, together with the boundary conditions are considered the two most interesting results from the parametric analyses in PLAXIS.

Based on the wide extent of the work done, as opposed to a more specified problem and conclusion, the list of possible further work would be large. However, the suggestions listed above are the ones considered most interesting from the writer's perspective.

Acronyms

English

A = cross-sectional area

a = attraction

c' = cohesion

E' = effective Young's modulus

g = acceleration of gravity

h, H = groundwater head

i = gradient

K = hydraulic conductivity

k = permeability

L = length

P = fluid pressure

p'_{re} = reconsolidation stress

q = groundwater flow

Q = discharge rate

R = range of PLAXIS model

r = radial distance

r_{in} = radius of the inner border of a soil cluster

r_o = radius of the outer border of a soil cluster

$\bar{r}_i$ = average radial distance to increment i

Δr_i = thickness in radial direction for increment i

Δs = flow distance

s_u = undrained shear strength

$\bar{u}_{yi}$ = average settlement for increment i

V = volume

v = flow velocity

w = water content

w_{re} = water content after reconsolidation

z = height of flow above a reference height

Greek

γ = soil weight

γ_s = applied shear strain

$\varepsilon_{vol},\ \varepsilon_V$ = volumetric strain

v = Poisson's ratio

ρ = fluid density

σ' = effective stress

τ = shear stress

φ = friction angle

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