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Doctoral thesis

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Norwegian University of Science and Technology
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Preface

This Phd study was carried out at the Geotechnical Group at Department of Civil and Environmental Engineering at the Norwegian University of Science and Technology (NTNU). The work is presented as a collection of journal and conference papers complemented by a summary. Professor Steinar Nordal at NTNU was the main supervisor, and Technical expert Kjell Karlsrud from NGI was the co-supervisor.

The study was partly funded by the Norwegian Geotechnical Institute (NGI) and through the research programs *Limiting Damage (*BegrensSkade, BIA project 219951/030) and *Risk Reduction of Groundwork Damage* (BegrensSkade II, BIA project 267674) financed by the Norwegian Research Council and the project partners.

The evaluation committee consisted of Professor Claes Alén (1st opponent) from Chalmers University of Technology, PhD Maj Gøril Bæverfjord (2nd opponent) from Dr. Techn. Olav Olsen AS, and Professor Rao Martand Singh (administrator) from the Geotechnical Group at NTNU.

Abstract

Several cases with unexpected damages related to excessive ground movements induced by initial and secondary effects from overburden drilling to install tieback anchors and piles from inside deep excavations were observed over the last couple of decades in Norway. Overburden drilling is characterized by continuous permanent casings that are drilled through varying soils (i.e. overburden) and with an embedment in bedrock. Based on the lack of knowledge regarding the effects of drilling on the surrounding ground, and the associated risks of excessive ground settlements, the objectives of this PhD study are to:

a. Identify and analyze effects on pore pressure changes and ground settlements caused by overburden drilling for piles and tieback anchors installed in different soil conditions by using different drilling methods and procedures.
b. Establish recommendations and guidance for execution of overburden drilling to limit negative installation effects and risk of damage on surrounding structures or utilities.

A full-scale field test program investigating drilling for tieback anchors through soft marine clay and into bedrock was conducted to study the impacts on pore pressures and ground settlements. Five different rotary percussive duplex drilling methods including air- and water-driven down-the-hole (DTH) hammers and top hammer systems were investigated. The field tests showed that drilling in clay with high penetration rates (>1 m/min) combined with water flushing caused excess pore pressures in the surrounding clay with a larger influence zone than experienced with driven closed-ended piles of the same dimensions. Further, the tests showed that drilling with the DTH air hammer caused larger impacts on both pore pressures and settlements than the other methods, indicating a higher risk of unwanted effects on the surrounding ground.

Although the full-scale tests provided new insight into overburden drilling, the complex drilling processes involved made it difficult to investigate in detail the mechanisms and drilling parameters affecting the surrounding ground. Therefore, a series of novel small-scale pile drilling tests in saturated sand were performed. The main objective of this physical model was to explore the role of drilling parameters such as flushing media (water or air) and flow and penetration rates on the surrounding soil. The results from the water flushing tests showed a distinct relation between the flow and penetration rate and the resulting influence on the surrounding ground. Increasing flow rates caused larger excess pore pressures at greater radial distances and generated more excess drill cuttings compared to the theoretical casing volume. Based on the test data, a framework of normalized flow rate and normalized mass of drill cuttings was introduced, which could be used to derive ideal drilling parameters in similar conditions. The air flushing tests were considerably limited by modelling constraints; thus, no clear conclusions could be drawn from those tests. However, notable reductions in pore pressures adjacent to the casing indicated an air-lift pump effect that can lead to extensive ground settlements as observed in the field.

Five well-documented case studies of overburden drilling for foundation piles are provided. The field data illustrates that rotary percussive duplex drilling with air flushing (top hammers and DTH air hammers) frequently caused an air-lift pump effect at the front of the drill bit that may lead to significant local erosion and loss of soil mass (i.e. cavities). This effect was pronounced when drilling in erodible soils like silt and sand, which are typically for glacial tills (e.g. moraine material) often found below marine deposits in Norway. Drilling through confined aquifers with artesian pore-water pressures and a high recharge of ground water further increased the risk of excessive erosion which may result in considerable ground displacements and damage in surrounding areas. Drilling with water-driven DTH hammers greatly reduced the risk of excessive soil volume loss and ground displacements compared to drilling with air hammers.

Practical recommendations were derived from the research carried out to guide practitioners in the design, execution, and evaluation of overburden drilling. Hence, this work will reduce the risk of damage caused by overburden drilling and thus realize potential cost savings in the building, construction, and property sector.

Acknowledgements

The work which is presented in this PhD thesis would not have been possible without the direct and indirect contributions from many colleagues in the geotechnical community in Norway. This includes the many companies being partners in the R&D projects *Limiting Damage* and *Risk Reduction of Groundwork Damage* (Limiting Damage II) as well as several colleagues at the Norwegian Geotechnical Institute (NGI). I would like to thank everyone for their collaboration and constructive discussions, and not to forget the willingness to share monitoring data and experiences from numerous construction projects during this study which has been going on for nearly 10 years. I am grateful for the research scholarship and funding from NGI making this study possible.

I would also like to thank my colleagues at NGI working together in the Limiting Damage projects. Without a dedicated team effort these research projects would not have been successful. I would particularly like to thank Jenny Langford (now Norconsult) for leading these large projects over so many years and with so many partners. Thank you for providing support to the entire team, and your efforts to help me finalize this work. I would thank Axel Walta, Ole Petter Rotherud and the excellent staff at NGIs mechanical workshop for the help with the mechanical design, data acquisition system and for helping solve practical issues related to the small-scale model tests.

To Guro Grøneng, Market Director at NGI, thank you for the great support and motivation during the last years by giving me the chance to prioritize and complete this work.

Special gratitude goes to my supervisors, Professor Steinar Nordal at NTNU, and Kjell Karlsrud at NGI for their guidance, support, and constructive feedback along the way. I appreciate the interesting discussions and advice during these years, helping me to develop as a researcher. Thank you both for believing in me and encouraging me to finish this work.

Special thanks go to my colleague Stefan Ritter, who joined NGI and the *Limiting Damage II* project in 2018. I truly appreciated working together on planning, executing, and later analyzing results from experimental small-scale model tests. Your great "research mind", enthusiasm and willingness to help have taught me a lot and improved the quality of the research.

I also thank Henning Tvvold (Norconsult) for helping with preparing and carrying out many of the model tests during his summer job at NGI, and later following up with refined model testing during his Msc thesis in geotechnics at NTNU.

To my beloved wife, Ingrid Helene – thank you for your continuous support and patience during all these years. Without your great efforts taking the main responsibility for our family, this would not have been possible. You are always there, caring for and motivating those around you. To our son Leon William and daughter Stella Charlotte, you have opened my eyes to what is truly important in life. I love you both deeply.

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Nomenclature

Latin letters

A_{pile}	cross-sectional area of pile (dm^2 or m^2)
c_u	undrained shear strength (kPa)
c_{uA}	undrained active shear strength (kPa)
d	installation soil depth
d_{10}	10% fractile in a grain size distribution
D_{50}	50% fractile in a grain size distribution
d_{60}	60% fractile in a grain size distribution
d_t	diameter of the drill rod (mm)
D_h	diameter of the borehole or inside of the casing (mm)
D_r	relative soil density (-)
E	deformation modulus
e	void ratio (-)
e_{max}	maximum void ratio (-)
e_{min}	minimum void ratio (-)
G_s	grain density ($cm/^3$)
G_{50}	undrained secant shear modulus at 50% of shear strength mobilization
I_p	plasticity index (%)
$L_{p,c}$	length of casing in clay
$L_{p,t}$	length of casing in till
$M_{c,norm}$	normalized mass of drill cuttings (g)
M_c	measured mass of drill cuttings (g)
$M_{c,c}$	soil volume balance in clay given by $V_{c,c}/V_{p,c}$

$M_{c,t}$	soil volume balance in till given by $V_{c,t}/V_{p,t}$
M_{pile}	theoretical mass of soil given by the installed pile volume (g)
OCR	over-consolidation ratio = p_c'/σ'_{v0} (-)
PZ	piezometer
p_c'	apparent pre-consolidation stress, defined from oedometer test (kPa)
Q	flushing flow rate (dm^3/min)
Q_{air}	air volume passed by the hammer at a given air pressure (m^3/min)
Q_{norm}	normalized flow rate (-)
r_{pl}	plasticized radius from the pile center
r_0	outer radius of the pile (casing)
r_1	radius of completely remolded clay around casing
r_2	radius of partly remolded clay
S	settlement anchor
SPW	sheet pile wall
S_t	clay sensitivity (-)
U	measured pore-water pressure (kPa)
u_{ref}	reference pore-water pressure (kPa)
V	drilling penetration rate in model tests (mm/s)
V_1	volume of completely remolded clay
V_2	volume of partly remolded clay
$V_{BF,c}$	total volume of backflow from drilling in clay
$V_{BF,t}$	total volume of backflow from drilling in till
V_{pen}	drilling penetration rate (m/min)
$V_{p,c}$	volume of pile casing in clay
$V_{p,t}$	volume of pile casing in till

$V_{c,c}$	volume of in-situ clay from drilling backflow
$V_{c,t}$	volume of in-situ till from drilling backflow
w	water content (%)

Greek letters

δv	vertical displacements, ground settlements (mm)
γ	soil unit weight (kN/m^3)
γ_{rz}	shear strains (%)
ΔU	change in pore-water pressure (kPa)
ΔU_{max}	maximum pore-water pressure changes related to u_{ref} (kPa)
ΔV_1	volume loss in completely remolded clay
ΔV_2	volume loss in partly remolded clay
ε_v	volume reduction due to re-consolidation of clay
$\varepsilon_{vol;1}$	volume reduction of completely remolded clay due to re-consolidation
$\varepsilon_{vol;2}$	volume reduction of partly remolded clay due to re-consolidation
ν	Poisson's ratio (undrained conditions = 0.5)
ρ	soil density (kg/m^3)
σ'_{v0}	in-situ vertical effective stress (kPa)

1 Introduction

1.1 Background and motivation

It is well known that deep-supported excavations in soft clay deposits can cause significant ground settlements in the areas surrounding the excavations. In urban areas the potential for causing damage to neighboring buildings and structures can be considerable, and the monetary costs related to these damages can be substantial. A study by the Swedish Geotechnical Institute (SGI 2013) found that the costs related to damages from building- and construction projects was about 10% of the total project costs. About 1/3 of these costs were related to ground works which gives an annual cost of about 9 billion Swedish kroner. Several cases with unexpected damages related to excessive ground settlements from deep excavations were observed over the last couple of decades in Norway. This was the background for the research project named *BegrensSkade* (*Limiting Damage*) that was carried out between 2012 to 2015 with funding from the Norwegian Research Council and the 23 Norwegian partners covering stakeholders, contractors, consultants, research institutes, and the Norwegian University of Science and Technology (NTNU). The project was followed up with *Risk Reduction of Groundwork Damage (Limiting Damage II)* which started in 2017 and lasted until autumn 2022.

One of the main objectives with the *Limiting Damage* project was to identify and investigate the causes of excessive settlements related to deep excavations and foundation works in soft clays. A number of well-documented case records were compiled and analyzed with respect to possible causes of unexpectedly large ground movements and pore-water pressure changes (Langford et al. 2015; Baardvik et al. 2016). The monitored data were assessed in relation to the specific construction methods adopted. Based on these data it was concluded that installation of tieback anchors and/or foundation piles by drilling from within the excavation pits significantly increase the risk of pore pressure reduction and subsequent consolidation settlements in soft clays, as well as settlements caused by mechanical disturbance and erosion of soil adjacent to the drilling.

In areas with limited depth to sound bedrock, tieback anchors and piles are often installed by drilling of a continuous permanent casing to support the borehole through varying soils (i.e. overburden) and with an embedment in bedrock. This drilling methodology is in this thesis called "overburden drilling".

In Scandinavia, the use of overburden drilling for anchors and piles (both micropiles and large diameter steel pipe piles) has increased significantly during the last decades. Statistics from Sweden show that about 30% of the total installed pile lengths in 2015 were steel piles, out of which 40% were installed by overburden drilling (Commission on Pile Research 2016). There are several reasons for this: (1) Typical ground conditions with soft clays overlying solid bedrock favors anchors and piles to bedrock due to considerably larger capacity compared to soil anchors and friction piles. (2) Contractors often prefer tieback anchors instead of internal strutting for deep excavations due to more efficient excavation and construction processes. (3) Installation of piles by drilling can be performed efficiently by relatively small, lightweight drill rigs. (4) Piles installed by drilling and grouting into bedrock can resist both axial compression and tensile loads.

Figure 1-1 illustrates the main contributions to ground movements, induced by a deep supported excavation in soft clay overlying bedrock or stiff permeable soils based on the studies by Langford et al. (2015):

I. Shear-induced ground movements caused by lateral displacements of the supporting wall and deep-seated displacements (tendency for bottom heave failure mechanism).

II. Ground water leaks up along the casing tubes and into the excavation pit, causing pore pressure reductions in the surrounding ground and consolidation settlements in soft clay.

III. Installation effects induced by drilling for tie-back anchors and/or drilled piles inside the excavation, such as disturbance of the soil structure, erosion, and loss of soil mass during drilling.

Peck (1969) suggested that for strutted excavations in soft clays, ground surface settlements corresponding to up to 2-3% of the excavation depth could occur in soft clays with poor stability conditions. Mana and Clough (1981) showed that the maximum expected lateral wall movement was closely related to the safety factor against bottom heave, but the stiffness of the support system is a significant factor. However, the data from Peck (1969) and Mana and Clough (1981) do not consider consolidation due to drainage of ground water and decreased pore-water pressure levels in the soils overlying bedrock.

A numerical parametric study by Karlsrud and Andresen (2008) generally confirmed the trend in data included in Mana and Clough (1981). The study also showed that the displacements should be limited to about 1% of the excavation depth when using current codes and guidelines, which require a minimum factor of safety of at least 1.4 against bottom heave.

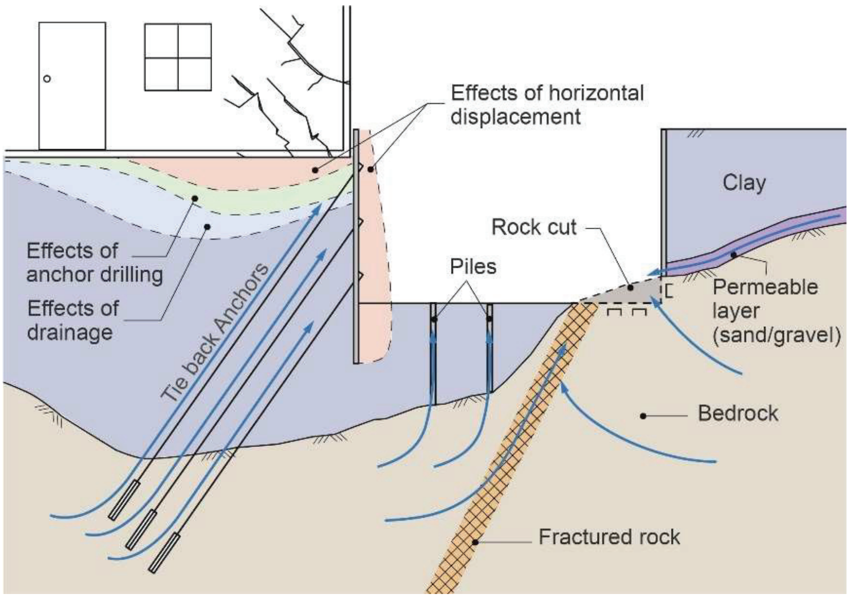


Figure 1-1. Illustration of main contributions to ground settlements induced by deep supported excavations in soft clay overlying bedrock or stiff soils (from Langford et al. 2015).

Figure 1-2 presents ground settlement data from 48 construction projects with deep excavations in soft and medium stiff marine clays supported by sheet pile walls (SPW) analyzed in the R&D-projects *Limiting Damage* and *Limiting Damage II*. Measured "long-term" (one year after final excavation) ground settlements (δ_V) is normalized with the excavation depth (*H*) and plotted against the distance from the excavation (*x*) normalized to *H*. Red circles represent projects where tieback anchors and foundation piles (e.g. steel-core piles) both were installed with overburden drilling. Projects with drilled tieback anchors and driven or no piles are shown with red triangles. Projects involving internal struts and drilled piles are marked with yellow circles, while the green squared symbols are for projects with internal struts and driven or no piles. Expected displacements based on the studies by Peck (1969) and Hsieh and Ou (1998) are included for comparison, shown by the dotted black lines and the solid black line respectively.

The monitoring data indicates that the use of drilled tieback anchors and/or piles may cause significantly larger settlements close to the excavation than expected due to displacement of the retaining wall alone. These excessive settlements are related to the installation effect III described above. Ground settlements observed at further distances from the support structure (x/H > 4) are considered to be related to consolidation of soft clay deposits due to drainage (i.e. leakage) of ground water into the building pits and reduced pore-water pressures at bedrock level. The water leakages may come from excavation of bedrock surface or permeable soil layers within the excavation, but also from drainage along casings for anchors and piles as illustrated in Figure 1-1.

Although installation effects related to drilling through soft cohesive soils, granular soils (e.g. sand, gravel, moraine) and into bedrock is recognized in some literature, the problem has not been systematically addressed and scientifically studied. There is a lack of knowledge regarding the effect of drilling on the surrounding ground, and the risk of causing excessive ground settlements. Apart from some general guidelines regarding the design and implementation of drilled piles (Finnish Road Authorities 2003), there are no specific guidelines for selecting appropriate methods for overburden drilling to reduce the risk of ground settlements which could have detrimental effects on surrounding structures and utilities.

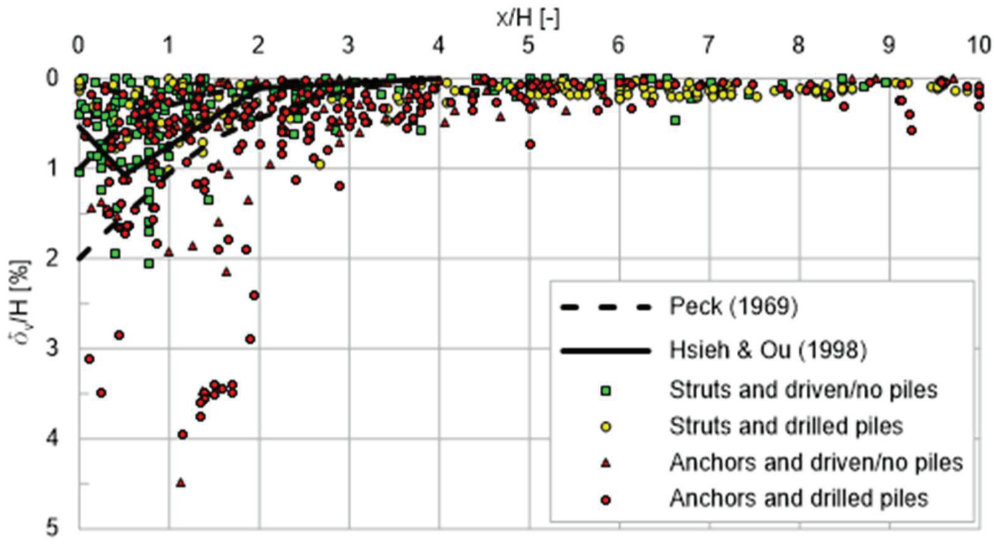


Figure 1-2. Normalized measured ground settlements (δ_V) with excavation depth (H) against distance (x) from the support structure normalized with the excavation depth (H). Modified from Langford et al. (2015).

1.2 Scope and objectives

The scope of the present PhD research is to investigate and deepen the understanding of the effects from overburden drilling on the surrounding ground. The work has been carried out as part of the research projects *Limiting Damage* and *Limiting Damage* II as a combination of field studies including a full-scale field test and numerous case studies, plus physical modelling in the laboratory. The objective of the PhD research is to:

- Identify and analyze the main effects of overburden drilling on pore pressures and ground settlements under different soil conditions and using different drilling methods. Installation effects were studied through full-scale field tests with drilling in clay, model tests of drilling in sand, and case studies with deep excavations and pile drilling.
- Use the obtained field and laboratory results to provide guidance on drilling methods and procedures that can reduce the risk of excessive ground settlements.

1.3 Outline of the thesis

The PhD thesis is paper based and written as an extended summary of the papers included at the end of the thesis. The results and findings in the papers are included in the following sections and are organized as follows:

1. Introduction: presents the background and motivation, scope and objectives, and a list of the publications from the present PhD study.
2. Drilling methods for piles and tieback anchors: describes the methodology of overburden drilling for piles and tieback anchors and basics of different drilling systems as mentioned in Paper I, II and V.
3. Literature review: summarize relevant literature related to overburden drilling and its effect on the surrounding ground. The section relates to Paper I, II and V.
4. Effects of overburden drilling: describes the main installation effects from drilling based on case studies, available literature, and field and model tests discussed in Paper II, IV and V.
5. Full-scale field tests: summarizes the main findings from full-scale field tests with drilling of anchors through soft clay and into bedrock as described in Paper I and II.
6. Small-scale modelling of pile drilling in sand: describes and presents the main results from small-scale model tests on pile drilling in saturated sand.
7. Case studies of overburden drilling of foundation piles: documents effects on pore-water pressures and settlements from overburden drilling of piles as described in Paper V.
8. Recommendations to reduce impacts on the surroundings from overburden drilling: describes the recommended practice for drilling of piles and tieback anchors based on the results from full-scale field tests, small-scale model tests, and case studies as described in Paper V.
9. Summary
10. Conclusions:
11. Further research: presents suggestions for further research related to drilling for piles and tieback anchors.

1.4 List of publications and declaration of authorship

The thesis is paper based and consists of five papers in total. Two of the papers are published in peer-reviewed journals (Papers II and IV). One of the papers is accepted for publication in a peer-reviewed journal (V). Two papers have been published in peer-reviewed conference proceedings (Paper I and III). The papers were written by EJ Lande with contributions from the co-authors listed below together with acknowledgments.

Paper I: Lande, E.J. and Karlsrud, K. (2015). *Full scale field test – drilling of anchors to bedrock in soft clay.* In Proc. of the XVI European Conference on Soil Mechanics and Geotechnical Engineering, Edinburgh 13-17th September 2015. ICE Publishing, London, pp. 625-630.

- The paper describes and briefly presents the main results from a full-scale field test with drilling of anchors through soft clay and into bedrock using five different drilling methods. The tests were carried out in 2013 as part of "work package 4" of the research project Limiting Damage. E.J. Lande was responsible for planning the tests including the instrumentation program, and he followed up the tests on site and was responsible for data acquisition and analysis. A.K. Lund, K. Karlsrud, A. Eggen (all NGI), and the drilling contractors Fundamentering AS (FAS), Brødrene Myhre AS, Nordisk Fundamentering AS, Entreprenørservice AS and Hallingdal Bergboring AS who carried out the drilling all contributed greatly to the planning with valuable discussions on the test set-up. K. Karlsrud are acknowledged for valuable discussions of the results and the manuscript.

Paper II: Lande, E.J, Karlsrud, K., Langford, J. and Nordal, S. (2020). *Effects of drilling for tieback anchors on surrounding ground - results from field tests*. Journal of Geotechnical and Geoenvironmental Engineering, 146(8): 05020007.

- Paper II presents the same full-scale field test as paper I, however it includes more complete results and assessment of the results. E.J. Lande analyzed the results and wrote the manuscript. K. Karlsrud, J. Langford and S. Nordal are all acknowledged for discussions of the results and the manuscript. Three unknown reviewers are acknowledged for comments greatly improving the overall quality of the paper.

Paper III: Lande, E.J., Ritter, S., Tyvold, H. and Nordal, S. (2021). *Small-scale modelling of pile drilling in sand – investigation of the influence on surrounding ground*. In Proc. of the 10th International Symposium on Geotechnical aspects of Underground Construction in Soft Ground, Robinson College, Cambridge, UK., 27-29 June 2022. Taylor and Francis publishing.

- The paper describes and presents preliminary results from a novel model test on drilling with water flushing in saturated sand with a miniature pile. The tests were carried out as part of the research project Limiting Damage II in 2019. The idea was proposed by E.J. Lande who together with S. Ritter (NGI) planned and carried out the tests. H. Tyvold (NTNU) is acknowledged for helping to carry out the tests and presenting results. A. Walta (NGI) is greatly acknowledged for the mechanical design of the model pile and rotation motor and O.P. Rotherud (NGI) for help with setting up the instruments and data acquisition system. The excellent staff at NGIs workshop are also deeply acknowledged for help with several practical issues related to the model test. S. Ritter and S. Nordal were heavily involved in discussing the results and the manuscript. One unknown reviewer is acknowledged for valuable comments on the manuscript.

Paper IV: Lande, E.J., Ritter, S., Tyvold, H. and Nordal, S. (2021). *Physical modelling of pile drilling in sand*. Canadian Geotechnical Journal, 58: 1437-1451(2021).

- The paper provides a more detailed description and supplementary test results from the same model tests as in Paper III. The effects of varying parameters such as flushing media (water or air), flow and penetration rate on the penetration force, pore pressure changes, soil displacements and drill cutting transport are studied. S. Ritter and H. Tyvold are acknowledged for their help carrying out the tests and processing of results. S. Ritter and S. Nordal were both central in discussing the results and manuscript. Gratitude is given to two unknown reviewers who helped improve the quality of the manuscript.

Paper V: Lande, E.J., Ritter, S., Karlsrud, K. and Nordal, S. (2024). *Understanding effects from overburden drilling – a rational approach to reduce the impacts on the surroundings"*. Canadian Geotechnical Journal. Paper accepted and published as "Just-In", https://doi.org/10.1139/cgj-2023-0404.

- The paper presents two case studies from Norway with overburden drilling of casings for end bearing piles through massive deposits of clay, compact glacial till and into bedrock. The first case involves drilling of 711 mm diameter casings for pile group foundations with air driven DTH hammer and a concentric drilling system. The second case involves a unique comparison of drilling casings (273-406 mm diameter) for steel core piles with air vs water DTH hammer. Results from measurements of pore-water pressures and ground settlements are assessed to study the installation effects from drilling. This provides a basis for planning and execution of overburden drilling to reduce the risk of potential detrimental impact on the surroundings. E.J. Lande was responsible for planning the instrumentation for the first case, gave advise on the second case, and analyzed the monitoring results for both cases. S. Ritter is greatly acknowledged for discussions of the field data and together with K. Karlsrud and S. Nordal raising the quality of the research and manuscript. The Norwegian Public Roads Administration, NCC Norway AS, Hercules Fundamentering AS, Sykehusbygg HF, Norconsult AS, and Keller Geoteknikk AS are all acknowledged for their willingness to share and discuss results and experiences from the cases.

2 Drilling methods for piles and tieback anchors

2.1 Introduction

The successful installation of tieback anchors and/or end-bearing foundation piles requires efficient and safe drilling of holes through soils (i.e. overburden) and rock. The diameter of the boreholes typically varies between approx. 75-194 mm for anchors and 114-1016 mm for piles, while the lengths can vary from a few meters to more than 80 m. There are several methods and systems available for drilling in ground conditions varying from very soft soils to hard bedrock. The drilling method should generally be able to create straight and stable boreholes with a constant diameter, from which the drill cuttings have been fully removed. The drilling method selected by the specialty contractor has traditionally been based on efficiency, cost of construction and reliability, and often with less focus on minimizing soil disturbance and detrimental effects on the surroundings. Drilling in an urban environment close to sensitive older buildings sets different constraints on the execution compared to drilling in open areas.

The Federal Highway Administration (FHWA 2005) and the Finnish Road Authorities (2003) have published guidelines on micropile design where different drilling methods are described. According to FHWA drilling can be divided into two main categories: 1) open hole drilling, i.e. without casing, or 2) with a continuous casing supporting the borehole. The latter is referred to as "overburden drilling" and is the focus in this study. The micropile manual by the FHWA divides overburden drilling into the following 7 methods:

1. Single tube advancement;
2. *Rotary duplex;*
3. *Rotary percussive concentric duplex;*
4. *Rotary percussive eccentric duplex;*
5. *Double head duplex;*
6. Hollow-stem auger and
7. Rotary vibratory (sonic)

The focus in this study is on the four rotary duplex methods (No. 2-5) which are most relevant for drilling tieback anchors and piles through soils and further into bedrock. The basic principles of these rotary duplex drilling methods and systems are given in the following.

2.2 Drilling methods

2.2.1 *Rotary duplex*

The rotary duplex method involves combined rotation and advancement of the internal drill rod with a suitable drill bit inside the drill casing. The drill rod and casing are attached to the rotary head on the drill rig from which the flushing fluid (often water or polymer) is pumped through the drill rod and exits from the flushing ports in the drill bit. The flush-borne drill cuttings are transported up to the surface through the annulus between the casing and the drill rod where it either exits directly from the top of the casing or through ports in the drill head (FHWA 2005). Such "rotary-flush" drilling is well-suited for soft and sensitive ground conditions (e.g. clays).

2.2.2 *Rotary percussive duplex*

Overburden drilling through varying fill materials, boulders, dense granular layers and into bedrock necessitates percussive drilling to achieve penetration. This is usually carried out with the so-called rotary percussive duplex method (FHWA 2005). This method is similar to the rotary duplex method but includes percussion on the drill bit.

The percussion on the drill bit is provided by either a top hammer acting on top of the drill rod or a down-the-hole (DTH) hammer located just above the drill bit (Finnish Road Authorities, 2003). Figure 2-1 illustrates three systems for overburden drilling where the drill bit is both percussed and rotated. Figure 2-1 a) shows a hydraulic powered top drive (top-hammer) with an eccentric drill bit where both rotation and percussion are applied at the top of the drill rod by the drill head of the rig. Since the percussive energy is transferred through the drill rod the net energy on the pilot bit progressively decreases with increasing depth/length of the borehole. Therefore, top hammer drilling is limited to typically 30-40 m depths and casing diameters up to 140 mm.

With the DTH hammers the percussion hammer is located just above the drill bit, and the drill rod is rotated by the drill head as illustrated in Figure 2-1 b) and c). Figure 2-1 c) illustrates a reversed circulation (RC) drilling system with a double tubed drill rod (dual wall) where the cuttings and flushing returns along the inner tube. Unlike the top hammers, DTH hammers maintain an effective penetration rate at depth and produce less noise.

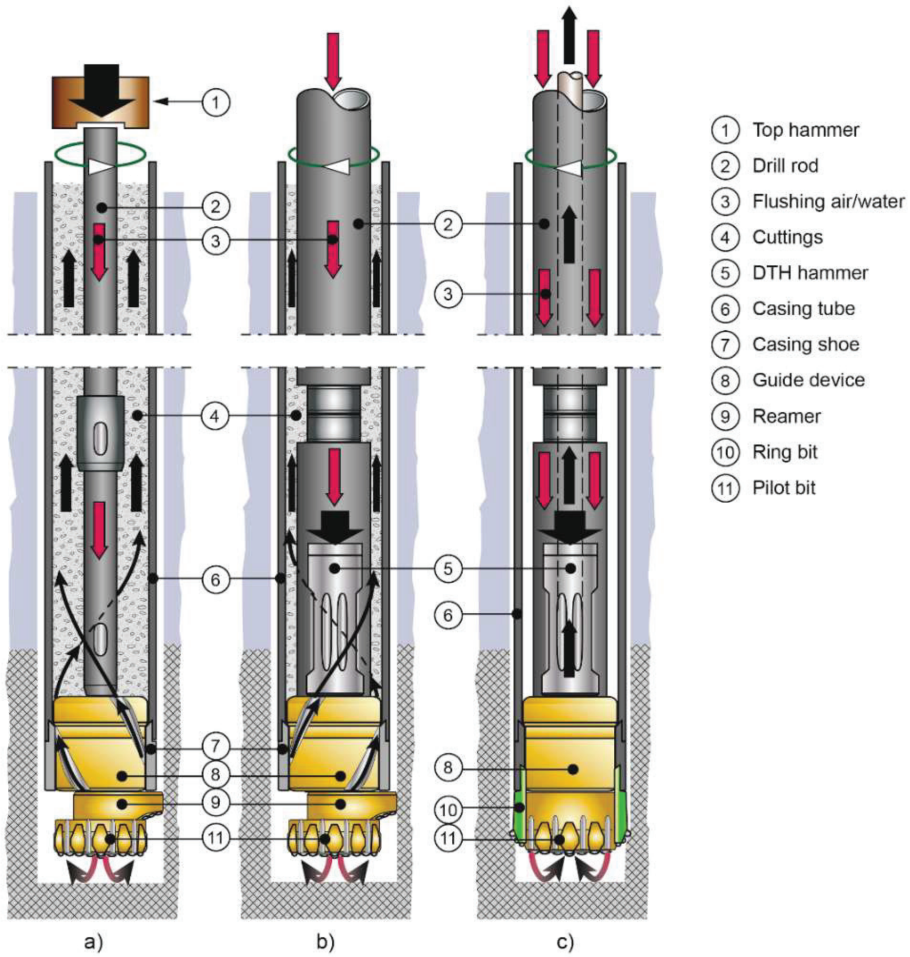


Figure 2-1. Illustration of drilling methods and drill bits used for overburden drilling with casing advancement: a) top-hammer, b) Down-The-Hole (DTH) hammer, c) DTH hammer with reversed circulation (RC).

2.2.3 *Double head duplex*

There is a drilling method known as double head duplex where the drill rod and casing are rotated in opposite direction by separate drill heads on the rig. This enables the inner drill rod to be operated "freely" from the casing, meaning that the front of the drill bit can be placed a small distance behind or in front of the casing tip depending on the ground conditions. The drill rod may be operated by purely rotation or rotary percussion using DTH or top hammers as the other methods described above.

2.3 **Air vs water flushing**

Efficient flushing with a suitable fluid is needed to continuously cool the drill bit and transport drill cuttings up from the borehole to avoid reduced penetration and increased bit wear. Top hammer drilling can be carried out with either water or air flushing through the drill rod. DTH hammers on the other hand require flushing with compressed air or high-pressure water to run the hammer (Halco Rocktools 2021; Wassara 2021). Both types of DTH hammers follow the same general concept; a moving piston inside the hammer is driven by the air/water flushing. The piston transfers the percussive energy directly on to the top of the pilot bit in which a stress wave propagates through the pilot bit inducing high point stresses between the tungsten-carbide points and the soil/rock (Chiang and Elias 1999).

DTH air hammers can operate under flushing pressures varying from about 7 bars to more than 30 bars. It is the air pressure which controls the hammer's impact energy and frequency, given that the compressor can deliver adequate air volumes to maintain the pressure. The air consumption expressed in m^3/min is a result of the number of blows per minute multiplied with the volume of the hammer chamber above the piston. As the compressed air passes the hammer and flows out through the ports in the drill bit it expands (i.e. air volume increases) due to the lower pressure. The air flow "pushes" the drill cuttings (soil particles) from the face of the drill bit and transports them up to the surface in the annulus between the drill rod and the casing. The expanding air causes an increased backflow velocity in the casing which is required to lift the cuttings. The uphole velocity (UHV) of the flushing fluid needs to be larger than the "sinking velocity" of the cuttings (Kutzner 1996). To maintain an effective drill cutting evacuation the UHV should be between 900-1800 m/min. UHV in m/min is calculated as follows (Halco Rocktools 2021):

$$UHV = \frac{Q_{air} \times 1\,305\,096}{(D_h{}^2 - d_t{}^2)} \tag{1}$$

were:
Q_{air} is the air volume (m^3/min) passed by the hammer at a given air pressure.
D_h is the diameter of the borehole or inside of the casing (mm).
d_t is the diameter of the drill rod (mm).

Typical pressures when drilling in stiff/dense soils where the hammer is required is 8 to 15 bars with an air consumption (i.e. flow rate) varying between 2.5 to 30 m^3/min depending on the hammer size (Halco Rocktools 2021).

Water hammers require pressures between 50 to 180 bars with a flow rate of 50-150 L/min for a 2-inch hammer up to 1200 L/min for a 12-inch hammer (Wassara 2021). The pressure of the flushing media decreases significantly as it passes the hammer. The water pressure at the front of the drill bit is mainly governed by the height of the water column up to the top of the casing.

2.4 Eccentric vs concentric drill bit systems

There are two main drill bit systems for cased rotary percussive drilling: eccentric and concentric. The two systems are available for both top-hammers and DTH hammers and can be used in different ground conditions and applications. Figure 2-2 show photos of an eccentric (a) and concentric (b) drill bit.

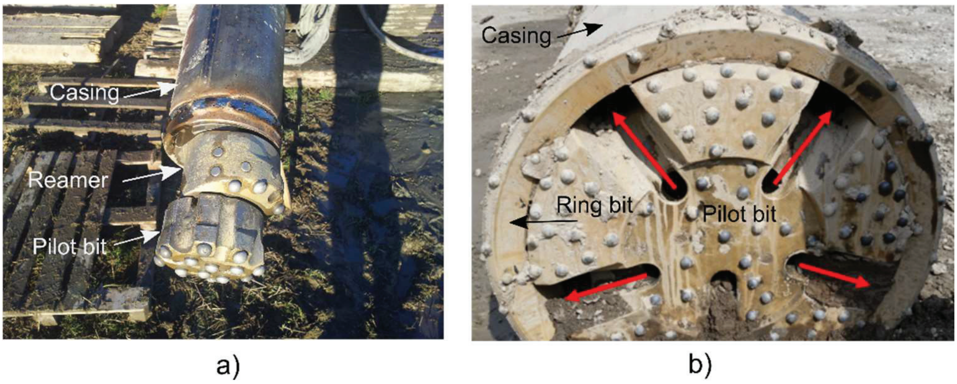


Figure 2-2. Pictures of: a) drill bit with eccentric reamer (Ø139 mm casing) and b) concentric pilot- and ring bit system (Ø711 mm casing). (photos by Einar John Lande).

The traditional eccentric drill bits known as the Overburden Drilling Eccentric (ODEX) are one of the most cost-effective and hence commonly used for overburden drilling. The eccentric systems can drill casings with diameters from 102 mm up to a maximum of 273 mm. This system consists of a concentric pilot bit in the front followed by an eccentric reamer with slightly larger diameter than the casing, as illustrated in Figure 2-1 a) and b) and depicted in Figure 2-2 a). During penetration, a guide device on the drill bit acts on a casing shoe that is welded to the bottom of the casing, thus pulling down the casing. A disadvantage with the eccentric system is that the reamer may cause "over-coring", i.e. a gap between the casing and the borehole wall. Since the pilot bit with its flushing ports are located approximately 10 cm in front of the reamer the risk of compressed flushing air escaping up through the gap increases which may result in excessive erosion and disturbance of the surrounding soil. Experiences also indicate that entering steeply inclined bedrock surfaces can be problematic due to the eccentric reamer causing deviations. This can further increase the risk of excessive soil erosion by the flushing fluid.

To mitigate some of the shortcomings of eccentric systems, concentric systems have been developed. The concentric system consists of a pilot bit in the center, a casing shoe that is welded to the casing, and a symmetrical ring bit, as illustrated in Figure 2-1 c). Figure 2-2 b) show a photo of a widely used concentric pilot- and ring bit system. The red arrows illustrate the direction of the flushing fluid (air or water) as it exits through ports in the pilot bit and returns up inside the casing. The ring bit interlocks with the pilot bit and a rotary joint between the casing shoe and the ring bit enables it to rotate freely from the casing during drilling. The ring bit drills a borehole slightly larger than the outer diameter of the casing, allowing the casing to advance. This may also cause some over-coring like the eccentric reamer. The face of the pilot bit is placed almost in line with the ring bit, which facilitates keeping the borehole in its desired alignment and in entering an inclined bedrock surface. Concentric systems can drill casings with diameters from 90 mm to 1016 mm.

During the last decade several manufacturers have developed drill bits aimed to reduce negative and possible detrimental impacts when drilling in sensitive ground conditions. The main concept with these concentric drill bits is to redirect the air flow at the front of the bit, to limit compressed air from evacuating into the ground, and creating unwanted cavities, hence reducing the risk of settlements. Figure 2-3 show examples of concentric drilling system developed particularly for overburden drilling in challenging and sensitive ground conditions.

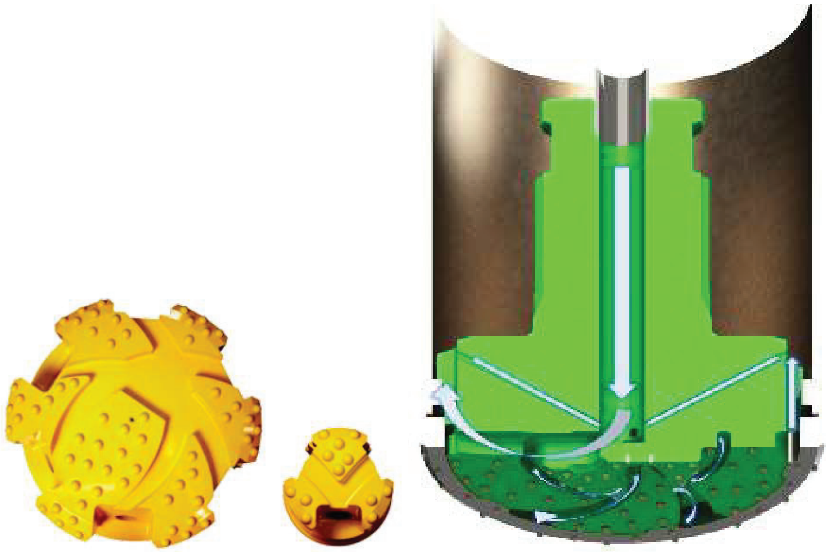


Figure 2-3. Examples of pilot drill bits designed to reduce risk of compressed air evacuating into the ground. Left: Elemex system by Atlas Copco. Right: Spiral Flush system by PPV Finland.

2.5 **Comparison of drilling methods**

An overview of the general pro's and con's with different drilling methods with respect to reduce negative/detrimental installation effects were made as part of the *Limiting Damage* project. This is shown in Table 2-1 for the four rotary duplex methods described above.

Table 2-1. Overview of rotary duplex drilling methods.

Method/ system	**Pro's**	**Con's**
Rotary duplex	No air flushing required. Stabilizing water inside casing. Reduced risk of excessive erosion and soil disturbance.	Require a percussive hammer in stiff/hard soil or rock. Limited diameters with standard rigs.
Rotary percussive duplex – top hammer	No air flushing required. Stabilizing water inside casing. Reduced risk of excessive erosion and soil disturbance.	Limited depth, 30-50 m. Casing diameter 90-140 mm. Drill hole deviations due to percussion on top of drill rod. Deviation in block/boulders and inclined bedrock.
DTH air hammer	Equal percussive energy with depth. Large depths (50-100 m). Large diameters and selection of drill bits. Reduced drill hole deviations.	Increased risk of compressed air evacuating into ground and soil erosion. No stabilizing water in the casing. Eccentric drill bits can cause deviation when entering inclined bedrock.
DTH water hammer	No air flushing required. Stabilizing water inside casing. Reduced risk of excessive erosion and soil disturbance.	Requires large amounts of clean water (200-1000 L/min). Limited casing diameter of 500 mm. Frost and winter drilling. Heavy drill rod.
Double head duplex	Casing and drill bit operated "free" from each other. Drill bit can be retracted up in the casing.	Requires threaded casings. More expensive drill rigs

3 Literature review

3.1 General

Based on work in the *Limiting Damage* project it seems that negative effects from overburden drilling on the surrounding ground is well known within the building and construction industry, and particularly among most contractors. However, the problem has not been systematically addressed and scientifically studied. This section presents a short summary of relevant literature related to drilling.

3.2 Kempfert and Gebreselassie (1999)

Kempfert and Gebreselassie (1999) present a case study from a deep excavation in soft lacustrine (silty) clay in the city of Constance, Germany. The excavation was 5.3 to 7 m deep and supported by walls consisting of soldier piles (HEB 600 steel) with timber or concrete piles as support in between. A part of the wall was made of 0.9 m diameter tangent bored concrete piles. Tieback soil anchors (120 mm drill bit) with center distances of 0.9 m were installed by rotary flush drilling through the soft clay followed by rotary percussive drilling in the deeper gravel layer. Water flushing was used to enhance the drilling. The tendons were grouted with cement mortar in the bonded length before the unbonded part of the anchor was filled with cement-bentonite slurry while pulling of the temporary casing.

Excessive settlements and damage to an adjacent building occurred during the anchor installation. A maximum ground settlement of 56 mm was recorded on the building only a few meters from the wall. Assessment of the monitoring indicated that 60-70% of the total settlements were related to the anchoring works, while only 10-35% were related to the excavation and displacement of the retaining structure. Local failure of boreholes supported by the fresh cement-bentonite slurry and vibration from percussion drilling and pulling of casings were considered the main reasons for the excessive settlements. Mitigating measures were taken which included only using rotary drilling and water flushing in all layers combined with installing permanent plastic casings into the soft soil by pushing and partly rotating before the drilling. The following measurements showed insignificant settlements implying that the changed procedures were successful.

3.3 Konstantakos et al. (2004)

Konstantakos et al. (2004) reported a case study from an up to 23 m deep excavation for the Dana Farber research tower in Boston, Massachusetts. The ground consisted of up to 5 m surficial fill over 10-17 m thick stiff to very stiff clay. Further down there was a sand and silt deposit above 0.3-3 m dense glacier till and conglomerate bedrock. The excavation was supported by a permanent 0.9 m thick diaphragm wall embedded in bedrock and braced by four to six levels of pre-stressed anchors drilled into bedrock.

Even though the support system was successful in controlling lateral wall deformations to less than ±15 mm, ground surface settlements up to 65 mm were recorded on two sides of the excavation. The excessive settlements were explained by local loss of soil mass creating cavities around the anchors during drilling through sand and silt layers at 15-18 m depth. The hypothesis was confirmed by FE-analyses, where loss of soil mass was considered by including volumetric strains in clusters of soil elements around the tieback anchors, corresponding to 0.36-0.50 m^3/lin-m of the supported diaphragm wall. Results from the analyses agreed well with monitoring results. Specific details on drilling method and execution are however not presented.

3.4 **Kullingsjö (2007)**

Kullingsjö (2007) investigated the effects of deep excavations in soft clay on the immediate surroundings. This PhD study was related to the large infrastructure project *Göta Tunnel* in the city center of Gothenburg, Sweden. One of the study areas, *Section 1/470 South*, involved a deep excavation of about 10 m supported by a stiff sheet-pile wall (SPW) and three levels of tieback strand anchors installed in permanent casings drilled into bedrock. The ground consisted of a top layer of 2-3 m fill material over a normally consolidated homogenous marine clay deposit and a 2-4 m thick layer of sandy silt/silty sand above bedrock. The depth to bedrock was about 20 m at Section 1/470.

Figure 3-1 show the plan view of instrumentation installed in Section 1/470 South (a) and the different levels of excavation stages and anchoring. Monitoring results from inclinometers on the sheet pile wall and extensometers installed behind the excavation showed that the ground settlements by far exceeded lateral deflection of the wall. Figure 3-2 show results from the extensometers located behind the SPW. At completion of the excavation to +0.8 masl, the maximum measured settlement was about 50 mm at 10 m distance from the wall. The settlements increased to 70 mm about one year after the excavation. This corresponds to 0.7% of the excavation depth. Approximately 50 mm of the settlements occurred in the sandy/silty layer above bedrock, caused by excessive erosion of sand material due to flushing with air and water during rotary percussive duplex drilling with DTH air hammer.

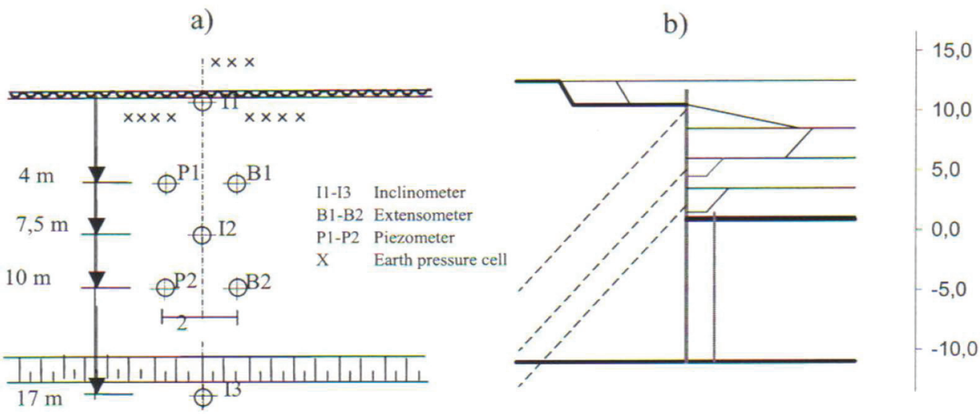


Figure 3-1. a) overview of instrumentation at Section 1/470 South, and b) cross section with excavation levels and tieback anchor levels (from Kullingsjö 2007).

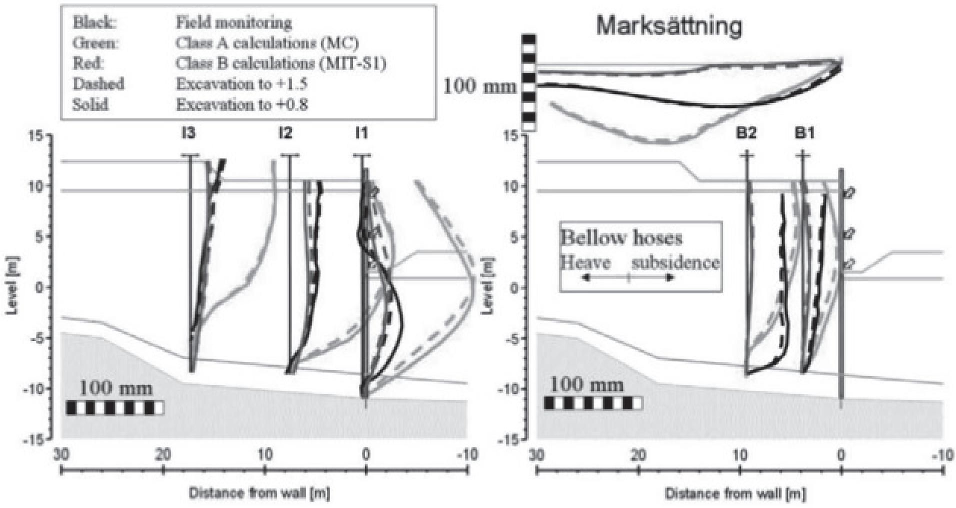


Figure 3-2. Cross section of the deep excavation in section 1/470 South for the Göta tunnel project. Results from extensometers installed at different distances from the SPW between time for casting concrete slab to "long-term" conditions (from Kullingsjö 2007).

3.5 Lande (2009)

Lande (2009) carried out Finite Element back-analysis of three case records of deep-supported excavations in soft clays where unexpectedly large ground settlements occurred behind the wall. The study included the *Göta tunnel* project reported by Kullingsjö (2007). The aim was to investigate if simulated local soil volume loss around tieback anchors could explain the excessive settlements observed in extensometers outside the supporting wall. The same approach as used by Konstantakos (2004) by applying volumetric strains in soil clusters representing the anchors in moraine/sand was adopted. The analysis generally showed good match with the measurements of horizontal wall displacements and ground surface settlements. To obtain a reasonable agreement between the field measurements and the back-analysis, a total soil volume loss of 0.4 m^3/lin. meter of the support wall was necessary. For the case in Section 1/470 South, that corresponded to approx. the same volume loss in m^3 per anchor.

3.6 Rønning (2011)

Rønning (2011) presents results from test drilling of two piles for a bored steel-pile wall (610 mm diameter) as support for a deep excavation in Trondheim, Norway. The ground consists of highly sensitive (quick) medium stiff marine clay and a thin layer of glacial till (moraine) overlying bedrock. A monitoring program including pore pressure sensors (piezometers) at different depths and distances from the piles, total pressure cells at different depths outside the piles, plus an inclinometer tube about 1 m from the piles were used to assess the installation effects and drilling procedure.

Table 3-1 show an overview of the piezometers (PZ) and the maximum pore pressure changes, ΔU_{max}, registered during the drilling. Figure 3-3 presents measured pore pressures against time for piezometers approx. 0.5 m from the two piles. Rotary duplex drilling using water flushing through the clay caused minor excess pore pressures. The drilling was carried out with a penetration rate between 0.5-1.0 m/min. When the till layer was reached and the air driven DTH hammer was activated the pressure temporarily dropped in all sensors. The total pressure cells on the piles all showed increased pressure during drilling in the clay. The cells at the pile tip typically showed pressures which corresponded to about 140-200% of the hydrostatic pore pressure. A maximum pressure of 310 kPa was registered at about 18 m drilling depth. Like the piezometers, the pressure dropped immediately to about 70% of the hydrostatic pressure when the air hammer was activated to drill further through the till and into bedrock. The pressure increased again when drilling was completed, and after some days it stabilized at a pressure equal to the theoretical effective horizontal earth pressure plus pore pressure.

Measurements on the inclinometer indicate that drilling of the first pile caused soil displacements, i.e. the soil was pushed away, in the upper 5-6 m depth (approx. 20-50 mm). Above bedrock the soil seemed to move towards the pile indicating some loss of soil mass around the pile. Installation of the second pile seemed to cause displacements towards the first pile which indicated re-consolidation of remolded clay.

Table 3-1. Overview piezometers and maximum pore pressure changes (ΔU_{max}) measured during test drilling of pile wall (from Rønning 2011).

Sensor	Depth (m)	Distance (m)	ΔU_{max} (kPa)
PZ 3.1	6	0.5	+23
PZ 3.2	13	0.5	+22
PZ 3.3	18.3	0.5	+15 / -10
PZ 2.1	6	7	-
PZ 2.2	19	7	+4

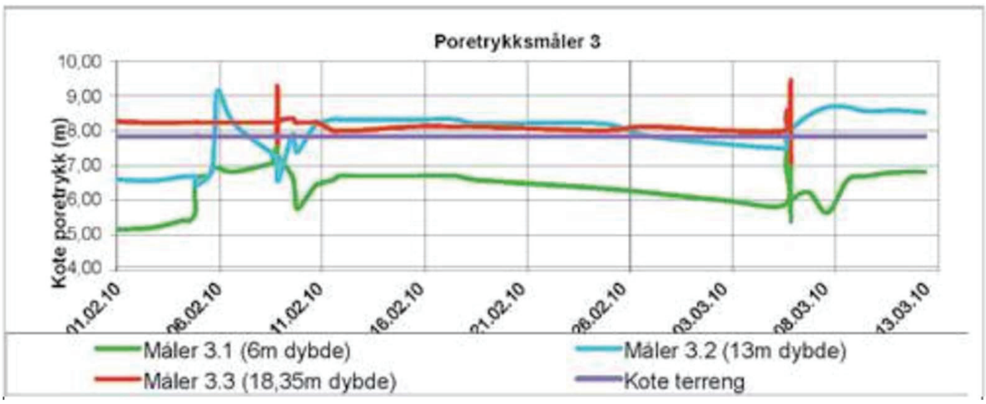


Figure 3-3. Measured pore pressures against time during test drilling for pile wall (from Rønning, 2011). Piezometers installed at approx. 0.5 m distance from the pile wall at 6, 13, and 18.35 m soil depths respectively (from Rønning 2011).

Based on the tests the project decided to carry out the pile wall installation by first vibrating the open piles through the sensitive clay followed by rotary percussive duplex drilling through the till and min. 1.2 m into bedrock using rotary percussive duplex method. Insignificant temporary pore pressure changes were registered during the production drilling. Rønning (2011) concludes that it is possible to drill through sensitive and quick clay causing only a limited mechanically remolded zone close to the pile wall. During excavation in the building pit, he observed that the remolded clay closest to the pile wall was consolidating in layers, with clay particles still suspended in water one year after drilling.

3.7 **Bredenberg et al. (2014)**

Bredenberg et.al (2014) describes a case record from Stockholm, Sweden. Casings (outer diameter = 168 mm) for steel core piles were installed by rotary percussive duplex drilling and DTH air hammer. A concentric drill bit designed to reduce the risk of causing cavities and excessive soil loss around the drill bit due to high pressure air flushing was used. Monitoring showed that drilling with the concentric drill bit caused about 10-15 mm settlements on existing basement floors, which was about 30% compared to an adjacent construction project with similar ground conditions where the pile drilling was carried out using the conventional eccentric drill bit.

3.8 **Degago et al. (2015)**

The paper by Degago et al. (2015) presents a case study where steel tube piles were installed by drilling in a natural slope with quick clay as part of four pile group foundations for the Skaudal Bridge in Rissa municipality in Norway. Each pile group consisted of two piles with 813- or 914-mm diameter. The soil conditions were characterized by about 2 m of silty/sandy/gravelly soils over 7 to 16 m of medium stiff clay (highly sensitive) with high degree of over-consolidation. Pore pressure measurements showed a clear under-hydrostatic condition in the top of the natural slope compared to the upper ground water level observed at about 0.8 m soil depth. Special attention was therefore paid to the pile drilling to avoid excess pore pressures that could initiate a large quick clay slide.

The drilling contractor suggested to use a recently developed DTH drilling system with a pilot drill bit designed to avoid compressed air evacuating uncontrolled into the surrounding ground (see Figure 3-4). The project owner (Norwegian Public Road Authorities, NPRA) decided to assess the suitability of the drilling system by monitoring of the initial drilling in pile group 4 where there were no slope stability issues.

Figure 3-4. Concentric pilot- and ring bit system used to drill pile casings for the Skaudal Bridge. (Photo by Samson Degago and Ørjan Edvardsen, NPRA)

Figure 3-5 presents a layout of pile group 4 including piezometers installed adjacent to the pile group. Piezometers were installed in two positions at 3, 4 and 6 m soil depths respectively. Measurements showed that the drilling of piles P44, P42 and P41 caused maximum excess pore pressures of 75, and 105 kPa respectively which exceeded the acceptable limits set for drilling in the slope with quick clay. Figure 3-6 show the pore pressures measured during drilling of pile P41 located closest to the piezometers. The large excess pressures were related to a combination of unsuccessful transport of drill cuttings up from the borehole resulting in soil displacements and uncontrolled blowouts of compressed air (i.e. pneumatic fracturing). The monitoring data shows that most of the excess pore pressures seemed to dissipate within a few hours after the drilling was completed.

Before the remaining pile drilling could carry on the NPRA required changes in the drilling procedures to primarily use of water flushing instead of compressed air. This was done to improve control of the backflow of drill cuttings and to avoid soil displacements. A test pile was drilled to verify the changed procedures. Results showed minor excess pore pressures up to a maximum of 9 kPa at 4.7 m soil depth and 1.5 m distance from the pile center. The following production drilling in pile group 1 caused temporary excess pore pressures up to 28 kPa due to some minor issues with the backflow of drill cuttings. Only minor impacts were observed when drilling in pile group 2 in the slope with quick clays.

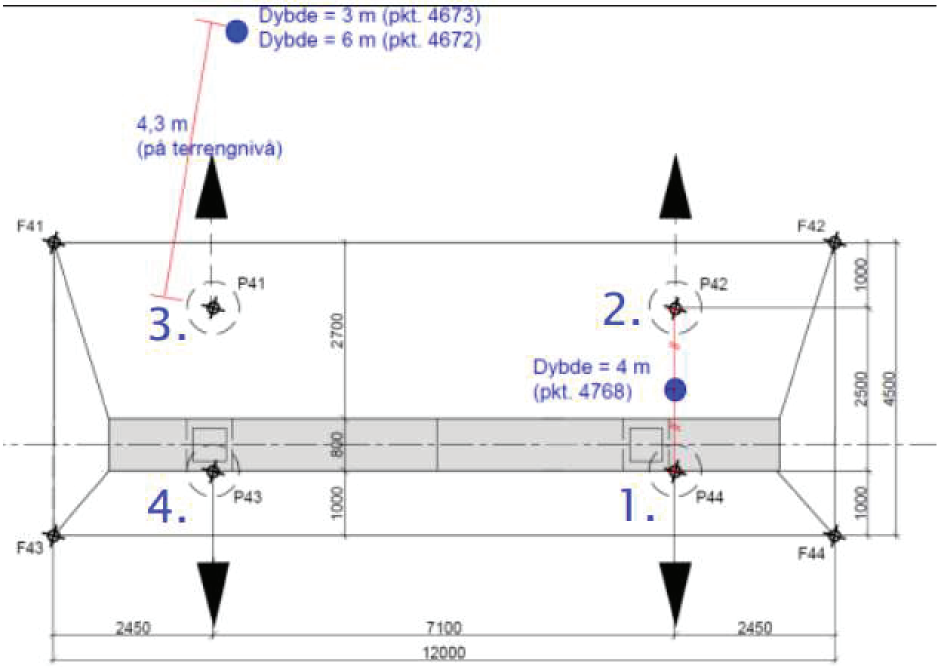


Figure 3-5. Layout of pile group foundation 4 with positions of the piles (Ø813 mm) and the piezometers (blue dots). Units in mm. (from Degago et al. 2015).

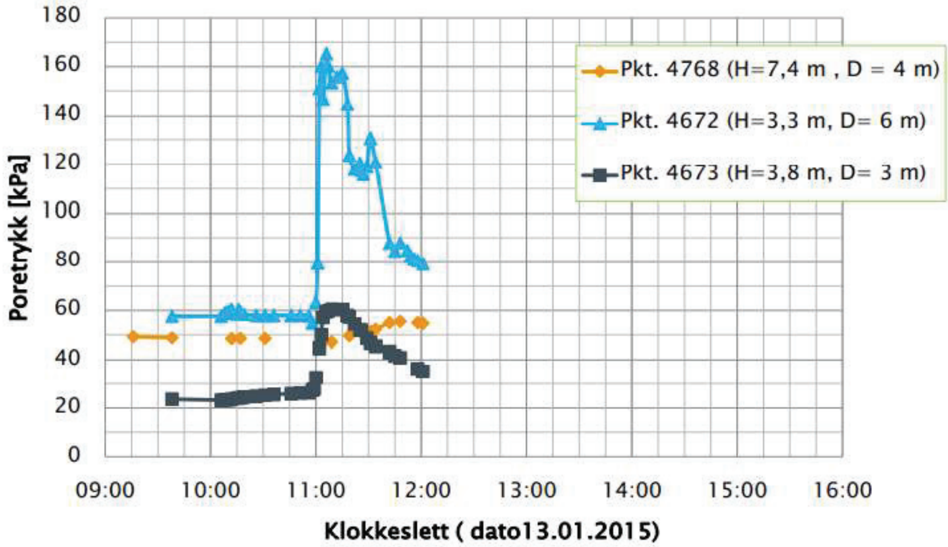


Figure 3-6. Measured pore pressures during drilling of pile P41 in pile group 4 (from Degago et al. 2015).

3.9 Ahlund and Ögren (2016)

Ahlund and Ögren (2016) presented results from a field study comparing the installation effects from overburden drilling with air and water driven DTH hammers. Two steel tube test piles were installed with each drilling method at separate test groups located about 15 m from each other. The piles were placed 2 m apart in each test group. The test site consisted of 8-11 m of marine clay over an approximately 4 m layer of silt and granular soils above bedrock. The impacts from drilling were at each test group monitored by 1 settlement plate at ground level, 1 settlement anchor at 8 m depth and 1 pore pressure sensor at 5 m soil depth. The pore pressure sensor was installed about 1 m from the piles and the settlement points about 1.5 and 2 m from the piles.

The measurements showed that drilling caused maximum accumulated excess pore pressures of approximately 14 kPa for the air hammer (Figure 3-7) and 4 kPa for the water hammer (Figure 3-8). Most of the excess pressures dissipated within the first 24 hours after the drilling was completed. In test group 1 drilling with the air hammer resulted in immediate settlements of 10 mm at 8 m soil depth while a heave of 11 mm was registered on the ground surface plate adjacent to the piles. The air drilling also caused 2 mm settlement of the settlement anchor and 4 mm on the plate in test group 2 for water hammer drilling. Drilling with the water hammer caused immediate heave of 2 and 6 mm of the settlement anchor and plate respectively. About two weeks after the tests the final settlements became 20 mm in the settlement screw and 2 mm at ground surface for test group 1 while 5- and 7-mm settlements were registered at test group 2.

Measurements showed that both drilling methods produced less drill cuttings than the theoretical pile volume installed in the ground; about 50% and 30% for the air and water hammer respectively. This indicates that the drilling caused some soil displacements which could explain the ground heave. The settlements measured at 8 m soil depth for test group 1 further indicate that the air flushing has led to notable erosion and soil volume loss.

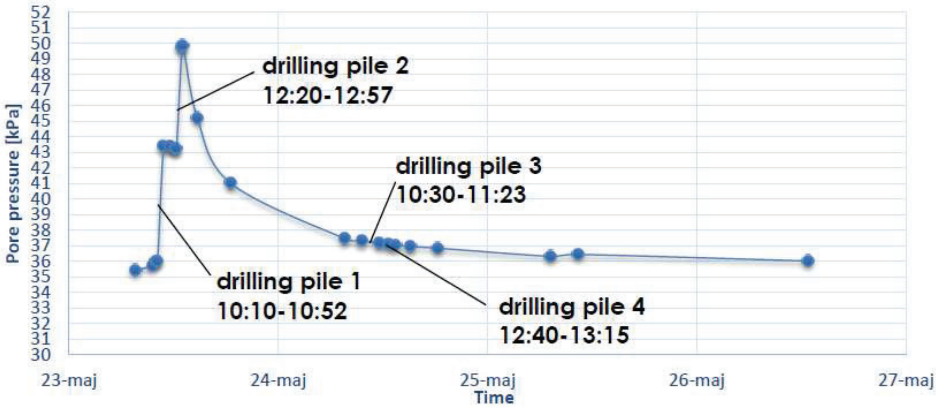


Figure 3-7. Measured pore pressure in clay (5 m depth) against time in test group 1. Pile 1 and 2 installed by drilling with DTH air hammer (from Ahlund and Ögren 2016).

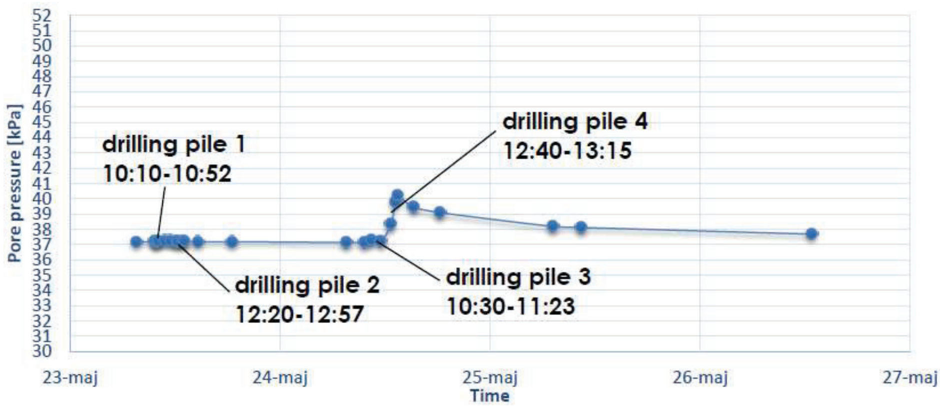


Figure 3-8. Measured pore pressure in clay (5 m depth) against time in test group 2. Pile 3 and 4 installed by drilling with DTH water hammer (from Ahlund and Ögren 2016).

3.10 **Asplind (2017)**

Asplind (2017) presents a case study where a 16 m deep RD-pile wall (406 mm diameter) was installed by drilling with a water driven DTH hammer through fill material mainly consisting of gravelly sand over a thick deposit of sand and gravel (glacial esker). By measuring the pore-water pressure and the settlements during the installation of the pile wall the magnitude and extent of the installation effects induced by the drilling was investigated.

The results indicate settlements close to the installed piles in both materials, larger in the esker material. Settlement anchors installed about 4 pile diameters (~1.6 m) from the wall showed that the drilling resulted in 25 mm ground settlement at 13 m depth and 15 mm at 5 m depth. Visual observations showed that the flushing water typically did not return up from the casings as intended until typically 3-6 m of drilling were carried out. Instead, the water "escaped" into the fill material and often came up through the surrounding ground or neighboring casing(s) adjacent to the casing being drilled. Excess pore-water pressures up to about 25 kPa were also registered in electrical piezometers at 5 m depths. Figure 3-9 show measured pore pressures against distance in pile diameter (D) during drilling of the pile wall. The largest increases of the pore pressure are seen when the hammer flushes water out into the formation and not during drilling. The ground conditions with granular and permeable material (erodible soil) in the entire pile length and the observed water escaping into the ground could indicate that the water flushing partly fluidized and eroded soil.

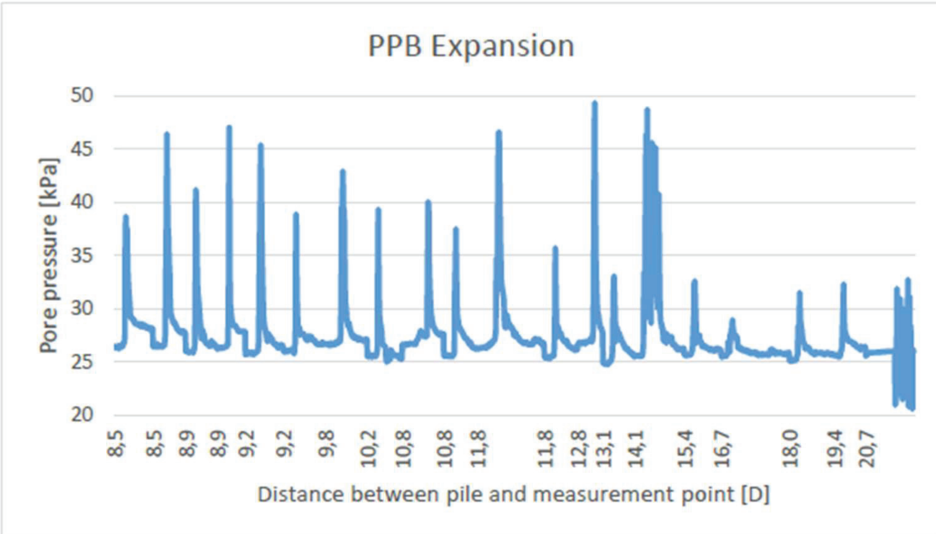


Figure 3-9. Measured pore pressures in piezometer B at 5 m soil depth against distance related to the pile diameter D (from Asplind 2016).

3.11 **Sandene et al. (2021)**

The paper by Sandene et al. (2021) presents a case study of a 9 m deep excavation in soft low sensitive clay for the new National Museum in central Oslo, Norway. The excavation was supported by sheet pile walls (SPW) driven to bedrock and up to 5 levels of post-tensioned tieback anchors drilled into bedrock. Casings for the strand anchors were drilled by rotary percussive eccentric duplex drilling using DTH air hammer (Figure 2-1 and Figure 2-2 a). The sheet pile toe was horizontally fixed with steel dowels installed into bedrock before excavation. The dowels were installed through casings welded to the SPW.

The ground conditions are described as typical of the lower areas in central Oslo. Historically, the area has been reclaimed for building purposes. Hence, the upper 2-2.5 m of soil is characterized as urban fill material including sand, gravel, rocks and bricks, concrete etc. from demolished buildings. Below the fill material lies a 2-4 m slightly weathered marine clay over normally consolidated (NC) clay to bedrock. The depth to bedrock varies from about 2 m to over 20 m in the construction area. In deeper areas and in local depressions in the bedrock surface, moraine deposits are found between the clay and bedrock. A moraine deposit including layers of silty, sandy material was observed from 8 to 10 m depth in an area north-west of the building pit.

Figure 3-10 shows the layout of the building pit with the neighboring building (Nobel Peace Center) and the adjacent main roads (Dokkveien, E18 and Dronning Mauds gate). In two specific areas, illustrated by cross sections A-A and B-B, relatively large excess ground settlements occurred outside the building pit during the excavation and anchor drilling works. Figure 3-11 shows the plan view of the area around section B-B including the locations of geodetic surveying points along the E18 road embankment.

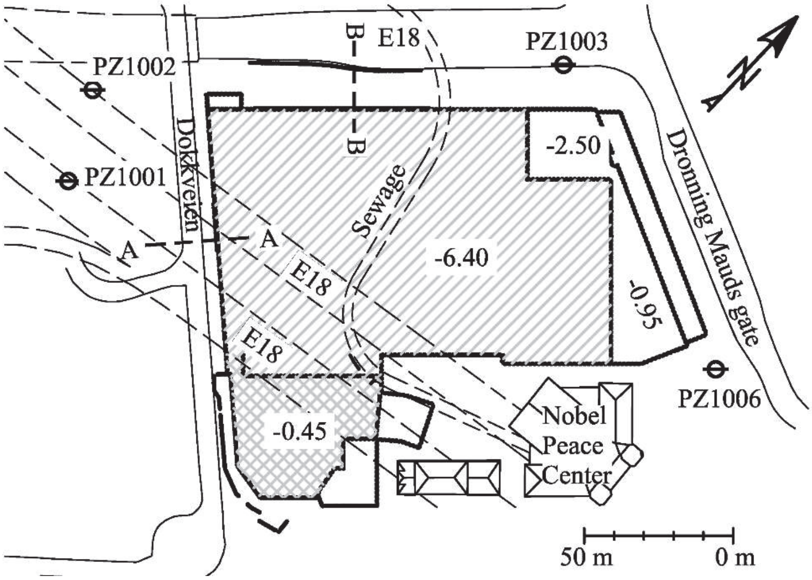


Figure 3-10. Layout of building pit, neighboring structures, roads and cross sections A-A and B-B. The E18 road and a sewage tunnel, both marked with dotted lines, cross in rock tunnels under the building pit (from Sandene et al. 2021).

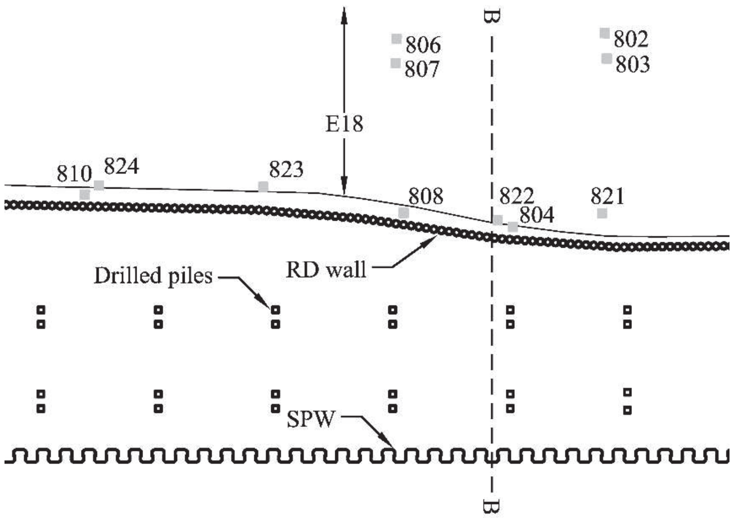


Figure 3-11. Plan view of the area around section B-B adjacent to the E18 road (from Sandene et al. 2021).

Figure 3-12 present ground settlements measured on surveying points along the E18 road embankment close to section B-B during the excavation works for the main building pit. Results from pore pressure measurements near the bedrock surface at PZ1002 and PZ1003 are included. Progress of anchor drilling in four levels (R1 to R4) and foundation piles in the area between the SPW and the road embankment are marked.

Sandene et al. describes that the vertical settlements at section B-B are comparable to the trend measured for section A-A during drilling of the first two rows of anchors. During drilling of rows R3 and R4 there is however a sharp increase in settlement rates, which decreases after the anchor drilling is completed but continues at a constant rate of nearly 7-8 mm/month for as long as the measurement points were available. The total settlements after excavation to the final level in the building pit were around 10-11 cm at points 804 and 808. The significant increase in settlement rates during drilling of anchor R3 and R4 was most likely caused by disturbance and erosion in the silty and sandy soils due to extensive air flushing. Observations during drilling revealed that compressed air from the drill bit escaped through the ground and up neighboring casings for both anchors and sheet pile dowels, as well as underneath the foot of the SPW on bedrock. Monitoring results from an inclinometer at section B-B showed that the SPW moved outwards with about 55-60 mm during the final phases of the excavation. FE-analyses were carried out where loss of soil mass was simulated by applying negative volumetric strains in defined soil clusters along the ground anchors. The analyses indicate that a volume loss of 0.4-0.5 m^3/anchor is required to replicate the measured settlements and displacements in the SPW. The back-analysis is comparable to the studies by Konstantakos (2004) and Lande (2009).

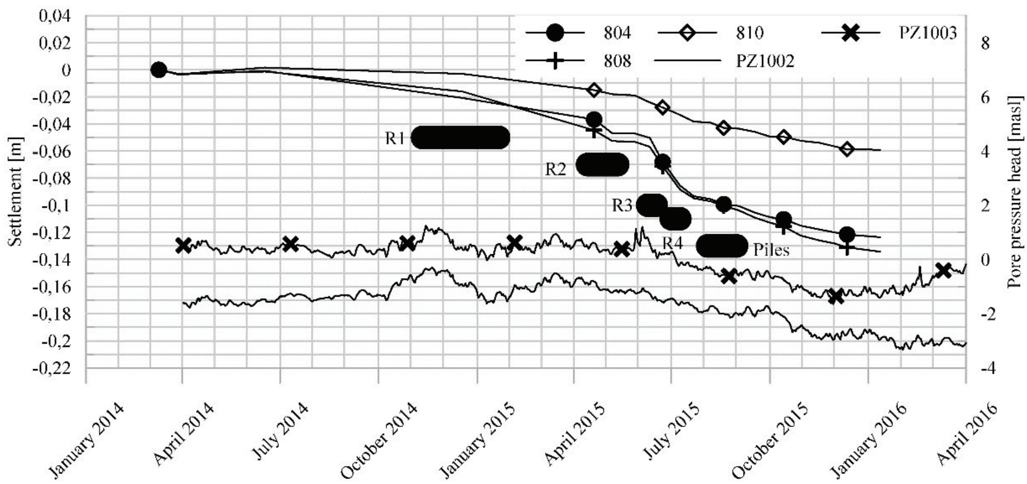


Figure 3-12. Measured ground settlements on surveying points close to section B-B during ground works. Results from piezometers installed to bedrock in the area are also included (from Sandene et al. 2021).

3.12 **Gloppestad (2021)**

This conference paper by Gloppestad (2021) presents a case study from construction of the first two buildings in a large residential development project at the Nyhavna Øvre site in Trondheim, Norway. The paper focus on assessments of the design choices and monitoring of influence on the surroundings during drilling and installation of steel core piles.

The ground conditions at the site are characterized by a top layer of 4-5 m with fill material over a soft to medium stiff marine clay and a thin layer of granular and more permeable material over bedrock. The clay is defined as quick with a high sensitivity in parts of the area. Results from piezometers showed artesian pore-water pressures at bedrock level at 20-35 m depth, corresponding to a total head 3-4 m above ground surface level.

The pile drilling and installation procedures were carefully addressed to limit the risk of large excess pore pressures in the clay deposit that could affect the slope stability. In addition, the potential drainage effects along the pile casings and the risk of long-term consolidation settlements at the site and nearby areas were considered. A total of 194 and 229 piles were installed for the construction phases 1 and 2 respectively. Systematic injection grouting was carried out using a packer at the bottom of the pile casings after a socket in bedrock was drilled. When the cement grout had cured for minimum 1 day, the borehole was drilled again, and leakage testing performed. About 20% of the pile casings in both construction phases needed two additional grouting attempts at the pile tip, and an additional third round of grouting for approx. 2-3% of the casings.

Test drilling was performed near piezometers installed at the construction site. Excess pore pressures of approximately 60 kPa were registered in the clay layer during drilling of a casing 1,3 m from the piezometer. The pressure dissipated and stabilized at a reduced level of 30 kPa the next days but did not reduce further until later when there were challenges related to ground water leakages observed from 2-6 piles daily at the site. Towards bedrock, where most of the piezometers adjacent to the site was placed, drastic pore pressure reductions were observed. However, the pore pressure was also rapidly reestablished during periods with no pile installation activities.

During the second construction phase (building no. 2) the pore pressure reductions at bedrock level was remedied with water injection into wells in bedrock during the most critical period. These measures were introduced after a more drastic pore pressure drop occurred compared to what the experience from phase 1 showed. Water injection gave a positive impact on the pore pressures. The project did not include measurement of ground settlements, but settlements of nearby buildings were sporadically measured over a one-year period.

4 Effects of overburden drilling

4.1 Introduction

Like other specialty foundation works (e.g. pile driving, deep soil mixing, jet-grouting and vibro replacement), drilling for ground anchors and foundation piles will have effects on the surrounding ground. The Micropile manual by FHWA (2005) state that: "*The drilling method selected by the contractor should avoid causing an unacceptable level of disturbance to the site and its facilities, while providing for the installation of a micropile that supports the required capacities in the most cost-effective manner*." It is further mentioned that intense flushing to increase drilling rates and removal of cuttings should be approached with caution to avoid the risk of creating voids and ground settlements, or uncontrolled hydro-fracturing of the ground which could lead to ground heave.

Results from the case studies investigated in the *Limiting Damage* project (Langford et al. 2015) and the literature review in section 3 indicates that the installation effects from drilling can be divided into three categories based on how they can induce ground settlements:

A. Pore pressure reduction and consolidation settlements in soft clay (i.e. long-term effect); caused by ground water leakage/drainage up along inside and/or outside of the casing tube (Langford et al. 2016).
B. Local disturbance and remolding of the clay structure during drilling; causing local excess pore pressures followed by re-consolidation (volume change) of the clay surrounding the casing.
C. Loss of soil mass (i.e. volume) during drilling likely caused by:
    - "Suction" at the front of the drill bit, which can lead to local erosion and cavities around the casing tube (e.g. Konstantakos 2004; Kullingsjø 2007; Bredenberg 2014; Sandene et al. 2021).
    - "Over-coring" around casing tube from eccentric reamer or ring bit.
    - Uncontrolled blowouts and erosion of soil along the outside of the casing/drill string due to high pressure flushing with air or water, or natural ground water flow.
    - Collapse of borehole wall (local ground failure) for instance if the casings are removed (e.g. Kempfert & Gebresellassie 1999).
    - Hydraulic ground failure

The field and laboratory studies presented in the following sections focus on the installation effects B and C. Secondary effects related to pore pressure reductions and consolidation settlements (effect A) are not addressed in any detail in this thesis.

4.2 Pore pressure reductions and consolidation settlements

Drainage of ground water and reduction of pore pressures in soft clays above bedrock are a well-known cause of consolidation settlements when performing deep-supported excavations below ground water level, especially when excavating to bedrock. Langford et al. (2016) presented monitoring data from several building projects showing that there often is a substantial reduction of pore pressure levels at bedrock, and that the reduction can be observed hundreds of meters from the excavation.

Tieback anchors and piles that are drilled through clay, glacial till (moraine) and into bedrock can form a leakage path for ground water if they are installed from below the ground water level or in artesian conditions, as illustrated in Figure 1-1. The leakage may be temporary on the inside of the casing before grouting of the anchors or piles is performed. However, there is also a risk of a longer lasting leakage up along the outside of the casing if the drilling procedure has created a gap between casing and surrounding soil. The number of anchors and piles are often numerous; hence the effects of leakage can be substantial even though it is not possible to observe with the eye. The effects depend on the hydrogeological properties, as well as the extent and time for which an excavation is open.

4.3 **Disturbance and remolding of clay structure**

Drilling for anchors and piles in soft clay will cause some unavoidable local remolding and distortion of the clay structure around the casing tube. This will occur due to mechanical disturbance by the rotating drill bit. If the penetration rate (V_{pen}) is too high, the soil in front of the drill bit will partly be displaced (i.e. pushed away) and not transported up through the casing as intended. This may cause considerable undrained soil displacements, comparable to those from driven closed-ended piles (Karlsrud and Haugen, 1984; Edstam and Kullingsjö, 2010; Sagaseta et al. 1997). Soil displacing piles in undrained clay will cause a remolded and plasticized zone adjacent to the pile shaft which can be calculated according to the formula by Vesic (1972):

$$r_{pl} = r_0 \times \sqrt{\frac{G}{c_u}} = r_0 \times \sqrt{\frac{E}{2 \times c_u(1+\nu)}} \tag{2}$$

were:

r_{pl} is the plasticized radius from the pile center,
r_0 is the outer radius of the pile,
G is the undrained shear modulus up to failure (G_{50}),
c_u is the undrained shear strength,
E is the deformation modulus,
ν is the Poisson's ratio (undrained conditions = 0.5)

The displacements should theoretically lead to a certain degree of ground heave presuming that remolded clay is not "squeezed" up along the casing shaft to the ground surface and that no volume changes occur. Additional remolding and disturbance of the surrounding clay may occur due to flushing with compressed air during drilling, which may cause uncontrolled pneumatic fracture (i.e. blowouts) into the surrounding soil if the backflow in the drill bit or inside the casing are blocked (Degago et al. 2015). In sensitive clays, this can result in local ground failure and excessive settlements. However, problems related to uncontrolled blowouts can be reduced significantly by using water flushing.

4.3.1 *Re-consolidation of remolded clay*

The soil displacements and remolding of clay as described above will cause a rapid increase in excess pore pressures (Δu) around the casing. The following dissipation process may take several months, which is comparable to experiences from pile driving described by Karlsrud (2014). As the excess pore pressure dissipates, the mean effective stress changes and re-consolidation takes place around the casing (Lehane and Jardine 1994). This process causes volume reduction which can generate excessive ground settlements. The amount of volume reduction depends on factors such as the degree of shear strains, sensitivity, and water conditions in the clay plus the effective soil stress.

Karlsrud and Haugen (1984) presented results from a field test with jacked closed-ended piles (Ø153 mm) in clay. Laboratory investigations on samples of re-consolidated clay around the piles showed that the degree of remolded clay decreased quickly with increasing radial distance from the pile shaft as can be seen in Figure 4-1. The clay was completely remolded within a radial distance of about 2 cm from the pile wall (zone "A"). In this zone the undrained shear strength ($s_u = c_u$) increased to almost twice the initial in-situ strength while the water content (w) reduced by 25-30%. The re-consolidation resulted in a net soil volume reduction, ε_v of 16-17%. The degree of disturbance (shear strains, γ_{rz}) decreased rather evenly to a radial distance of 10-12 cm from the pile wall. The results coincided well with previous oedometer tests on remolded quick clay from a site in Oslo, Norway that showed 10-15% volume reductions (NGI 1964).

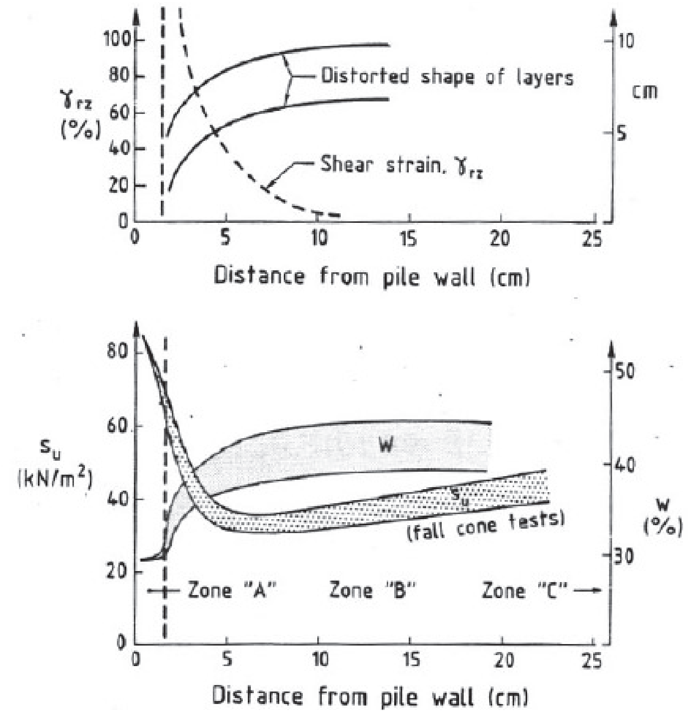


Figure 4-1. Measured shear strains (γ_{rz}), water content (w) and undrained shear strength (s_u = c_u) in clay against radial distance from the pile wall from pile tests at Haga, Norway (from NGI 1984).

Based on a suggestion by Karlsrud (personal communication), Borchtchev (2015) carried out laboratory tests on clay samples subjected to different degrees of shears strains by extrusion as illustrated in Figure 4-2. The aim was to investigate changes in strength and stiffness parameters as well as the potential for volume changes due to re-consolidation. Tests were carried out on undisturbed, partly disturbed (shear strain, γ_s equal to 18, 66 and 117% respectively) and completely remolded clay. Samples (3-5 m depth) of soft, low-plastic non-sensitive clays with over-consolidation ratio (OCR) between 2.5-3 and water content between 25-40% from Trondheim, Norway was investigated. Additional tests on medium stiff clay from 6-9 m depth in Stjørdal, Norway were carried out. This clay had a water content of 30-35% and an OCR of approximately 1.5 at 14 m depth.

The results showed some of the same general trends as the above pile tests (Karlsrud and Haugen, 1984). Highly disturbed clay (γ_s = 117%) gets a significant increase of 2-3.6 times the initial strength after re-consolidation for different stresses (42-200 kPa) in an oedometer. Considerable reductions in soil volume (8.5-16%) and water content (5.3-9.4%) were also measured. Tests on moderately disturbed clay (γ_s = 18 and 66%) showed similar reductions in water content and volume as the highly disturbed samples for stresses higher than 100 kPa.

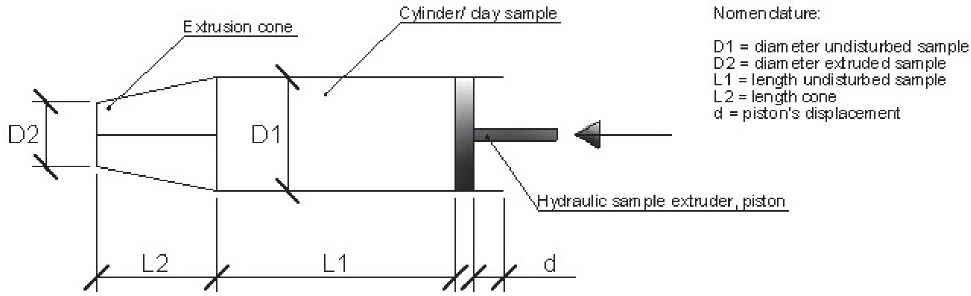


Figure 4-2. Schematic set-up for applying large shear strains on clay samples by extrusion (from Borchtchev 2015).

4.4 **Loss of soil mass during drilling**

The previous studies referred to in the literature review (Section 3) clearly indicate that overburden drilling for anchors and piles may create local cavities (voids) around the casing due to considerable loss of soil mass exceeding the theoretical volume of the casing tube. The cavities cause excessive settlements in the surrounding ground, which typically occur shortly after drilling is undertaken. There are several factors or a combination of them, which can cause loss of soil mass:

I: When drilling is carried out with air flushing below the ground water level it may cause an air-lift pump effect. The principals of such a pump are schematically illustrated in Figure 4-3 (a): Compressed air is injected through a supply line (e.g. drill rod) to the bottom of a discharge riser tube submerged in water (e.g. casing). When the air exits from the outlet of the supply line (e.g. drill bit) it reduces the density of the air-water mixture in the riser tube, thus creating a lower pressure compared to the water pressure outside the riser tube. This causes a flow upwards in the discharge tube by the surrounding water with higher density.

Figure 4-3 (b) illustrates overburden drilling with air flushing in granular soil where the air-lift pump effect may induce ground water flow towards the drill bit that may cause a substantial amount of fine-grained soils being eroded and "sucked" into the borehole resulting in cavities around the casings. It is assumed that such a “Venturi” effect enhances the air-lift effect. Bredenberg et al. (2014) explains this by the Bernoulli equation and that the flushing medium and cuttings have a significant higher flow velocity than the ground water, thus creating a lower static pressure around the drill bit than in the surrounding ground. Drilling in ground conditions with erodible silty and sandy soils combined with artesian pore pressures and a high degree of ground water recharge is expected to increase the negative drilling impacts.

II: The eccentric reamer or ring bit has an outer diameter that is slightly larger than the casing, typically between 10-30 mm depending on the drilling system and dimension of casing that are used. In theory this will cause "over-coring" and a small gap around the casing, which could lead to some ground movements when the surrounding soil sinks in and closes the gap.

III: When drilling through soft clay and further into dense granular soils (e.g. sand or moraine) or bedrock, there is a risk of uncontrolled blowouts and erosion of soil mass (remolded clay, silt) along the outside of the casing. Such blowouts are mainly related to the use of air flushing, and increased pressure to run the hammer. Since air is a compressible gas, it is challenging to maintain a steady air flow, i.e. pressure and velocity, at all times during drilling in varying soil conditions. If the flushing channels for the return flow in the drill bit are blocked or constricted by cuttings, the risk of such blowouts increases rapidly.

IV: In situations where the drilling procedure is temporary stopped while drilling in sensitive clays, silt or fine sand, there is a risk that the soil surrounding the drill bit "collapses" into the bottom of the casing, potentially causing both volume loss and blocking of the flushing medium. This may typically be a problem if the casing is not filled with water to stabilize the pressure from the ground before drilling is stopped, or if there is an artesian water pressure at the front of the drill bit.

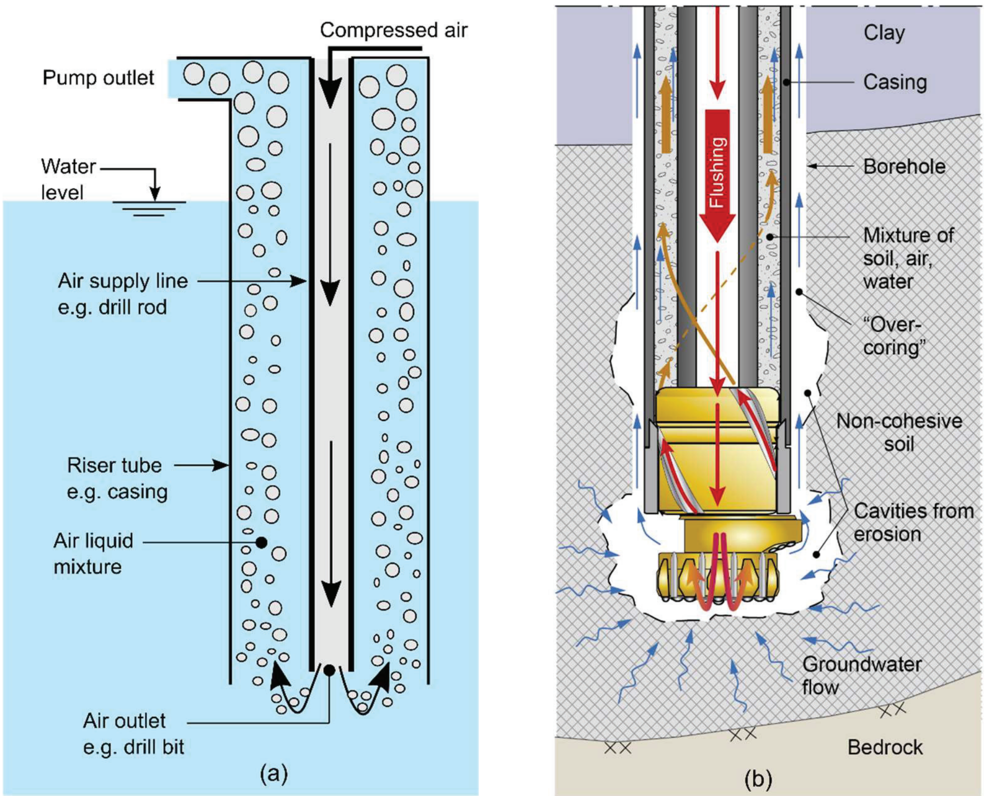


Figure 4-3. Illustration of (a) principles of an air-lift pump; and (b) erosion and loss of soil volume around the drill bit and casing caused by air flushing and the air-lift pump effect.

5 Full-scale field tests – drilling of anchors through soft clay and into bedrock

5.1 Introduction

This section describes and presents results from a full-scale field test program on anchor drilling that was carried out in 2013 as part of the research project *Limiting Damage*. The aim was to investigate the impacts on pore pressures and ground surface settlements from drilling with five different drilling methods/systems.

5.2 Test site

The field tests were carried out on a nearly flat agriculture field at Onsøy, about 100 km southeast of Oslo, Norway. Ground elevation varied from 6 to 7 m above sea level (masl) within the site, which had a total area of about 6000 m^2. The site was about 150 m from where pile load tests had been carried out previously, and where the ground conditions already were well known and documented (Karlsrud et al. 2014).

Figure 5-1 show the layout of the test site with the location of the five areas (A to E) where different drilling methods were tested. The layout gives an overview of boreholes and drilling directions for each anchor, as well as instrumentation installed to document effects of each drilling method. The directions of drilling were for each method oriented such that the potential overlapping effects would be minimized.

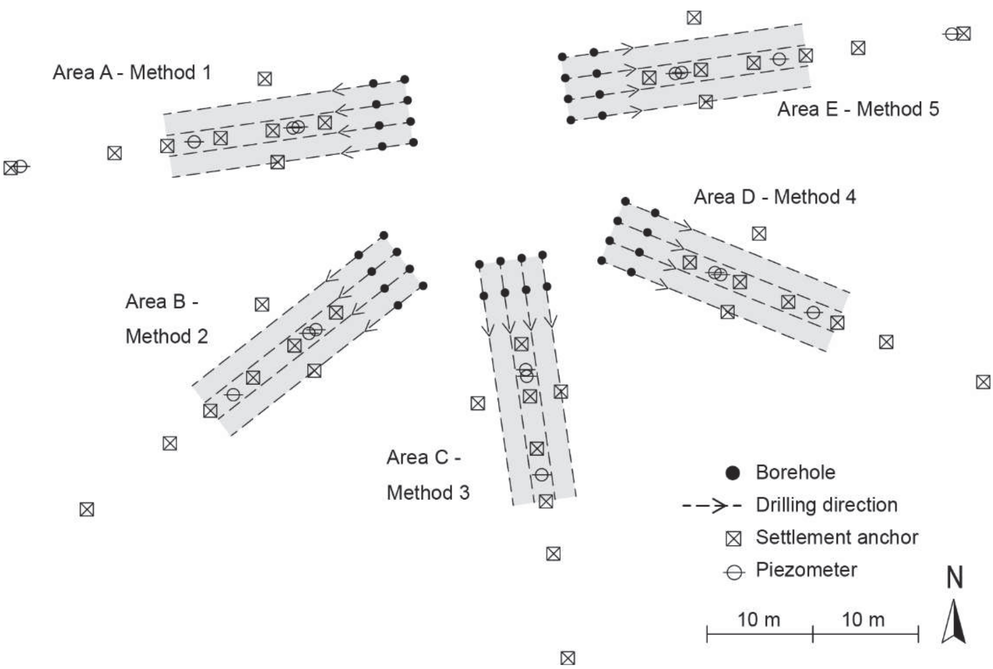


Figure 5-1. Layout of test site with area A to E and the related drilling methods.

5.3 Ground conditions

The ground conditions at the test site consists of about 0.5 m of organic topsoil over 1.0 to 1.5 m of dry crust. Underneath the dry crust is a layer of homogenous soft normally consolidated marine clay. The thickness of the clay deposit increased from about 13 m in Area E (north-east) to about 23 m in Area B and C (south-west). Figure 5-2 presents a typical soil profile with index data and in-situ stress conditions for Onsøy clay, based on soil investigations carried out at the site for pile load tests (Karlsrud et al. 2014). With a clay content in the range between 44-66% combined with plasticity index data, the clay is classified as medium to highly plastic. The bedrock is partly covered with a thin layer of dense sand/moraine. Observations during drilling of the anchors showed that the thickness of sand/moraine is 200 to 300 mm in Area A, C and E, and up to about 2 m at some of the anchors in Area B and D (Figure 5-1). The ground water level is registered at about 0.5 to 1.0 m depth below ground surface. Measurements show that there is a slight artesian pore water pressure of 10 to 20 kPa at bedrock.

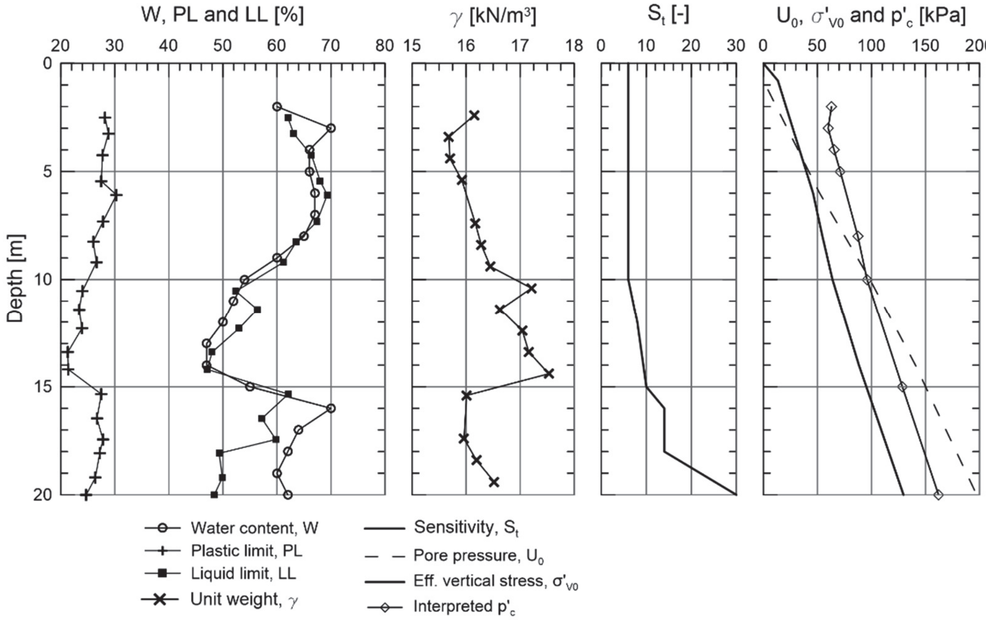


Figure 5-2. Index data and assumed in-situ stress conditions for Onsøy clay at site for pile load tests (after Karlsrud et al. 2014).

5.4 Instrumentation

To be able to measure and document the effects of drilling, the test site was instrumented with electrical piezometers (PZ) and settlement anchors. A total of 17 piezometers and 40 settlement anchors were installed about 3 weeks before the first tests started. Figure 5-3 shows the typical layout and cross section of the instrumentation installed at each test area, here represented by an example from Area B. Three piezometers with automatic logging were installed at 4.5 m, 10 m, and 17 m depth respectively in each test area. All were placed along the middle section of each test area, and with different distances from the boreholes (Figure 5-3). Two extra piezometers were installed as reference points in Area A and E. Due to smaller depth to bedrock in Area E (between 13 to 16 m), piezometer E10 and E11 (reference point) were installed to 14.8 m and 13.2 m depth respectively, both with the tip just above bedrock.

Ground settlements were monitored by means of eight Borros type settlement anchors (Geokon 2019) installed at 2 m depth within each test area. Settlements of the anchors were measured using a "total station" type theodolite. A bedrock outcrop about 100 m east of the test site was used as a reference point.

It was attempted to measure the volume of drill cuttings during drilling of some of the boreholes (anchors), and to compare this to the theoretical volume of the casings installed in the ground. In practice this turned out to be very difficult and it was not possible to get accurate measurements. However, based on observations during drilling it was possible to estimate if drilling caused loss of soil volume or soil displacements.

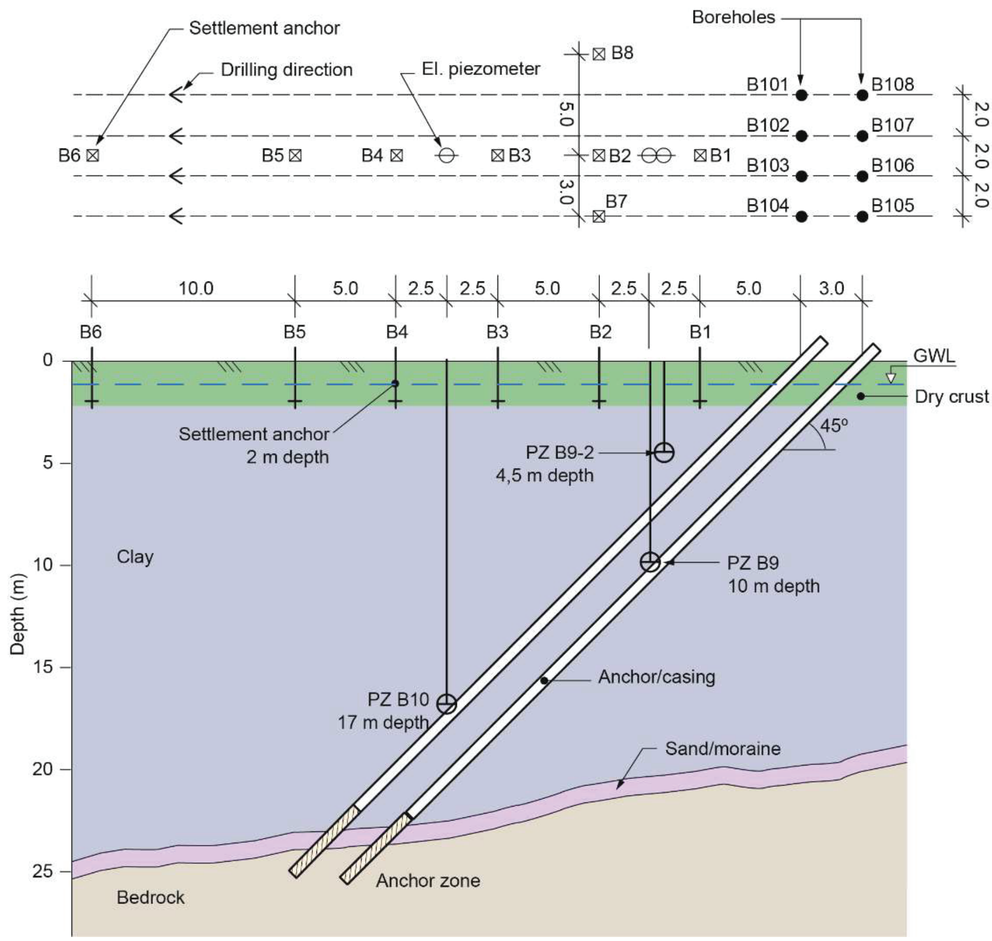


Figure 5-3. Layout and cross section of instrumentation for each drilling method (example from area B).

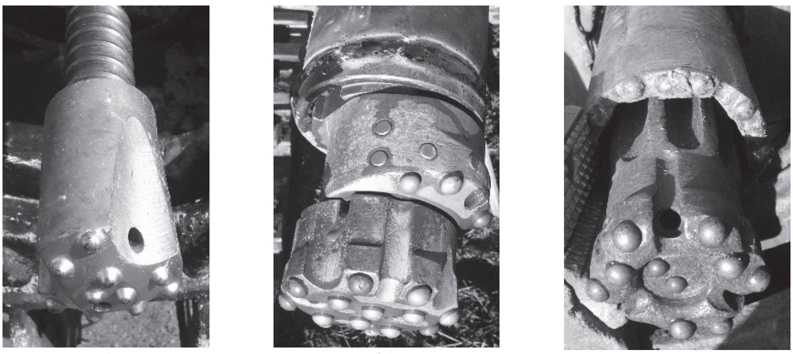
5.5 Drilling methods and procedures

Table 5-1 presents details regarding the five drilling methods used in the field test, including the time when drilling was carried out. While Method 2, 3 and 4 are commonly used in Scandinavia for overburden drilling through soft clays for both ground anchors and micropiles, there are limited experience with the other methods. Figure 5-4 show pictures of the different drilling systems and drill bits that were tested. Drilling in Area A were carried out with an uncased rotary percussive system with a 40 mm hollow-core steel rod (bars) with a 70 mm rock drill bit at the front (Figure 5-4 a). The same type of eccentric drill bit was used in Area B, C and D (Figure 5-4 b), however with a smaller dimension in Area D. In Area E a system with a concentric pilot drill bit and a ring bit on the casing was used (Figure 5-4 c).

Table 5-1. Overview of drilling methods used in field test.

		OD [mm]		
Area	Drilling method	Casing	Reamer	Time of drilling
A	1 – Top hammer with hollow-core steel	-	-	19.09.2013 – 24.09.2013
B	2 – DTH air hammer with eccentric drill bit	139.7	151.2	17.10.2013 – 22.10.2013
C	3 – DTH water hammer with eccentric drill	139.7	151.2	27.11.2013 – 02.12.2013
D	4 – Top hammer with eccentric drill bit	114.3	123.0	16.10.2013 – 17.10.2013
E	5 – Top hammer with concentric drill bit	114.3	120.0	30.10.2013 – 31.10.2013

Note: OD = outer diameter



a) b) c)

Figure 5-4. Pictures of different drilling systems and drill bits used in the field test, a) Method 1 with a 70 mm rock drill bit; b) Method 2, 3 and 4 with eccentric drill bit; c) Method 5 with concentric drill bit.

For each drilling method, a total number of eight "anchors" were drilled from ground surface with a 45-degree inclination, through the soft clay, the thin layer of dense sand/moraine and into bedrock. The anchors were placed in two rows 3 m apart, and with a spacing of 2 m between the anchors (Figure 5-3). The test program did not include installation nor post stressing of any anchor tendons/strands in the casings, since the focus was purely on the installation effects from the drilling.

Table 5-2 presents typical values for the main drilling parameters for the different methods that were tested. The drilling length in bedrock for each method is also given. All drilling with Method 1, 3, 4 and 5 was carried out using continuous water flushing. With Method 2 however, air flushing with about 12-15 bar air pressure was used to run the DTH hammer and to penetrate through the layer with sand/moraine and into bedrock. With Method 1 and 5 the top hammer was used to drill through the moraine and into bedrock. After the drilling was completed, the boreholes were filled (grouted) with a cement suspension (water to cement ratio = 0.4-0.7). The grout was pumped with low pressure through the drill rod, filling the borehole from the bottom up to ground surface. With Method 5 the casing was pulled up directly after grouting leaving the borehole supported only by the grout. To replicate a typical production drilling scenario the penetration rate through clay was generally high except with Method 3 where the rate was reduced significantly compared to the other methods (Table 5-2). The intention was to minimize excess pore pressures due to soil displacement as observed with the other drilling methods.

Table 5-2. Typical drilling parameter values in field test.

		Drilling method / Test area				
Drilling parameters		1/A	2/B	3/C	4/D	5/E
Water pressure (clay)	[bar]	5	20	90	5	5
Water flow rate (clay)	[l/min]	60	60	150-200	60	60
Penetration rate in clay	[m/min]	3-6	12	1-2	12	12
Rotation speed	[rpm]	60	60	60	120	120
Drilling length in bedrock	[m]	0-2.5	4	0.4-1.4	0	1.85-4.2

It was intended to drill all boreholes for the anchors 4 m into bedrock, and to provide a casing for the first 0.5 m (Method 2, 3, 4 and 5). However, for practical reasons, and to save time, the actual length of drilling into bedrock was reduced. The first two drill rods (anchors) with Method 1 broke trying to enter bedrock. To mitigate this problem, it was decided to drill the remaining six anchors steeper with about 56-degree inclination. The first three anchors with Method 1 were not grouted due to problems with clogging of the drill rod, and because one of the rods that broke sank into the borehole. With Method 4, it was decided to abort the test after drilling only two out of eight casings (D104 and D103). This was because drilling through the dense moraine layer and into bedrock was not possible with this drilling system. The main reason was probably a combination of depth to moraine (about 18 m), and that the top hammer and drill rod that were used could not supply enough energy. Drilling through the soft clay was carried out with water flushing, but additional flushing with pressurized air was used when trying to improve the drill cuttings transport and the penetration in the dense moraine.

5.6 **Results**

5.6.1 *Pore pressure measurements*

This section presents the main results from pore pressure measurements. All piezometers were logged continuously over a total period of about 8 months. To establish representative reference values, the piezometers were installed about 4 weeks prior to the drilling in Area A commenced. Data was logged in one-hour intervals during the whole test period and changed to one per day when all drilling was completed.

Monitoring data show that all the drilling methods caused excess pore pressures in the surrounding clay. The observed response on the pore pressure was generally much the same with all methods. However, Method 2 in Area B (DTH air hammer) and Method 5 in Area E (top hammer and concentric drill bit) resulted in significantly higher excess pore pressures than the other methods. Table 3 gives a summary of maximum excess pore pressures registered at each test area. The largest observed excess pore pressure was 70 kPa in PZ E9 at 10 m depth while drilling of anchor (casing) E107 in Area E with a minimum distance of about 1.1 m to the piezometer. The main reason for the large excess pressure was likely the high penetration rate of about 12 m/min combined with pressurized water flushing (Table 5-2). This resulted in the highest ratio of maximum excess pore pressure (ΔU_{max}) to the effective overburden stress (σ'_{V0}) of all the piezometers with a value of 1.14 (PZ E9). The minor changes in PZ E9-2 and PZ E10 (ΔU between 0 to 5 kPa) was most likely due to much longer distance between the casings and the piezometers as well as the relatively small dimension of the anchors (OD = 114 mm).

Measurements in Area A showed relative moderate changes in pore pressures with a maximum value, ΔU_{max} = 15 kPa in PZ A9. The results were likely affected by the relatively small dimensions of the drill bit (OD = 70 mm), and the change in inclination from 45 to 56 degrees for the last six anchors, hence increasing the theoretical minimum distance between the piezometers and the anchors.

Table 5-3. Summary of pore pressure data.

Area	PZ No.	Depth [m]	U_{ref} [kPa]	σ_{V0} [kPa]	σ'_{V0} [kPa]	ΔU_{max} [kPa]	$\Delta U_{max}/\sigma'_{V0}$	$\Delta U_{max}/U_{ref}$
A	A9	10	98	162,5	64,5	15	0,233	0,153
	A9-2	4,5	38	73,5	35,5	4	0,113	0,105
	A10	17	171	278	107	3	0,028	0,018
	A11	15,5	155					
B	B9	10	-	162,5	-	-	-	-
	B9-2	4,5	40	73,5	33,5	4	0,119	0,100
	B10	17	170	278	108	60	0,556	0,353
C	C9	10	101	162,5	61,5	18	0,293	0,178
	C9-2	4,5	42	73,5	31,5	10	0,317	0,238
	C10	17	175	278	103	18	0,175	0,103
D	D9	10	101	162,5	61,5	8	0,130	0,079
	D9-2	4,5	40	73,5	33,5	0	0,000	0,000
	D10	17	163	278	115	-17	-0,148	-0,104
E	E9	10	101	162,5	61,5	70	1,138	0,693
	E9-2	4,5	40	73,5	33,5	3	0,090	0,075
	E10	14,8	150	245	95	5	0,053	0,033

Figure 5-5 show changes in pore pressure (ΔU) with respect to time during drilling in Area B (Figure 5-5 a), Area C (Figure 5-5 b) and Area D (Figure 5-5 c) respectively. Time of drilling for each individual anchor is indicated with grey bars in the figures. Figure 5-5 a) show that drilling of anchor B104 caused an immediate excess pore pressure of about 60 kPa in PZ B10 at 17 m depth, while PZ B9-2 at 4.5 m depth only showed minor change (ΔU = 1-2 kPa). No data was available from piezometer B9 which was out of function during the field tests. Drilling of anchor B104 also caused an increase of about 13 kPa in PZ A9 (10 m depth) and 4 kPa in PZ A10 (17 m depth) in Area A, at about 30 m distance from anchor B104. The excess pressures were most likely caused by flushing with compressed air (12-15 bar) when drilling through the sand/moraine layer above bedrock. Small outbursts of air, water and remolded clay were observed up along the outside of the casing as well as along the previously installed anchor rods A104 and A103 in Area A. This shows that drilling with compressed air caused pneumatic fracturing, not only along the casing wall but through the moraine layer.

Despite the closer proximity to PZ B10, drilling of anchor B103 and B102 had less impact on the excess pressures than anchor B104. The results indicate that some of the flushing air evacuated through the moraine and joints/fissures in bedrock and up into the casing for anchor B104, rather than building pressures in the ground as with anchor B104. This mechanism was also observed for some of the other anchors in Area B.

Drilling of anchors B101, B108 and B107 reduced the excess pore pressure in PZ B10 by about 15 kPa in total. This reduction could be caused by ground water that was sucked into the casings with the backflow (drill cuttings) when drilling into bedrock. Based on visual observations the amount of water is roughly estimated to be between 20 to 30 l/min.

Piezometer B9-2 at 4.5 m depth showed insignificant changes, with a maximum accumulated excess pore pressure of about 4 kPa during drilling in Area B. Longer distance between PZ B9-2 and the anchors compared to PZ B10 combined with lower soil stress at shallow depth may explain this difference in response.

Figure 5-5 b) show that drilling with the DTH water hammer in Area C resulted in considerably lower excess pore pressures in the surrounding clay compared to Area B and E. The major difference is reasonable, considering the lower penetration rate when drilling through the soft clay in Area C (Table 5-2). Drilling of the first four anchors in Area C (C104 to C101) had minor influence on the piezometers except PZ C10 which showed an accumulated increase to a maximum value of 18 kPa after drilling of anchor C102 and C101. The excess pressure then decreased and where almost unaffected during drilling of anchor C108 to C105 due to longer distance to the casing. However, PZ C9 showed excess pressures of about 18 kPa while drilling of the anchors C107 and C106 with a minimum distance of about 1.1 m from the casings. Piezometer C9-2 at 4.5 m depth showed about 10 kPa increase in pore pressure during drilling of anchor C105, even with a minimum distance of about 5 m to the casing. This was 2-3 times higher compared to the piezometers at 4.5 m depth in the other test areas. The difference to the other drilling methods could be related to the significantly higher water pressures and flow rates used during drilling in clay (150-200 l/min @ 60-80 bar from the water pump). The flushing might have caused some hydraulic fractures in the upper part of the clay extending the influence zone.

The measurements in Area D (Method 1) are not directly comparable with the other methods since the test was aborted after drilling of the first two anchors (casings). The results are however interesting with respect to the installation effects from drilling. Figure 5-5 c) show that drilling of anchor D104 resulted in a pore pressure reduction of about 3 kPa in PZ D10 (17 m depth) and PZ E10 (14.8 m depth), which decreased to about 5 kPa during the following 24 hours. The pore pressure in PZ D10 reduced further to a minimum value of about 17 kPa right after drilling of anchor D103, still being about 10 kPa below the reference pressure 4 days later. Piezometer D9 (10 m depth) showed a temporary pressure reduction of about 2 kPa during drilling of anchor D103 before it increased evenly to a maximum excess pressure of about 8 kPa in the following 4 days. Some minor temporary increase in pressure between 2-4 kPa was also observed in PZ E10 and E11 during drilling.

The pore pressure reductions observed in both Area D and E were likely caused by some minutes of air flushing during drilling in Area D when trying to improve the transport of drill cuttings and penetrate through a layer of dense sand/moraine encountered at about 18 m soil depth. Flushing with air probably caused an "air-lift" pump effect (Behringer 1930; Kato et al. 1975) in front of the drill bit when the water and drill cuttings inside the casing was flushed up to the surface by pressurized air. This caused a lower pressure inside the casing compared to the pore pressure in the surrounding sand/moraine, thus creating a gradient, i.e. flow of ground water, towards the drill bit like a pumping well. Due to the higher permeability (hydraulic conductivity) in the sand/moraine layer compared to the clay, the effects of air flushing were noticeable in Area E over 20 m from the anchors. It is reasonable to assume that the recovery time for the pore pressure was increased since the two casings in Area D were not filled manually with water again after the drilling was aborted. The amount of drill cuttings generated from anchor D104 and D103 indicate that the drilling has formed a cavity, i.e. volume loss, around the casings in the moraine layer. The volume loss is likely the main reason for the pressure reduction in PZ D10, caused by suction in the clay above the cavity in the sand/moraine layer.

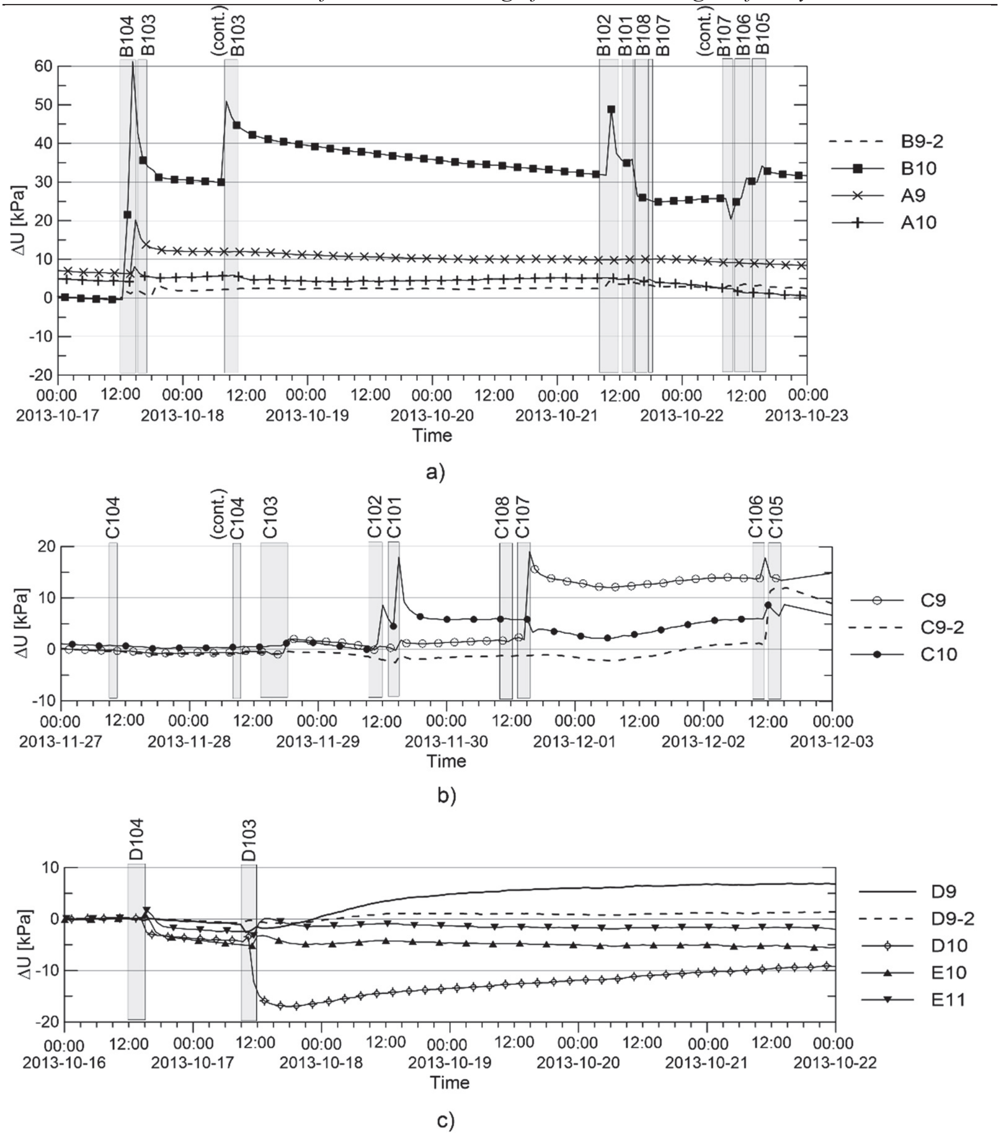


Figure 5-5. Changes in pore pressure (ΔU) against time during drilling in a) Area B; b) Area c); and c) Area D.

Figure 5-6 show dissipation of excess pore pressures against time for some piezometers in each test area. Most of the excess pore pressures dissipated rapidly after drilling was completed. This behavior coincides well with results from pile driving in clay reported by among others Li et al. (2019), Karlsrud (2012) and Edstam and Kullingsjö (2010). Similar dissipation trends are also observed with pile drilling, e.g. Ahlund and Ögren (2016) and Lande et al. (2016). In Area E only 10 kPa of the maximum excess pressure of 70 kPa remained two days after drilling of anchor E107, and about 30 days later it was completely dissipated. Pulling of the casings in Area E might have made it easier for the excess pressure to dissipate to the grouted borehole, hence speeding up the process compared to the other test areas. PZ B10 showed a slower trend with about 8 kPa excess pressure remaining 150 days after drilling was completed in Area B. The longer dissipation time indicate a more severe influence from drilling on the surrounding clay.

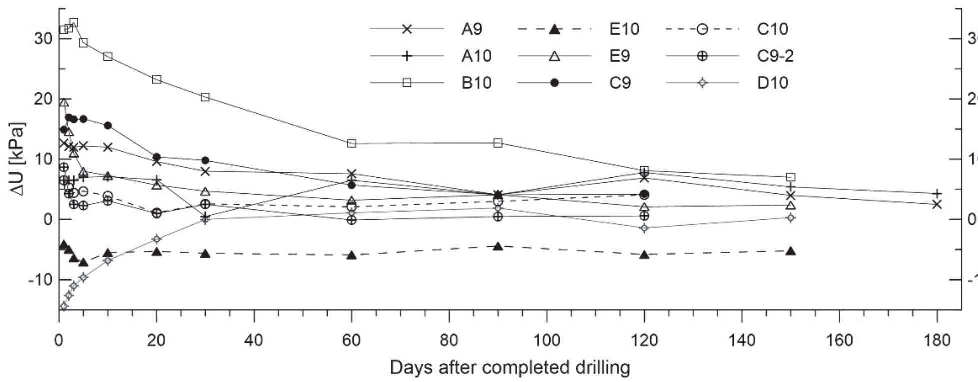


Figure 5-6. Dissipation of excess pore pressure (ΔU) in the clay over time.

Figure 5-7 presents the maximum change in pore pressure (ΔU_{max}) at different depths in each test area against the distance from the anchors (casings). Figure 5-7 a) show results against the ratio of radial distance from casing r to the radius of casing r_0, and Figure 5-7 b) against metric radial distance. The data represents the maximum values obtained from drilling of single anchors with each method. For comparison, typical excess pore pressure curves are also shown for a driven closed-ended pile, based on the strain path method (SPM) theory (Baligh, 1985) and procedure described by Karlsrud (2012). The estimated excess pore pressure is for a single pile with a diameter equal to the casings in Area B and C (OD = 140 mm), at a depth of 10 m and 17 m below ground level. Driven closed-ended piles are here considered to represent a "worst case" scenario in terms of generating soil displacements and excess pore pressures in the surrounding ground. Drilling should ideally represent the opposite, i.e. a method which removes the soil volume of the casing being installed thus limiting soil displacements.

Figure 5-7 include some results from a recent field test with jacking of open-ended concrete piles (OD = 300 mm and wall thickness 70 mm) in soft organic clay (Li et al., 2019). It also includes results reported by Ahlund and Ögren (2016) from a field test comparing air and water driven DTH hammer for drilling of piles (OD = 139.7 mm). The influence from air flushing was considerably larger than with water flushing, but both methods are in the lower range compared to the results in this field trials.

Results from these field trials show that drilling may cause considerable higher excess pressures than previously reported for driven closed-ended piles. The main reason for the excess pressures is likely related to the high penetration rate used for all drilling methods except in Area C (Table 5-2) causing soil displacements. The results indicate that the flushing process enhances the effect on soil displacements.

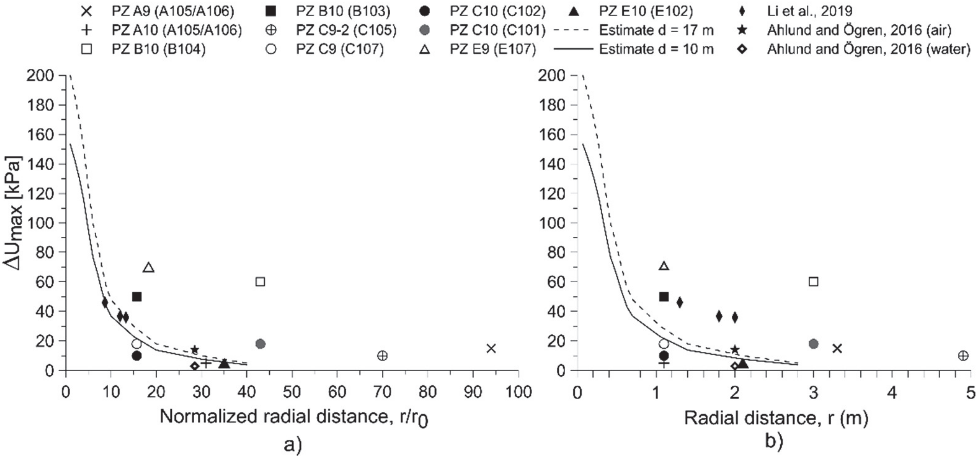


Figure 5-7. Maximum change in pore pressure (ΔU_{max}) against: a) normalized radial distance; and b) metric radial distance from anchors (casings).

5.7 Ground settlements

All the 40 settlement anchors at the test site were monitored over a period of about 9 months. The first measurements ("baseline") were made 9th September 2013, ten days before the field tests started in Area A. Settlement were measured more frequently during the field test (between 1 to 3 times a week) to be able to document any immediate effects from drilling. The frequency was reduced to every 4-5 weeks after drilling was completed in Area C. Accuracy of the measurements were specified as ±1-2 mm by the surveying company.

Figure 5-8 presents the vertical ground settlements (δv) measured on settlement anchor number 4 (see Figure 5-3) in each test area from 9th September 2013 to 7th January 2014. The grey bars show the time when drilling took place in each area. The monitoring data shows that drilling in Area B (Method 2), and likely in Area D (Method 4), caused almost immediate settlements between 2 and 7 mm on all settlement anchors over the entire test site. Results of the pore pressure measurements and observations during drilling clearly indicate that these settlements were caused by air flushing used with Method 2 and 4. The settlements in Area B and D can be explained by local erosion and volume loss around the casings, but such volume loss can hardly explain the influence on the other test areas. None of the other drilling methods caused similar short-term settlements or influence on other areas.

After drilling in Area B was completed, subsequent measurements until June 2014 showed 2-6 mm settlements over a period of 3 months (between 7th January and 4th April 2014). During this period some of the remaining excess pore pressure (about 5-10 kPa) in PZ B10 dissipated, thus indicating re-consolidation of remolded clay. There were no significant further settlements in Area A, C, D or E during this period, but rather indications of heave (1-3 mm) in some points in Area C, D and E. Freezing of the topsoil during the winter may have led to the registered small uplift on the settlement anchors in this period. Mitigating measures in terms of insulating the outer pipe (casing) above ground surface and filling frost inhibiting liquid between the outer pipe and the settlement anchors were carried out. Despite this effort the anchors seem to have experienced some frost induced uplift.

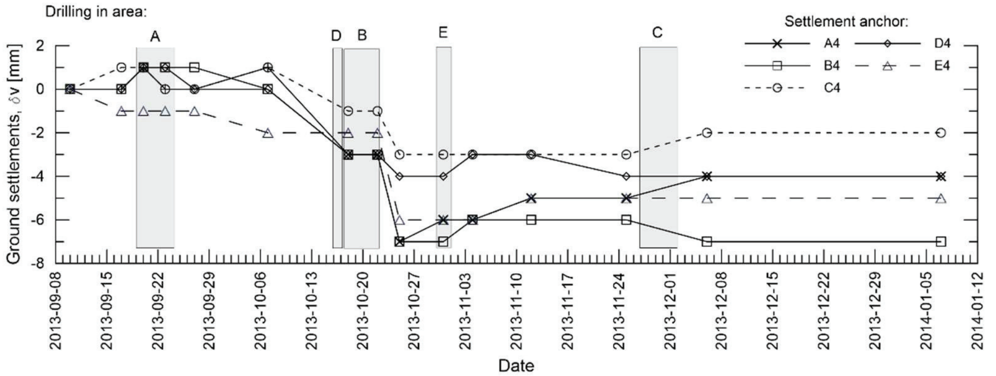


Figure 5-8. Vertical ground settlements (δ_V) measured on Settlement Anchor 4 in Areas A-E from September 9, 2013, to January 7, 2014.

Figure 5-9 shows the resulting ground settlements of all anchors measured on June 6, 2014. Method 2 (Area B) clearly stands out with significantly larger settlements compared to the other methods with a maximum value of 12 mm. This is also clear from Figure 5-10 were settlement profiles for anchors 1 to 6 in each test area are presented. Settlement anchors 2 to 5 in Area B settled the most with 11-12 mm which is about twice the settlements in Area E. This shows that the area directly above where the anchors hit the sand/moraine layer and entered bedrock were most affected. The fact that most of the settlements measured in all the test areas occurred after drilling with Method 2 (and 4) took place, indicates that drilling with air flushing had a major impact and a large influence area.

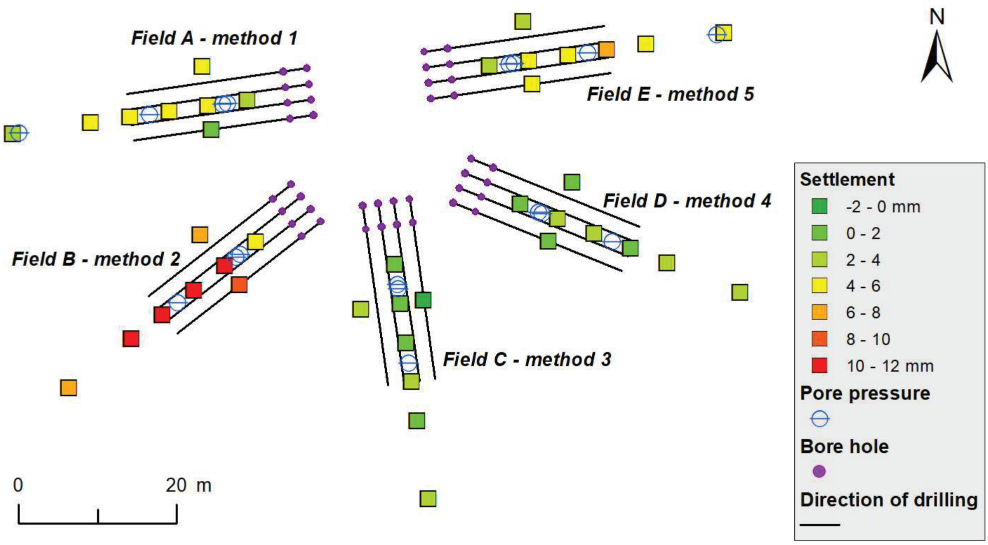


Figure 5-9. Layout of each test area with resulting ground settlements measured on June 6th, 2014.

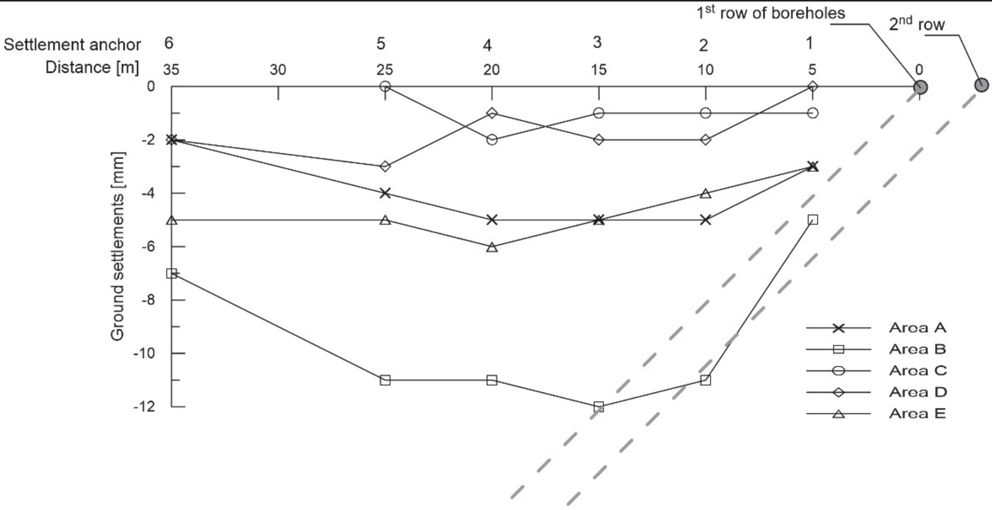


Figure 5-10. Ground settlement profiles for Anchors 1-6 related to distance from the first row of boreholes at ground surface. Data from final measurement on June 6, 2014.

5.8 Discussion

Monitoring data presented above clearly show that drilling Method 2 (DTH air hammer with eccentric drill bit, Area B) and Method 5 (top hammer with concentric drill bit, Area E) resulted in significantly larger excess pore pressures in the surrounding clay than any of the other methods. The main reason for this difference is likely due to the higher penetration rate with Method 2 and 5 (12 m/min) compared to Method 3 (1-2 m/min). Measurements in Area B and E showed that the amount of drill cuttings from single anchors were 50 to 80 percent less than the volume of casings installed, thus indicating significant soil displacements. With Method 1 (Top hammer with hollow-core steel, Area A) and 3 (DTH water hammer with eccentric drill bit, Area C) the drill cuttings tended to be larger than the volume of the drill rod/casing, indicating a net volume loss or "over-coring" which may explain the smaller excess pore pressures measured in these areas (Figure 5-7).

The effects of soil displacements when installing a casing through clay may be comparable to installation of displacement piles in clay if the displacement ratio is similar. Effects of pile installation on displacements are reported by among others Randolph and Wroth (1979), Baligh (1985), Lehane and Jardine (1994), Edstam and Küllingsjö (2010), and Karlsrud (2012). However, the results presented in Figure 5-7 show that the excess pore pressures and influence zone was much larger than expected based on past experiences with driven closed-ended piles. This indicates that flushing with pressurized water (5-20 bar) may have increased the soil displacements.

The soil displacements in Area B and E should in theory have caused some minor ground heave. This was however not observed by the monitored settlements since they only captured the accumulated total effects after drilling of one or more anchors. It is therefore reasonable to assume that the high penetration rates and soil displacements may have contributed to reduce the ground settlements.

The immediate ground settlements occurring over the entire test site right after drilling with Method 2 (Area B) and 4 (Top hammer with eccentric drill bit, Area D) are likely explained by a combination of two main effects: local erosion and loss of soil volume, and temporary pore pressure reduction above bedrock:

- The uncontrolled outbursts of air, water and remolded clay which were observed on the outside of anchor B104 (Area B) plus A104 and A103 (Area A) may have contributed to some local erosion of moraine and clay causing cavities around those specific anchors. However, this effect probably gave a minor contribution to the ground settlements and cannot explain the large, influenced area.
- A significant amount of ground water was flushed up through several of the casings in Area B when drilling into bedrock with air flushing. It was not possible to measure the volume of water, but the discharge was estimated to be between 20-30 l/min. It is likely that the air flushing used in Area B and D caused a temporary drop in the in-situ pore pressure within the thin permeable layer (sand/moraine) overlying bedrock, which probably also caused some settlements. This air flushing effect could have influenced a larger area than what is caused by a local volume loss just around casings. The pore pressure measurements at PZ E10 and PZ E11 installed just above bedrock in Area E gave evidence of such air flushing effect.

Both monitoring data and observations substantiate the hypothesis that drilling with air flushing may cause an "air-lift" pumping (see Figure 4-3). Flushing with air will reduce the density of the soil-air-water mixture inside the casing, thus creating a lower pressure inside the casing than in the surrounding ground. The difference in pressure induces a flow of ground water towards the drill bit that also may cause substantial amounts of erodible soils such as silt and fine sand being transported ("sucked") into the borehole. This effect was also observed and reported by Ahlund and Ögren (2016), and it could explain the ground settlements reported in case records by Konstantakos et al. (2004), Kullingsjö (2007) and Bredenberg (2014).

The ground settlements in Area B continued to increase between 2 and 6 mm over a period of about 5 months after drilling was completed. This indicates that about 40 to 50% of the resulting settlements is associated with dissipation of excess pore pressures and re-consolidation (i.e. change in void ratio) of possibly remolded/disturbed clay around the casings. The other four methods had similar, but smaller impact on pore pressures and settlements, and the settlements stopped shortly after drilling in these areas. Apart from some variations in the depth to bedrock, the marine clay deposit across the site is considered homogeneous. Variability in soil conditions is therefore not likely to explain the difference in settlements observed for the different drilling methods.

The total volume loss (ΔV_{1+2}) caused by re-consolidation of disturbed clay around the anchors have been calculated, and compared with the measured ground settlements, see Table 5-4. The method is specific for tieback anchor installation, but some of the inputs are based on results from field tests with driven closed-ended piles in clay (Karlsrud and Haugen 1984) and model test (Ni et al. 2009) which provide information on displacements and volumetric strains in the clay surrounding a closed-ended pile after complete re-consolidation. On basis of these experiences, a potential volume reduction due to re-consolidation of disturbed or highly strained clay around the drill string is estimated as follows: A volume reduction, $\varepsilon_{v,1}$ of 15% within a 20 mm thick layer of assumed completely remolded clay. A 10 percent volume reduction ($\varepsilon_{v,2}$) within partly remolded clay assumed to extend to a radial distance of 2 times the radius of casing.

Table 5-4 presents the different parameters used to calculate the estimated settlements from re-consolidation. Figure 5-11 illustrates the total area assumed to be influenced by re-consolidation of clay around the anchors in each specific test area. The ground surface area A_{cons} is assumed limited horizontally by an inclination of 2:1 from the depth of bedrock to the ground surface. The ground surface area A_{cons} is multiplied with the measured mean ground settlements from re-consolidation $\delta_{V;cons}$ to find the total volume loss for all anchors ΔV_{cons}. The measured volume loss can then be compared with the estimated volume loss ΔV_{1+2}. In test Area B, a total area of about 680 m^2 experienced a mean ground settlement of 2-3 mm due to re-consolidation.

This equals a total volume loss for all anchors, ΔV_{cons} between 1.36 to 2.04 m^3, which coincides reasonably well with the calculated volume loss, ΔV_{1+2} = 1.55 m^3. The estimates for drilling Method 1 and 5 also show rather good agreement, but the results are only used as an indication of the potential volume loss due to re-consolidation. The measurements in test area 1, 3 and 5 did not show any clear trends of settlements from re-consolidation.

Table 5-4. Estimated total volume loss due to re-consolidation of disturbed clay around anchors on the test site.

Method / Area	$\sum L$ [m]	r_1 [cm]	r_2 [cm]	V_1 [m^3]	ΔV_1 [m^3]	V_2 [m^3]	ΔV_2 [m^3]	$\sum\Delta V_{1+2}$ [m^3]	A_{cons} [m^2]	$\delta_{V;cons}$ [mm]	ΔV_{cons} [m^3]
1/A	176	5.5	7.0	0.995	0.149	1.037	0.104	0.253	315	0-1	0-0.315
2/B	266	9.5	15.0	2.846	0.427	11.277	1.128	1.555	680	2-3	1.36-2.04
3/C	276	9.5	15.0	2.948	0.442	11.684	1.168	1.611	710	0	0.0
5/E	223	8.0	12.0	1.963	0.295	5.610	0.561	0.855	480	0-1	0-0.48

$\sum L_{anchor}$ = Total length of all 8 anchors in each area
$r_1 = r_0 + 2$ cm = radius of completely remolded clay
$r_2 = 2\times r_0$ = radius of partly remolded clay
V_1 = Volume of completely remolded clay
V_2 = Volume of partly remolded clay
ΔV_1 = Volume loss in completely remolded clay
ΔV_2 = Volume loss in partly remolded clay
$\varepsilon_{v;1}$ = 15 % = volume reduction of completely remolded clay due to re-consolidation
$\varepsilon_{v;2}$ =10 % = volume reduction of partly remolded clay due to re-consolidation
A_{cons} = Area at ground level assumed to be influenced by re-consolidation
$\delta_{V;cons}$ = Mean ground settlements measured
ΔV_{cons} = Total volume loss based on measured ground settlements

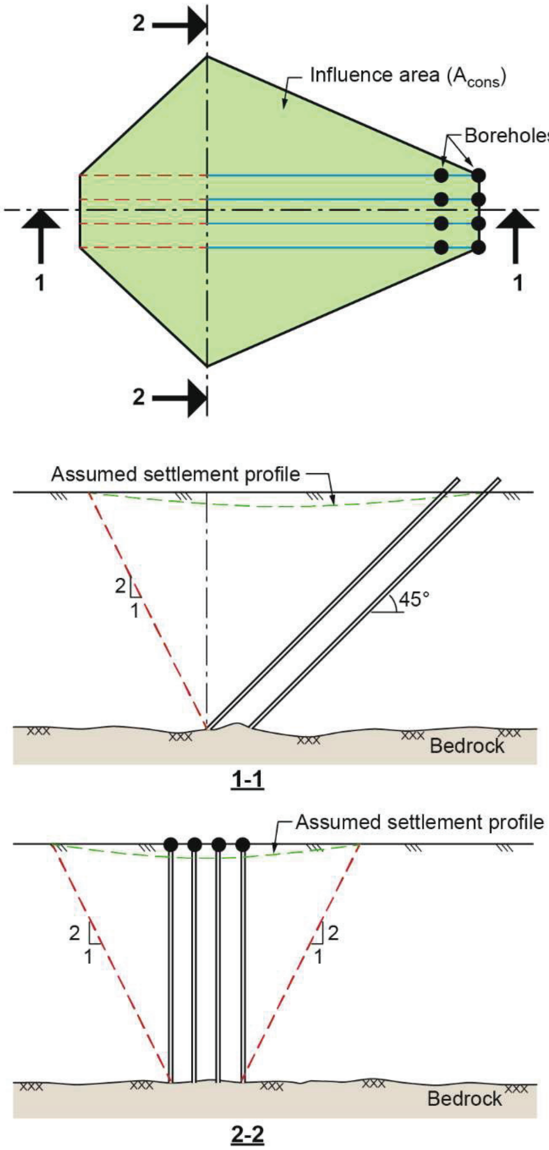


Figure 5-11. Assumed influence area from reconsolidation of remolded clay around anchors.

5.9 Conclusions

Results from a full-scale field test program with drilling for tieback anchors through soft clay and into bedrock are described and presented. Monitoring data and observations during the tests have provided new valuable information about the main installation effects due to overburden drilling, with focus on the influence on pore water pressures and ground settlements.

Drilling through soft clay is often performed with a high penetration rate, thus causing significant soil displacements, and excess pore pressures. Results from test Area B and E shows that the excess pore pressure in the surrounding clay can become much higher than what is expected for driven, closed-ended piles with the same diameter. However, most of the excess pore pressure seems to dissipate during the first days after drilling, while the remaining dissipates as part of re-consolidation of remolded clay which may take several months. The results from Area C (Method 3) give reason to assume that a drilling penetration rate of about 1 m/min reduces unwanted soil displacements and excess pore pressures in the surrounding clay. This is assumed valid for flushing with water and may be different if air flushing is used in soft soils. The soil displacements caused by high penetration may to some extent compensate for other effects that may cause ground settlements, i.e "over-coring" and loss of soil volume.

Drilling with air flushing may cause uncontrolled outbursts of compressed air along the casing and into the surrounding ground, as observed in Area B with the Down-The-Hole air hammer. Such outbursts can lead to "over-coring" and cavities around the casings. Air flushing can also cause temporary reductions in pore pressure due to the "air-lift" pump effect. Results from piezometers installed down to the moraine layer in Area E (PZ E10 and PZ E11) and just above it in Area D (PZ D10), clearly indicated that air flushing with Method 4 (Area D) caused a temporary drop in pore pressure within a thin permeable layer (sand/moraine) above bedrock. The same effect likely occurred in Area B but was not detected by the piezometers since they were here installed about 5 m above the moraine layer. Reduced pore pressures are most likely the main reason for the immediate settlements that were measured in the entire test field area. Air flushing may also cause erosion and cavities around the casing due to the “air-lift” pump effect. Drilling with only water flushing will not cause such a large “pumping effect”, and therefore reduce the risk of causing large ground settlements.

The relatively small ground settlements generated in this field tests stand in strong contrast with the large settlements (up to 40 cm) that were reported by Langford et al (2015) around excavations in soft clays supported by tieback anchored sheet pile walls. The main reasons for this difference are probably:

- The relatively few anchors/casings with small diameter installed at the test field, hence limited affected soil volume.
- Drilling at the test field was carried out from ground level. This implies reduced un-balanced earth pressures at the top of the casing during drilling as compared to drilling from a lower level within an excavation.
- More importantly, drilling from ground level excludes any effects of drainage up along the casing, which is commonly observed when drilling from below the water table within an excavation. Drilling from below the water table commonly reduces the pore pressure to the level of the top of the casing, and starts a consolidation process from bedrock and up through the clay deposit which can cause large settlements, e.g. Langford et al (2015)

Overburden drilling of casings involves a combination of rotation, penetration and flushing with air or water, thus making it a very complex process. The natural variations in ground conditions, quality of workmanship, drilling systems and procedures used make it difficult to foresee the installation effects on the surrounding ground. Given the limited research on this topic, the authors recommend gathering and analyzing more field data from drilling in different ground conditions. It is particularly important to investigate further the effects of drilling with air vs. water flushing, and to test out existing and new drilling methods from the bottom of excavations in a similar controlled manner as done in this study from the ground surface. In due course that may lead to methods that limit the undesirably large ground movements observed because of drilling for tie-back anchors or piles to bedrock from within deep excavations involving soft clays.

6 Small-scale modelling of pile drilling in sand

6.1 Introduction

Previous research on installation effects from drilling for anchors and piles have mainly been related to field studies (Sandene et al. 2021; Lande et al. 2020; Asplind 2017; Ahlund and Ögren 2016; Bredenberg et al. 2014; Küllingsjö 2007; Konstantakos et al. 2004). Different drilling methods, borehole dimensions, ground conditions and procedures in the field make it difficult to investigate the mechanisms and drilling parameters affecting the ground. In this context, a physical modelling approach was chosen. A series of pile drilling tests in saturated sand was carried out at the Norwegian Geotechnical Institute (NGI) in Oslo, Norway. The main objective was to deepen the understanding of the mechanisms due to flushing with water or air and to investigate how the drilling parameters penetration and flushing rate affect the influence on the surrounding soil. The aim of the testing was to provide knowledge that can be used in planning and execution of drilling to reduce the risk of unwanted influence on the surrounding ground.

None of the previous studies included systematic and accurate measurements of drill cutting volume or mass to assess the potential soil volume loss and to verify the hypothesis of the air-lift pump effect. Neither have systematic studies of the effects of drilling parameters on the soil response to drilling including pore pressure changes and soil displacements been carried out.

Another mechanism that has been observed with overburden drilling is uncontrolled piping or hydraulic fracturing (i.e. pneumatic blowouts) along the outside of the casing caused by flushing with compressed air (Lande et al. 2020; Sandene et al. 2021). This behavior typically occurs during drilling of the first meters below ground surface due to low soil stresses but has also been observed when drilling at large depths. Such piping effects are comparable to fluidization which was investigated by among others Tsinker (1988), van Zyl et al. (2013), Alsaydalani and Clayton (2014), and Passini and Schnaid (2015).

There has been limited research specifically focused on installation effects of drilling in sandy soils, hence the mechanisms affecting the surrounding ground are not fully understood. A novel test set-up made it possible to replicate drilling with a miniature pile with simultaneously penetration, rotation and flushing with water or air through the pile. The following sections describe the experimental set-up including the model tank and pile, instrumentation, drilling simulation and test procedures. After the results are presented and discussed, conclusions are drawn.

6.2 Experimental set-up

6.2.1 *Model tank and instrumentation*

Figure 6-1 illustrates the model test set-up. The soil model was placed in a cube shaped steel tank (Figure 6-1 a). An aluminum reaction frame was fixed to the top of the model tank, acting as support for a linear actuator used to vertically move the pile. The actuator had a maximum stroke length of 300 mm and a push capacity of 8000 N. A load cell with a capacity of 5000 N was connected between the actuator and a rotation motor unit to measure the penetration force on the pile during the tests. The rotation motor had a swivel unit that made it possible to flush with water or air through the pile at the same time as it rotated and penetrated. Both the penetration rate and rotation speed (rpm) of the pile were controlled by adjusting the voltage on the power supply. The pile penetration was measured with an extensometer connected to the frame and rotation motor.

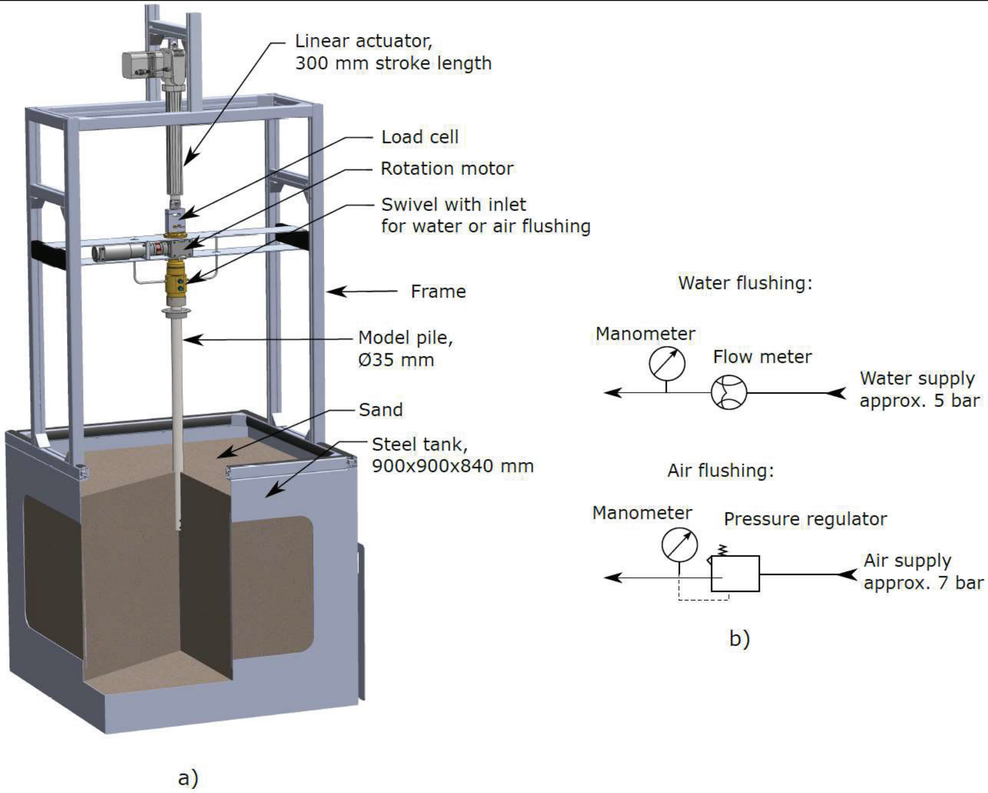


Figure 6-1. Test set-up: a) model tank, b) flushing pressure line.

Figure 6-1 b) shows a schematic illustration of the pressure lines for both water and air flushing. The water supply came directly from the main supply tap with an approximate pressure of 500 kPa. A flow meter was used to control the flow rate while a manometer was used to monitor the water pressure delivered to the pile during a test. For the air flushing tests, a pressure regulator with a manometer was used to control the pressure from the supply that had a pressure of approximately 700 kPa.

The entire tests were carried out with the model pile placed in the center position of the soil model. Figure 6-2 presents a layout of the model test set-up while Figure 6-3 show cross sections A-A and B-B through the model tank showing the positions of the instrumentation used to monitor the soil response. Measurements of pore-water pressures in the sand model were obtained using standpipes, i.e. plastic tubes with a diameter of 4 mm, that were connected to pressure sensors located at the outside of the model tank. Six standpipes were installed at two different soil depths (170 and 370 mm) and with three distances from the pile center (70, 140 and 210 mm) as can be seen in Figure 6-3 a). The standpipes were supported by vertical rods (Ø10 mm) that were connected to a steel plate which was placed on top of the bottom of the model tank. A filter was placed at the top of each standpipe to prevent sand grains from entering and affecting the measurements.

Vertical displacements of the soil surface (i.e. settlements) were measured with four linear variable differential transformers (LVDTs) positioned at different distances from the pile. A gantry (template for pile in Figure 6-3 b) was used to keep the pile and the LVDTs in position during the tests.

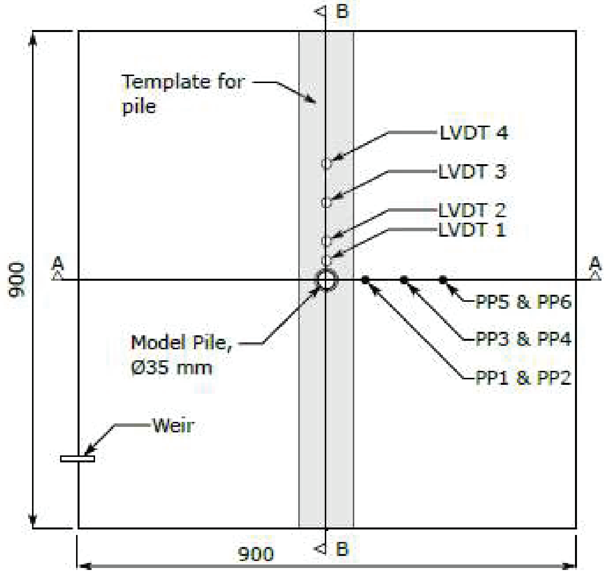
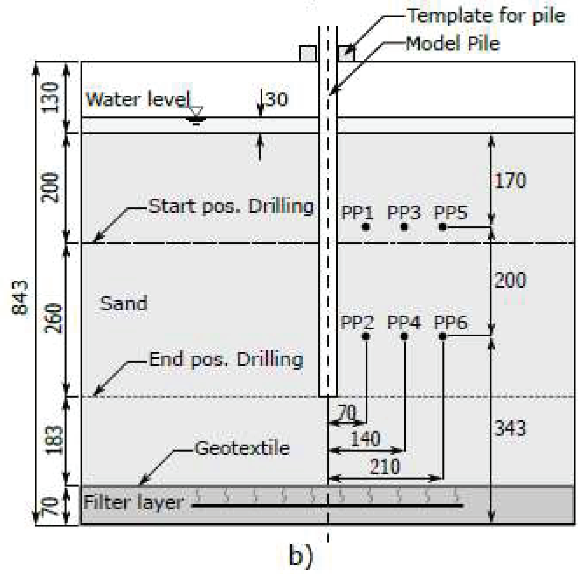


Figure 6-2. Plan view of model tank with positions of model pile, pore pressure sensors (PPs) and extensometers (LVDTs). Units in mm.



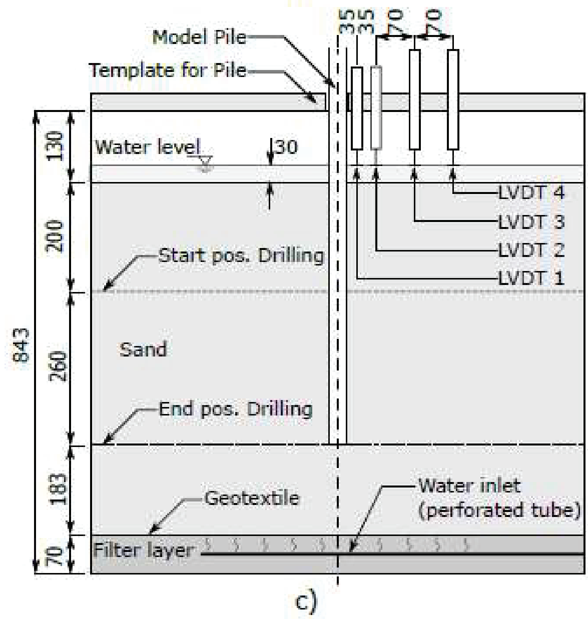


Figure 6-3. Experimental setup: b) cross-section through pore water pressure sensors (PPs) and c) cross-section through linear variable differential transformers (LVDTs). Units in mm.

6.2.2 *Model pile and drilling simulation*

Figure 6-4 shows a drawing of the model pile including details of the drill bit. The mechanical design was based on a prototype concentric drill system with a 114 mm diameter casing, giving a scale ratio of about 1:3.2 between the model pile and the prototype. The model pile is 890 mm long and consist of a casing, i.e. a steel tube with an outer diameter of 35 mm and 2 mm thickness as can be seen in Figure 6-4 a).

The flushing medium (water or air) is applied from the swivel device on the top of the pile through an inner steel tube with internal diameter of 6 mm. Between the casing and the inner tube is a middle steel tube that creates an annulus against the outer casing. The flushing backflow is transported through this annulus to the top of the pile, where a small catch-pot is used to collect the backflow (water and soil).

A drill bit is connected to the bottom of the casing/pile with 6 bolts. Four openings with a diameter of 4 mm were drilled through the drill bit and are used as the flushing inlet during drilling (Figure 6-4 b). The face of the drill bit was designed with cutting grooves at each flushing inlet to direct the flushing media and drill cuttings towards the backflow paths. The upper part of the drill bit has a packer system that enables one to close the annulus between the casing and middle tube to collect the soil particles in this annulus at the end of each test. The packer consists of a rubber membrane fixed with tie wrap and can be activated by pumping air into it.

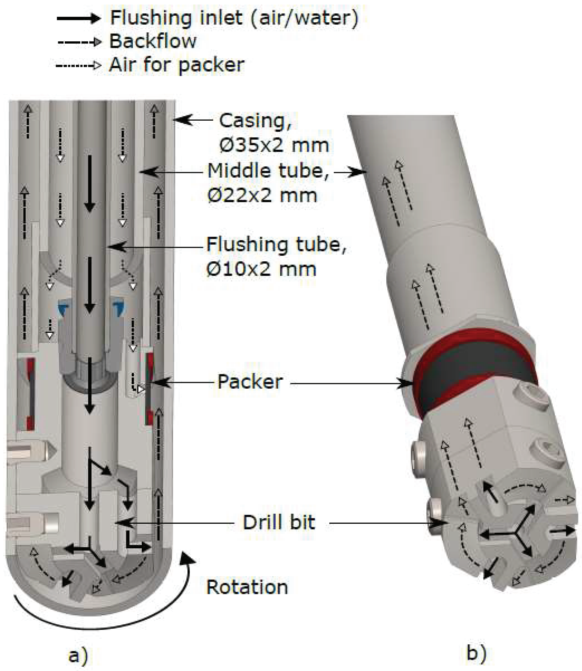


Figure 6-4. Model pile: a) cross section, b) inner parts excluding casing.

6.2.3 *Sand model preparation*

Figure 6-5 depicts different stages of the initial soil model preparation which was only carried out once for the entire test series. Two perforated plastic tubes were placed at the bottom of the model tank (Figure 6-5 a). One tube was used to pump water into the tank, i.e. applying an upward gradient, and the second tube for draining water from the bottom, i.e. applying a downward gradient. A permeable layer of approximately 70 mm lightweight expanded clay aggregates (LECA) was then placed over the perforated tubes (Figure 6-5 b). A geotextile layer was placed on top of the LECA and taped to the sides of the steel tank (Figure 6-5 c). The model tank was then filled with dry sand up to a thickness of about 640 mm (Figure 6-5 d).

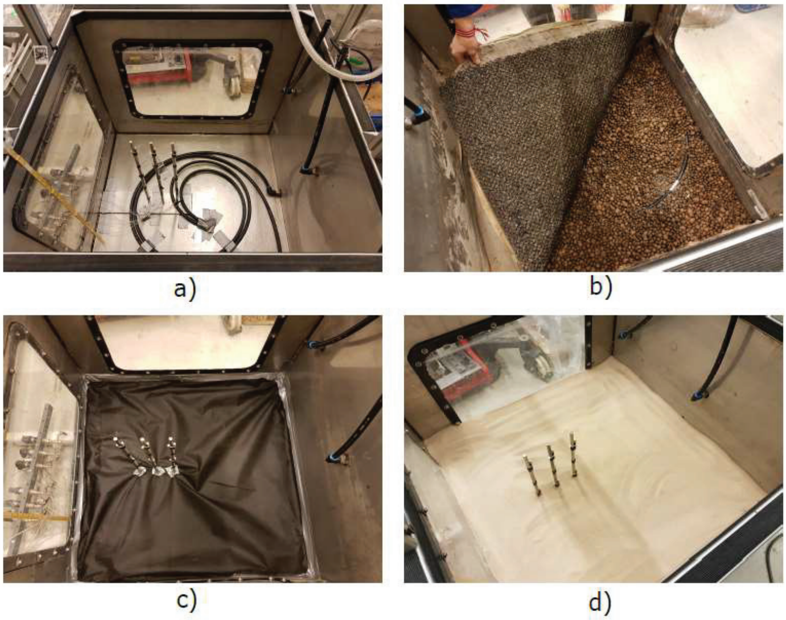


Figure 6-5. Stages in initial soil model preparation: a) support for pore pressure standpipes and perforated tubes for saturation and drainage of soil model; b) filter layer (LECA and geotextile); c) geotextile as separation layer; d) filling of dry sand.

All tests were carried out using Baskarp sand No. 15 (from Sibelco AB), which is a graded fine sand with well-documented properties based on extensive laboratory investigations (e.g. Ibsen and Bødker (1994) and Ibsen et al. (2009). Typical index properties of this sand are shown in Table 6-1.

Table 6-1. Index properties of Baskarp Sand No. 15 (after Ibsen and Bødker, 1994).

Property	Unit	Value
D_{50} grain size	mm	0.14
D_{60}/D_{10}		1.78
Specific gravity, G_s	kN/m^3	2.64
Maximum void ratio, e_{max}		0.858
Minimum void ratio, e_{min}		0.549

The soil was saturated using a similar procedure as reported by Passini and Schnaid (2015). An upward water flow from the perforated tube at the base of the model tank (Figure 6-3 b) and Figure 6-5 a) with a hydraulic gradient lower than critical was applied. After saturation, the water level was kept constant at approximately 30 mm above the soil surface throughout the tests by using a weir at the top of the tank (Figure 6-2).

For each test, a model preparation technique following Foglia and Ibsen (2014) was adopted. This systematic approach enabled to reuse the initial soil model without emptying the model tank. The following procedure was used for each water flushing test:

1. Sand loosening (approximately 15 min): Apply an upward water flow from the bottom of the tank through the perforated pipe using a hydraulic gradient close to critical.

2. Pile positioning: Position the pile vertically and horizontally above the soil surface using the pile template (Figure 6-2). Fill the annulus between the casing and middle tube with water up to the backflow holes at the top of the pile while the packer remains closed.
3. Pile pre-installation: Penetrate the pile until it reaches its start position approximately 200 mm below the soil surface (Figure 6-3) with limited water flow and no rotation. Open the packer to ensure that the water level inside the pile equalized to the water level in the model tank. Close the packer.
4. Sand compaction: Densify the sand using a concrete vibrator by following a specified pattern (Figure 6-6). The concrete vibrator was gently pushed down vertically until a defined penetration depth of approximately 600 mm was reached before it was slowly pulled up. Open the packer.
5. Uniformity testing: Test the uniformity of the sand model using a miniature cone. Figure 6-7 depicts the positions of these cone resistance tests.
6. Pile drilling: First, the data acquisition was switched on. Then, the flushing was turned on by opening the flow meter to a predefined value. Five seconds later, the pile rotation and penetration were turned on simultaneously. When the pile reached the end position of approx. 460 to 470 mm soil depth, penetration, rotation, and flushing was stopped at the same time and the packer was closed immediately to prevent sand particles flowing out of the pile casing.
7. Cone resistance testing: Cone resistance tests were carried out to investigate the influence from pile drilling.
8. Pile lifting and collection of drill cuttings: The pile was lifted and drill cuttings in the catch-pot and inside the pile were collected, dried, and weighed.

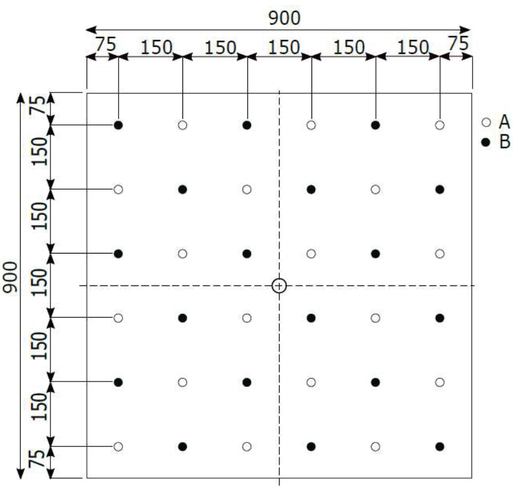


Figure 6-6. Grid for compaction of sand with concrete vibrator. Points "A" followed by points "B". Dimensions in mm.

After the initial soil model preparation (Figure 6-5), the sand was very loose with a mean relative density, D_r, of about 0.2. The relative density increased gradually after several rounds of loosening and compaction, reaching values between 0.6-0.65 for the presented tests. Cone resistance tests were carried out before the pile drilling to verify a consistent sand model preparation. A miniature cone with a diameter of 10 mm and an apex angle of 60° was connected to a steel rod and pushed 500 mm into the sand model using an actuator. The used penetration rate was 5 mm/s. Cone resistance tests were generally carried out after each soil model preparation and pile drilling test at distances of 90, 175, and 300 mm from the pile center to assess the influence from drilling (Figure 6-7). The order of testing in given positions was swapped for some of the pile tests to investigate potential local differences after the vibro-compaction.

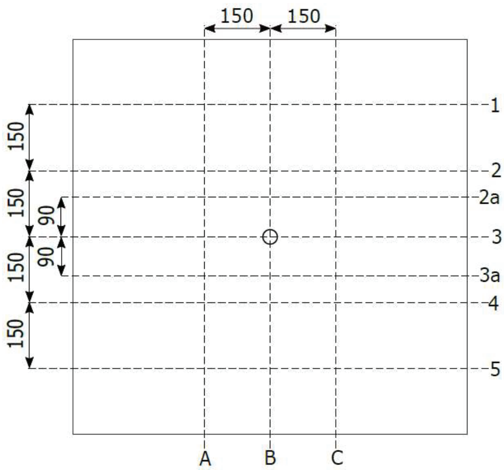


Figure 6-7. Layout for cone resistance testing of sand model. Dimensions in mm.

Figure 6-8 shows measured cone resistance after vibro-compaction at different distances from the pile center against penetration depth. The results confirm a relatively uniform soil model. The data, however, reveals that the soil resistance at distance r = 300 mm (Figure 6-8 c) is slightly lower than at r = 90 mm (Figure 6-8 a) and r = 175 mm (Figure 6-8 b). This could likely be explained due to the vicinity to the model boundary. This trend was found to be consistent for the entire test series and considered to have a marginal impact on the test results. For test W-9, a reduced cone resistance was measured at a depth of approximately 220 mm below the soil surface (Figure 6-8 a). This irregularity was caused by hitting the catch pot on the model pile during penetration and does not represent the real soil response.

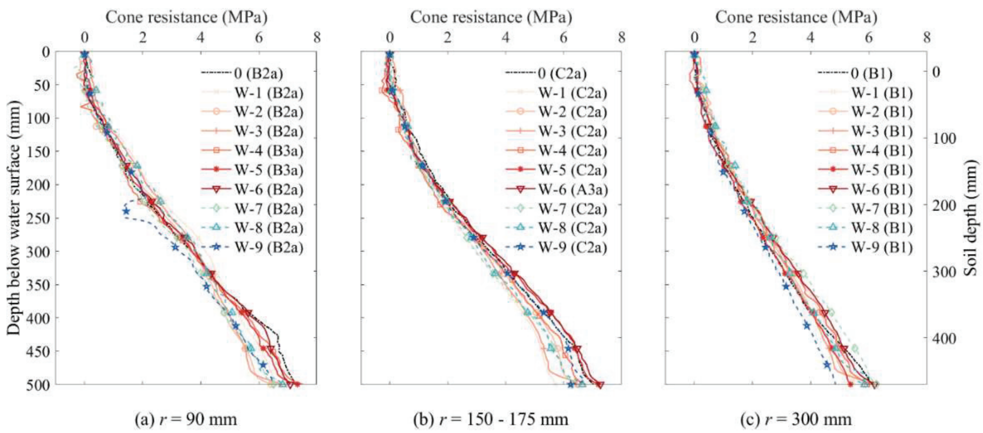


Figure 6-8. Cone resistance prior to pile drilling against depth measured at different radial distance from pile center, r.

6.2.4 *Test procedure*

An overview of the test program for this study is given in Table 6-2. The prefixes "W" or "A" indicate water or air flushing. Test 0 was carried out as a reference test by pushing the pile into the sand without any rotation and flushing. The flushing flow rate, Q, varied between 1.5 to 5.0 L/min for the tests W-1 to W-6, while the penetration rate, V, was kept constant at 2.5 mm/s. For the tests W-7 to W-9, the penetration rate varied between 2.0 and 4.0 mm/s while the flow rate was kept constant at 2.0 L/min. The starting value for the pile rotation was kept constant at 20 rpm for all tests except test 0. The flushing water pressure changed according to the given flow rate.

The tests A-1 to A-3 were carried out with the pile tip pre-installed to a starting depth of 400 mm and with flushing pressures of 50, 75 and 100 kPa respectively. An increased starting depth compared to the water flushing tests was required, because initial tests at a starting depth of 200 mm (i.e. identical to the water flushing tests) caused immediate piping effects on the outside of the pile and drill cutting transport was not observed.

Table 6-2. Test program

Test No.	Flow rate [L/min]	Pressure** [kPa]	Penetration rate [mm/s]
0	-	-	2.5*
W-1	1.5		2.5*
W-2	2.0		2.5
W-3	2.5		2.5
W-4	3.0	15	2.5
W-5	4.0	35	2.5
W-6	5.0	60	2.5
W-7	2.0		2.0
W-8	2.0		3.0
W-9	2.0		4.0*
A-1		50	1.5
A-2		75	1.5
A-3		100	1.5

*A notable reduction of the initial penetration rate of 2.5 mm/s was observed throughout the test.
**Water pressure not measured for the tests with flow rate below 3 L/min.

6.3 **Water flushing tests**

6.3.1 *Influence on penetration resistance*

Figure 6-9 shows the load cell measurements which provide a qualitative measure of the soil resistance against drilling depth. The start of drilling is at 200 mm soil depth (Figure 6-3). Test 0 (reference test) showed an immediate load increase to approximately 1400 N. This resistance aligns well with the expected bearing capacity of the pile tip at 200 mm soil depth. Further measurements show an almost linear increase in penetration force with depth, resulting in a maximum value of approximately 4100 N at drilling depth of about 260 mm (460 mm soil depth). Small decreases in load were observed at about 160 mm and 220 mm drilling depths respectively. These differences/deviations from the linear trend are likely explained by local inhomogeneity in the sand model.

Results from the water flushing tests W-2 to W-8 show no load or negative load values indicating tension caused by the self-weight of the model pile and rotation motor. The value of about -150 N corresponds well with this self-weight. This behavior indicates that the soil did not provide any resistance which is likely explained by local fluidization of the sand in front of the pile tip due to water flushing. Similar observations for pile jetting tests were reported by Tsinker (1988) and Shepley and Bolton (2014).

The load cell data for the tests W-1 and W-9 indicate that the flow rate (Q = 1.5 and 2.0 L/min) was too low to cause consistent local fluidization combined with the given initial penetration rate (V = 2.5 and 4.0 mm/s). For this reason, some soil resistance remained during drilling. This resulted in an increased penetration load compared to the other tests with maximum values of 1275 N and 1665 N in tests W-1 and W-9 respectively. A reduction of the initial rotation speed was observed for these tests which can be explained by substantial friction in the soil-pile interface. The results imply that both flow rate and penetration rate impact the soil behavior surrounding the pile.

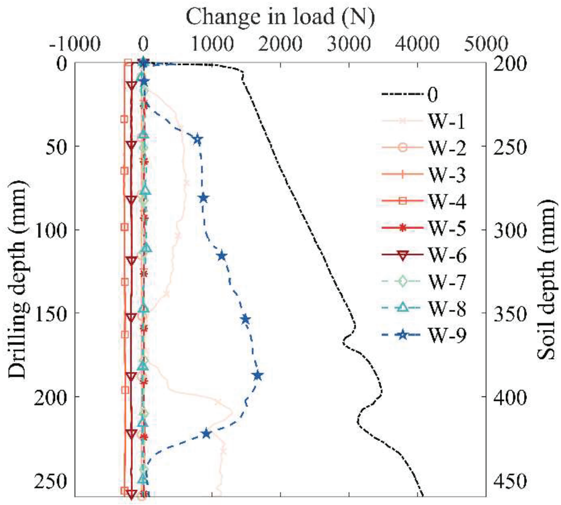


Figure 6-9. Load cell measurements against drilling depth.

6.3.2 *Influence on pore-water pressure*

Figure 6-10 presents the measured pore-water pressure changes (Δu) against pile drilling depth for all pore pressure sensors (PP1 to PP6) and for all water flushing tests including test 0 (reference). Test 0 clearly stands out compared to the other tests. The data show a significant decrease in pore pressure as the pile was pushed into the sand; a maximum value of about -2.3 kPa in PP1 (Figure 6-10 a) occurred rapidly after the pile penetration started. The pore pressure slowly increased during penetration, being about 0.5 kPa lower than the initial starting value at the end of installation. Similar trends were also observed in PP3 (Figure 6-10 c) and PP5 (Figure 6-10 e); however, the influence decreased at greater distance from the pile. PP2 (Figure 6-10 b) showed an immediate pressure drop of about -1.9 kPa, but unlike the "top PPs" (PP1, PP3, PP5) the pressure did not increase again before the pile tip reached a soil depth of about 320 mm. This response may be explained by the pile moving closer to the "base PPs" while the distance to the top PPs increased. Only minor pressure reductions of about 0.2 kPa were measured in PP4 (Figure 6-10 d) and PP6 (Figure 6-10 f). The pressure reductions observed in test 0 are likely explained by dilation effects in the sand surrounding the pile tip and shaft like what has been observed for a driven pile (e.g. White and Bolton 2004).

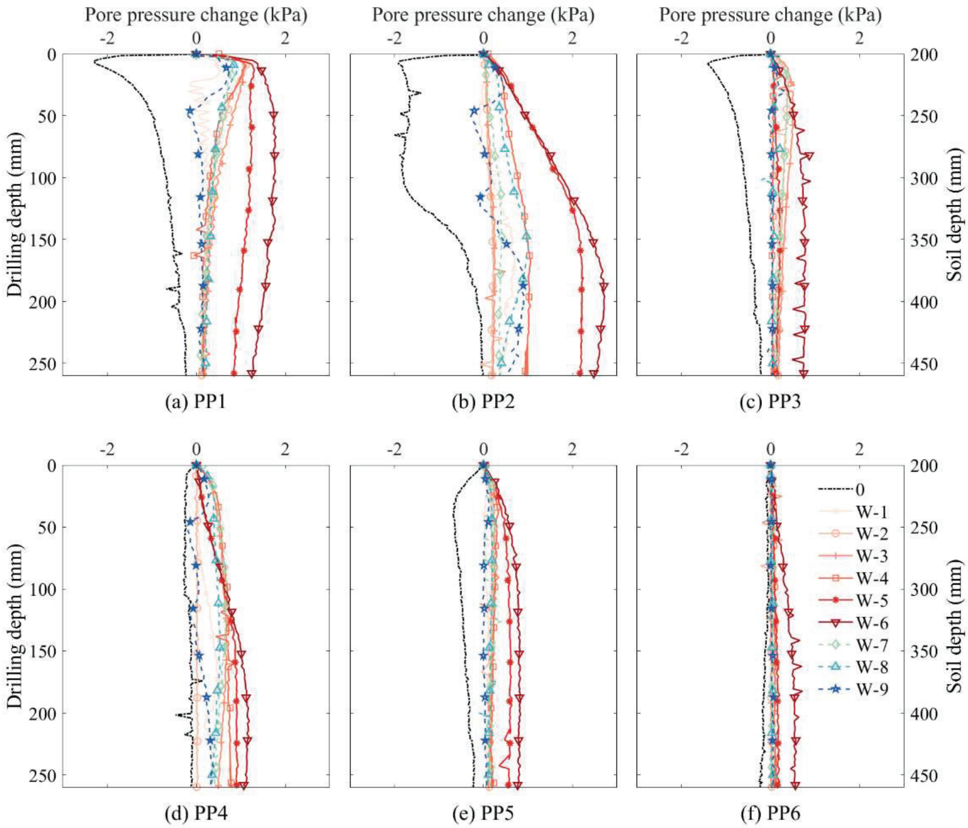


Figure 6-10. Measured pore-water pressure changes against pile drilling depth in PP1 to PP6.

All the water flushing tests (W-1 to W-9) caused excess pore pressures in the surrounding sand. As expected, the pore pressure values increase with the flow rate. Test W-1 (Q = 1.5 L/min) caused maximum pressure changes of about 0.5 kPa in PP1 and PP2 while test W-6 (Q = 5 L/min) showed corresponding values of about 1.7 and 2.8 kPa respectively. Test W-1 and W-9 were the only tests that caused some minor pressure reductions, i.e. negative pressure changes. This behavior is likely explained by the same dilation effects as observed with test 0, which agrees with the results from the load cell measurements (Figure 6-9).

PP1 at only 70 mm distance from the pile center typically showed an immediate excess pressure when the flushing was turned on, before slowly dissipating again as the penetration depth increased. The base PPs installed at 370 mm soil depth (PP2, PP4, and PP6) displayed a more delayed response in excess pressures compared to the top PPs at 170 mm soil depth. This was expected as the base PPs where furthest from the pile tip at the beginning of the tests; hence, the maximum influence was recorded when the pile tip reached approximately the same depth as the base PPs (i.e. a drilling depth of approximately 200 mm). PP5 and PP6 at a horizontal distance of approximately 210 mm from the pile center generally showed minor pressure changes during the tests. Only for test W-6, approximately 0.5 to 0.7 kPa excess pressure was measured by PP5 and PP6. For the tests with varying penetration rates (i.e. W-7 to W-9), clear trends in pore-water pressure changes were not observed.

Figure 6-11 shows the ratio between the pore pressure change (Δu) and the reference pressure (u_{ref}) against the normalized radial distance from the pile (r/r_0), where r_0 is the pile radius and r the radial distance from the pile. Figure 6-11 (a) and (b) present the maximum pore pressure change ($\Delta u_{\max}$) for the top PPs and base PPs respectively, while Figure 6-11 (c) and (d) presents the minimum pore pressure change ($\Delta u_{\min}$). The reference pressure is defined as the hydrostatic head at the theoretical position of the standpipe ends (i.e. filter positions) which results in a pore-water pressure of 2 kPa for the top PPs and 4 kPa for the base PPs. From Figure 6-11 the pore pressure change generally decreases with distance from the pile. This trend is more prominent for the tests with considerable flow rates (i.e. W-5 and W-6) and for the base PPs at 370 mm soil depth (PP2, PP4, and PP6). The data further indicate that the normalized change in pore pressure is less in the base PPs compared to the top PPs. This can be explained by the increase of the reference pressure with depth.

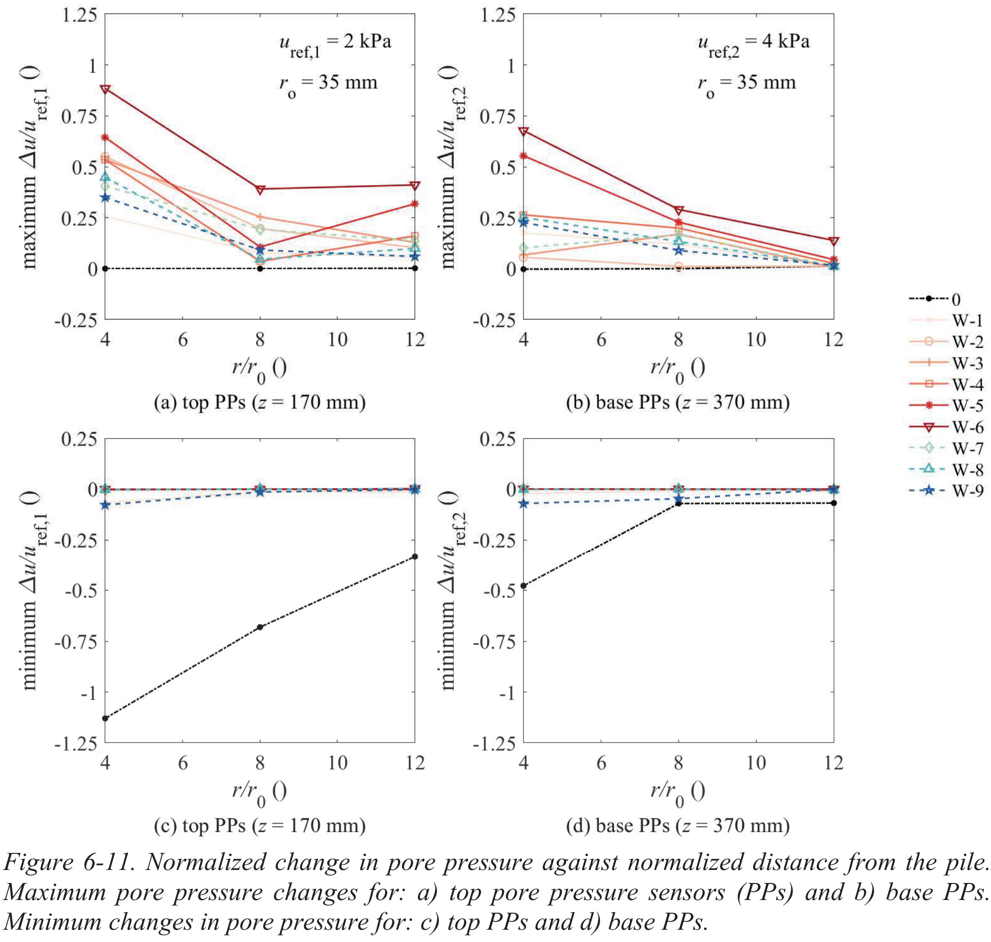


Figure 6-11. Normalized change in pore pressure against normalized distance from the pile. Maximum pore pressure changes for: a) top pore pressure sensors (PPs) and b) base PPs. Minimum changes in pore pressure for: c) top PPs and d) base PPs.

6.3.3 *Influence on soil displacements*

Figure 6-12 presents the measured vertical soil displacement (δ_v) against drilling depth for the four LVDTs. For test 0, a significant soil heave was monitored. LVDT1 positioned 35 mm from the pile center showed a maximum value of approximately 3 mm (Figure 6-12 a) which reduced to 1.8 mm in LVDT4 at about 210 mm distance from the pile center (Figure 6-12 d). This effect was expected since the pile was pushed in like a closed-ended displacement pile.

Results from the water flushing tests generally showed small soil surface displacements. Test W-1 indicates some minor heave (0.1-0.2 mm) in all LVDTs except LVDT1 closest to the pile which settled about 0.3 mm. Test W-9 caused heave in all the LVDTs. The heave in tests W-1 and W-9 could be explained by the pile drilling causing some soil displacements since the flushing was only able to partially fluidize and remove the sand in front of the drill bit (see above).

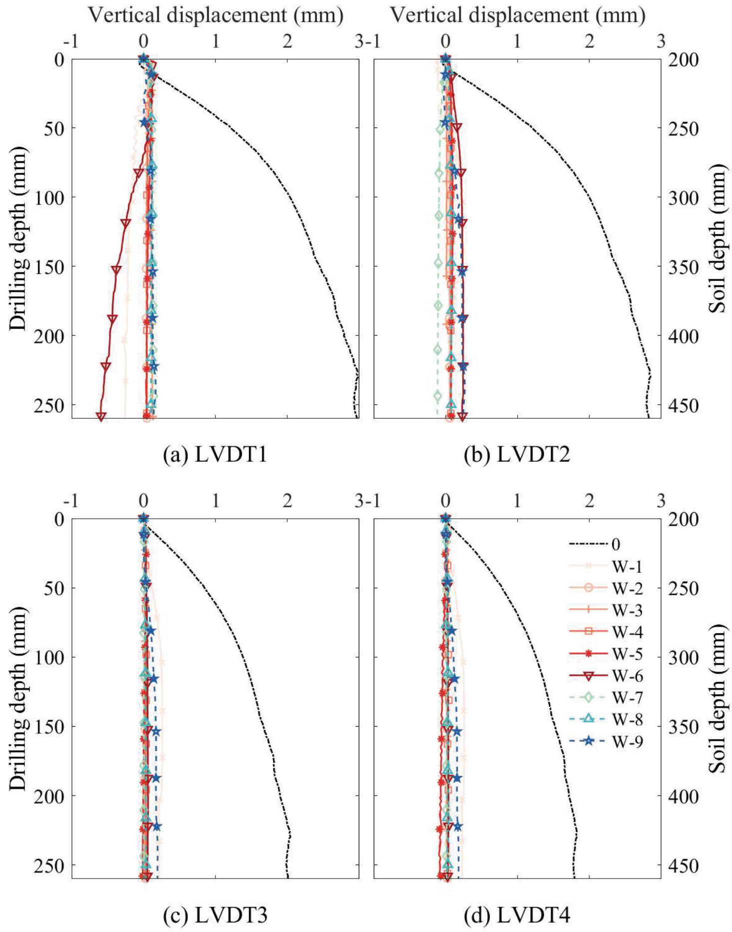


Figure 6-12. Soil surface settlements against drilling depth for a) LVDT1, b) LVDT2, c) LVDT3 and d) LVDT4. Negative values indicate settlements.

Visual observations after drilling showed that all water flushing tests as well as test 0 caused a small cavity (recess) in the soil surface with about 10 mm influence from the pile wall. Figure 6-13 shows a photo of such a cavity after the completion of test W-4. This effect could not be captured by LVDT1 due to its too large distance of about 17.5 mm from the pile wall. The size of this cavity remained almost constant for the conducted tests and thus independent of the flushing flow rate and penetration rate. However, a supplementary test which is not reported in this thesis was carried out with the same flow- and penetration rate as test W-2 but without pile rotation. This test resulted in a noticeable smaller cavity which could indicate that the pile rotation contributed to a closer packing of the sand adjacent to the casing.

The cavity formed during test 0 cannot be explained by effects from either flushing or pile rotation. A likely explanation is that the sand below the pile tip and adjacent to the casing was compacted (i.e. densified) within a zone of higher stress. This is in accordance with pile installation tests reported by White and Bolton (2004). The soil beyond this zone undergoes large shear strains and dilation which may explain the above-mentioned soil surface heave.

Since the flow rate in test W-1 was not able to fluidize the sand completely, some of the soil resistance remained (Figure 6-9). The penetrating pile caused less compaction effects as observed in test 0. Some drill cutting transport through the pile occurred which likely reduced dilation effects and probably contributed to the settlements measured in LVDT1 of test W-1. Test W-6 resulted in about 0.6 mm settlement in LVDT1 but no significant displacement in the other LVDTs. These settlements are likely due to the high flow rate causing considerable erosion and loss of soil volume around the pile which is further discussed below. The other water flushing tests showed negligible soil surface displacements.

Figure 6-13. Local cavity at the soil surface around the pile casing after test W-4.

6.3.4 *Influence on soil resistance*

Cone resistance tests after each pile drilling test were used to assess the impact of different drilling parameters on the soil. Figure 6-14 presents the measured cone resistance at different distances from the pile center (r) against penetration depth for test 0 and the water flushing tests.

The results for a distance r of 90 mm indicate a general trend of reduced cone resistance with increasing flow rate (Figure 6-14 a). This difference is particularly noticeable between the tests W-4 to W-6. For the tests W-1 to W-4, this trend is less obvious, which most likely is explained by the relatively small variations in flow rates. Test W-6 clearly stands out compared to the other water flushing tests with a significantly lower soil resistance from about 200 mm soil depth until the final depth of about 480 mm. The results show an unexpected lower soil resistance after test W-9 compared to test W-8 even though the penetration rate was higher. Based on the load cell measurements (Figure 6-9) it is likely that the high penetration rate (5 mm/s) with test W-9 caused soil displacements and dilation effects which reduced the soil resistance. The same effect could also explain why test W-1 shows less resistance than observed after the tests W-2, W-3, and W-8.

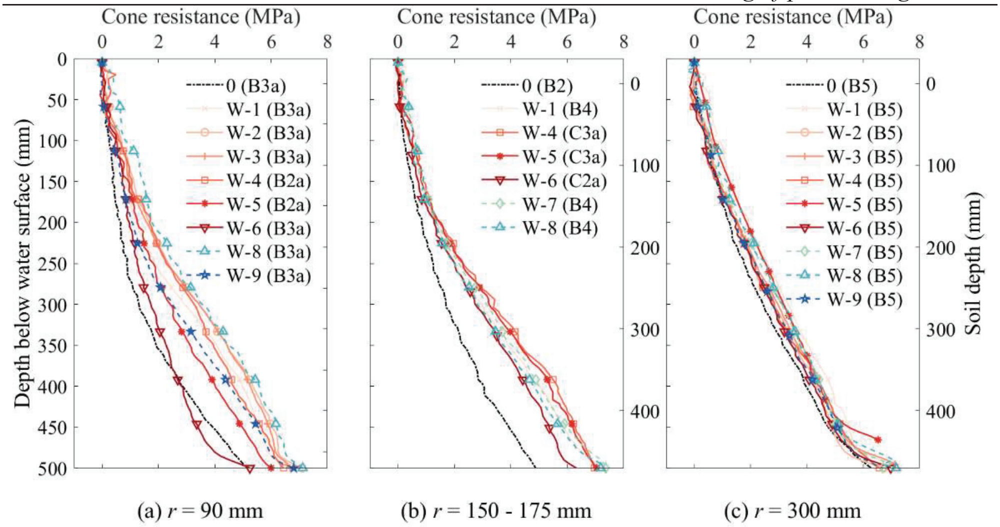


Figure 6-14. Cone resistance against depth for test 0 and tests W-1 to W-9 at a radial distance from pile center of: a) 90 mm; b) 175 mm; c) 300 mm.

Figure 6-14 (b) shows cone resistance from the positions with a distance between 150 to 175 mm from the pile center excluding data for the tests W-2, W-3, and W-9. Due to the greater distance to the pile, the trends observed above diminish. The results do not show an influence from any of the tests at a distance of r = 300 mm (Figure 6-14 c).

An interesting observation is that test 0 caused the lowest cone resistance in the surrounding sand for all tests. The considerable installation effect is clearly visible at both 90 mm (Figure 6-14 a) and 175 mm (Figure 6-14 b) distance from the pile while the impact diminished at a radial distance of 300 mm (Figure 6-14 c). A possible explanation could be that the soil displacements due to the pile penetration without flushing caused large shear strains and volumetric expansion which reduced the soil resistance. This behavior agrees with results from pile tests in sand (White and Bolton 2004) and triaxial tests on Baskarp sand No. 15 showing large dilation angles up to 18 degrees for low stress conditions (Ibsen et al. 2009). This finding is in accord with results from LVDTs and is to some degree also applicable for the tests W-9 and W-1.

Figure 6-15 shows the cone resistance at different radial distance from the pile center before ("pre") and after ("post") the tests W-6 (Figure 6-15 a) and 0 (Figure 6-15 b). As discussed above, test W-6 shows that at r = 90 mm the soil resistance reduced considerably after the pile drilling (B2a vs. B3a, Figure 6-15 a) with a maximum difference of approximately 3.2 MPa at about 400 mm soil depth. The results show a notable influence also at 175 mm from the pile (A3a vs. C2a, Figure 6-15 a) with a maximum reduction in the cone resistance of about 1.1 MPa. For r = 300 mm the difference between pre- and post-test resistance appears to be negligible (B5 vs. B1, Figure 6-15 a). Test 0 shows a similar behavior. However, a greater reduction in the soil resistance due to the pile test can also be seen at r = 175 mm (A2a vs. C2a, Figure 6-15 b). This implies that the radial influence was greater for test 0.

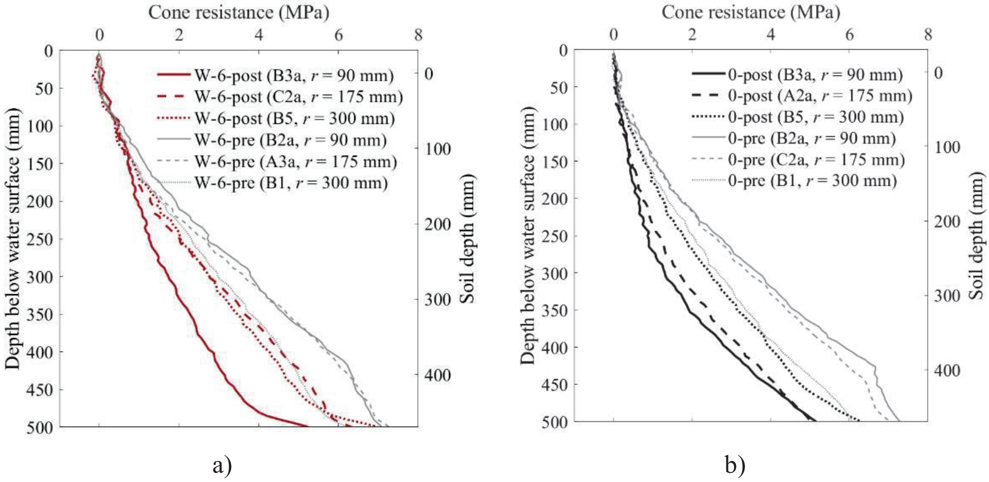


Figure 6-15. Comparison of cone resistance against depth at different radial distances (r) from pile center prior to ("pre") and after ("post"): a) test W-6 and b) test 0.

6.3.5 *Effect of flushing parameters on drill cuttings transport*

An important aspect to understand the mechanism of drilling is to assess the balance between the volume of generated drill cuttings and the theoretical volume of the pile installed. For this reason, the mass of drill cuttings, M_c, which is the sum of the soil collected in the catch pot and in the annulus between the casing and the middle tube of the model pile, was measured for each test. The obtained data indicates that the variations of the flushing parameters considerably affected the mass of drill cuttings. To highlight this finding, non-dimensional parameters of normalized flow, Q_{norm}, and normalized mass of drill cuttings, $M_{c,norm}$, were introduced. The normalized flow is defined as:

$$Q_{norm} = \frac{Q}{A_{pile} \times V_{pen}} \tag{3}$$

where Q is the flushing flow rate in dm^3/min, A_{pile} is the cross-sectional area of the pile in dm^2, and V_{pen} is the penetration rate in dm/min. This dimensionless parameter combines both the flow and penetration rate with the pile area, and therefore provides a simple mean to evaluate the effect of flushing parameters on the drill cuttings transport.

The normalized mass of drill cuttings, $M_{c,norm}$, is defined as the ratio between the mass of drill cuttings, M_c, collected from the pile throughout a test and the theoretical mass of soil, M_{pile}, given by the installed pile volume and the calculated relative density of the respective soil model. This calculation ignores potential drilling induced soil displacements and soil volume changes which is a simplification. A value lower than 1 indicates that the mass of drill cuttings is less than the theoretical one, meaning that the soil is likely replaced by the pile drilling. A value above 1 indicates that the mass of drill cuttings is higher than the theoretical mass, hence causing a potential soil volume loss. A value of 1 is defined as an "ideal" scenario.

Figure 6-16 presents normalized flow rate, Q_{norm}, against normalized mass of drill cuttings, $M_{c,norm}$ (Figure 6-16 a) and the maximum change in soil resistance, q_{lc}, measured by the load cell (Figure 6-16 b). The results show an overall linear trend of increase in normalized mass of drill cuttings with normalized flow rate. The data reveals that an increase in the flow rate caused an increase in the normalized mass of drill cuttings (compare tests W-1 to W-6). By contrast, an increase of the penetration rate reduced the mass of drill cuttings (compare tests W-2 and tests W-7 to W-9). This indicates an inverse correlation between the parameters flow rate and penetration rate.

Since test 0 was carried out without flushing the normalized flow and the mass of drill cuttings was zero. The tests W-1 and W-9 both resulted in a value for $M_{c,norm}$ of about 0.85 with a corresponding value of Q_{norm} just below 10. This indicates that the installation caused some soil displacement surrounding the pile, which is supported by the observed increase in penetration resistance in parts of these tests as can be seen in Figure 6-16 (b) (and Figure 6-9). For test W-1 the penetration rate of 2.5 mm/s means that a soil volume of approximately 0.14 L/min should be displaced by the pile tip, i.e. removed by drilling. The flow rate was about 10 times higher (Q = 1.5 L/min) which is similar values as with test W-9. This indicates that the flow rate needs to be large enough to be able to attain a specific penetration rate, or alternatively the penetration rate needs to be adapted to the flow rate.

Test W-2 (Q = 2.0 L/min) represents an almost ideal scenario for the modelled conditions with $M_{c,norm}$ of about 1.07, only about 7 % excess drill cuttings compared to the installed pile volume was measured. This is in line with the small load cell and pore-water pressure readings observed for this test (Figure 6-16 b) and Figure 6-10). A maximum $M_{c,norm}$ and Q_{norm} value of about 2.7 and 35 respectively was obtained for the test W-6 (Q = 5.0 L/min). This significant loss of soil volume likely explains the settlements observed with LVDT1 (Figure 6-12 a). However, the other LVDTs at greater distance from the pile showed minor settlements. This observation might be related to soil loosening (i.e. reduction in relative density) caused by the high flow rate which likely compensates for the soil volume loss adjacent to the pile. The significant reduction in cone resistance measured after the test (Figure 6-15 a) supports this interpretation. At prototype stress conditions it is likely that the extensive loss of soil volume observed for test W-6 would lead to considerable ground settlements.

The experimental data reveals that a normalized flow rate between 10 to 20 results in an "ideal drilling" in terms of drill cuttings balance, i.e. $M_{c,norm}$ equal or close to 1.0. Compared to prototype drilling with a casing diameter of 76 mm, typical values for normalized flow rate may vary between 35 to 55 with a given flow rate, Q, from 80 to 250 L/min and an assumed penetration rate, V_{pen}, from 500 to 1000 mm/min (8.33 to 16.66 mm/s). These values are notably higher than the obtained "ideal" normalized flow rate. The difference is likely a result of the low stress conditions modelled, and refined investigations are required to translate this framework into practice.

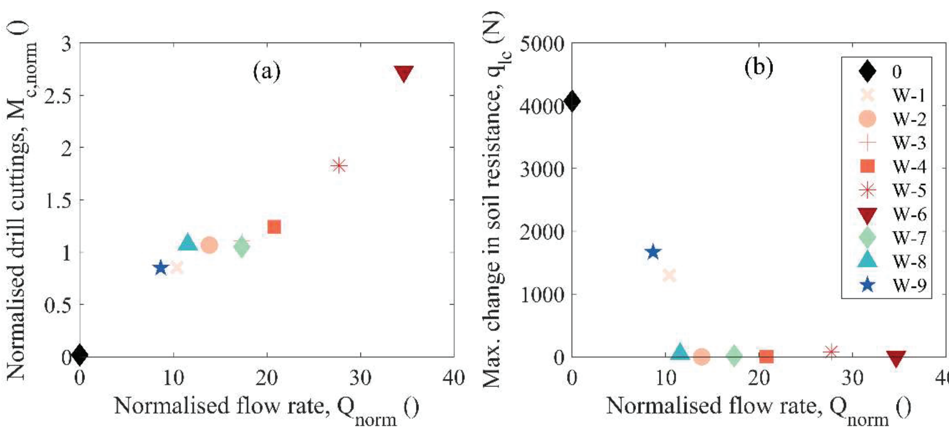


Figure 6-16. Normalized flow rate (Q_{norm}) *against a) normalized mass of drill cuttings* ($M_{c,norm}$) *and b) the maximum change in soil resistance,* q_{lc}*, measured by the load cell.*

6.4 Air flushing tests

Tests carried out with air flushing were generally not able to create a successful transport of drill cuttings. An increased starting soil depth of 400 mm compared to the water tests improved the flushing backflow. At the beginning of the tests A-1 to A-3 small outbursts of water, sand and air were observed coming out at the pile top. However, after about 15 to 20 seconds of drilling the air caused soil fractures and piping (i.e. flow paths) along the outside of the pile wall which continued until the tests were stopped after 100 mm of drilling. This effect has been observed in the field when drilling is carried out at shallow depths or when the flushing pressure is too high (e.g. Lande et al., 2020; Sandene et al., 2021). Due to these challenges the air flushing tests could not be compared with the water flushing tests.

Figure 6-17 presents pore-water pressure changes (Δu) against pile drilling depth for the tests A1 to A3. The results generally display a reduced pore pressure in the surrounding ground which could indicate that the air flushing caused an air-lift pump effect as suggested in case studies (Lande et al. 2020; Ahlund and Ögren 2016; Bredenberg et al. 2014). However, the limited data set makes it difficult to assess and compare results and no clear conclusions can be drawn.

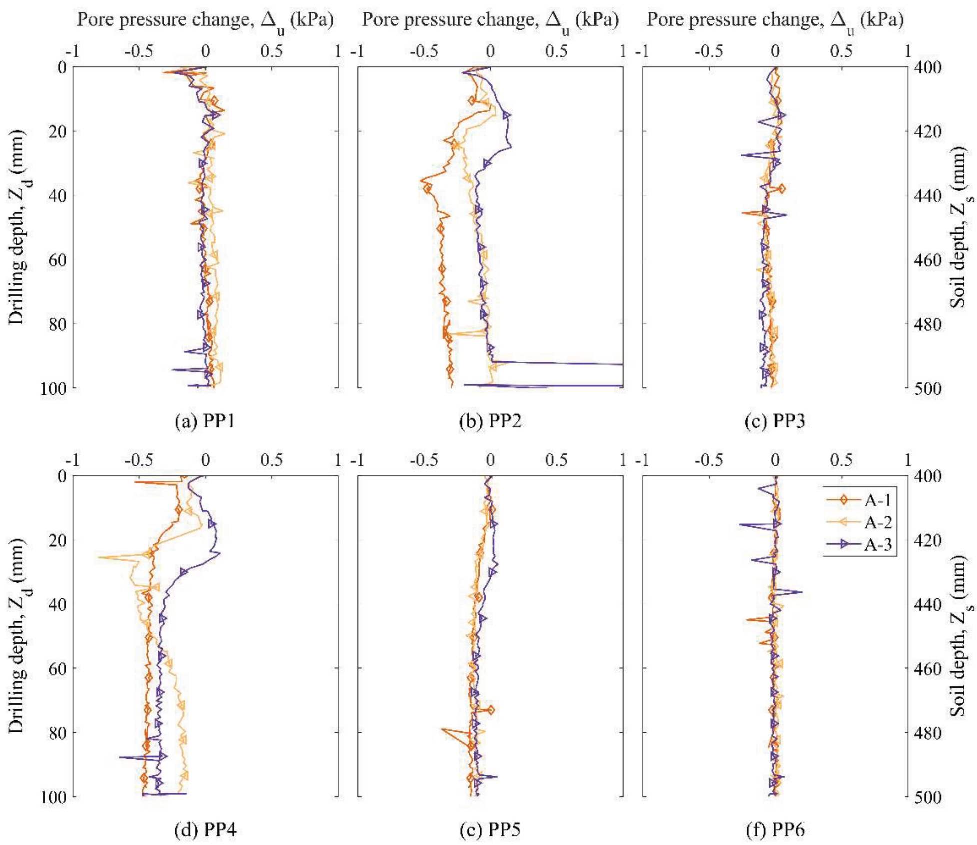


Figure 6-17. Measured pore-water pressure changes against pile drilling depth in PP1 to PP6 for the air flushing tests.

As seen in the Figures 6-17 (a), 6-17 (c), and 6-17 (e) only minor changes were observed in the top PPs (PP1, PP3, and PP6). This is likely explained by the starting depth of 400 mm and the distance from the pile tip to these top PPs. PP2 (Figure 6-17 b) and PP4 (Figure 6-17 d) at 370 mm soil depth experienced the largest pore pressure reductions with maximum values of about -0.6 kPa for test A-1 and A-2 respectively. Test A-3 mainly indicates less influence than the other tests despite the highest flushing pressure (100 kPa).

Results from the load cell measurements presented in Figure 6-18 suggest that the air flushing was not able to fluidize and loosen the sand as the water flushing did (Figure 6-9). In general, an increase in penetration force with depth was observed until the drilling stopped. Only test A-2 showed a reduced resistance from about 45 to 85 mm drilling depth until it increased again. The tests A-1 and A-3 reached a maximum value of approximately 3600 N and test A-2 a value of 2300 N. Since the load cell data display similar trends and penetration force as with test 0, the pore pressure reductions might be related to dilation effects.

Data from the load cell and the observed lack of drill cuttings transport gives reason to assume that the air flushing pressure and flow rate was too low to remove the drill cuttings in front of the drill bit during drilling. Due to the low effective soil stresses sudden piping was observed, and the air pressure could not be further increased. However, after the tests some sand sticking to the inside of the pile casing was detected.

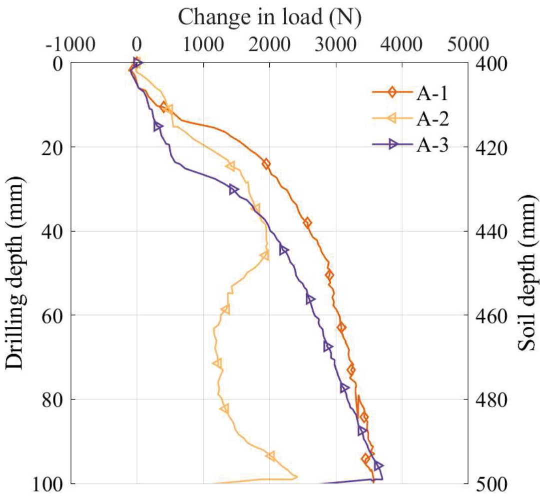


Figure 6-18. Load cell measurements against drilling depth for the air flushing tests.

6.5 **Applicability of results**

Overburden drilling is characterized by very complex simultaneously processes which are carried out in varying ground conditions. For that reason, some simplifications were necessary in the described experiments. All tests were carried out under 1g conditions at low soil stresses, thus having some unavoidable limitations compared to prototype drilling in the field. Consequently, the flushing pressures and flow rates used in the tests were much lower than in a real case scenario making it difficult to compare the normalized flow rate values from the tests directly with prototype drilling parameters. The limited soil depth and soil stress most likely affected the range of normalized flow rate at which an "ideal" drilling scenario was identified (Figure 6-16 a).

Secondly the test set-up did not replicate the details of a percussive hammer which is typically used in drilling to maintain an acceptable penetration rate when drilling in dense granular soils and rock (FHWA 2005; Finnish Road Authorities 2003). The effect of this parameter on the surrounding ground is likely insignificant for the modelled ground conditions.

To further investigate the physical effects from drilling through sandy soils on the surrounding ground and to be able to reliably translate the obtained results into practice, refined model testing including more representative soil stresses, hydraulic conditions (e.g. confined aquifer) and flushing parameters are recommended. Future tests should include higher and more representative soil stresses (e.g. by adding a surcharge) and pore-water pressures representative for drilling at larger soil depth. Higher soil stress would likely require increased flushing parameters (i.e. flow rate and pressure). Other aspects that should be investigated are (1) the effect of drilling in a confined aquifer (e.g. under an impermeable soil layer) as frequently observed in the field (Lande et al. 2020; Ahlund and Ögren 2016; Sandene et al. 2021), (2) the influence of varying soil density, (3) different degrees of saturation, and (4) the impact of different pile rotation rates.

It is expected that refined tests will provide further insight into drilling. Such results in combination with the introduced framework of normalized drill cutting transport and flow rate could lead to practical recommendations regarding a more informed choice of overburden drilling systems and parameters. The introduced framework should be investigated further through full-scale testing to validate its applicability.

6.6 Conclusions

Novel experimental tests revealed the main mechanisms of rotary flush drilling in saturated sand. The impacts from different flushing media (i.e. water and air) and drilling parameters such as flushing flow rate and penetration rate on the penetration force, pore pressure changes, soil displacements, and drill cuttings transport were studied. Based on the results of the water flushing tests, the following conclusions were drawn:

1. Increasing flow rates caused larger excess pore-water pressures and influence area in the surrounding soil. The measured pore pressure changes generally decreased with the distance from the pile. An increased flow rate also generated more drill cuttings and reduced the soil resistance significantly in the soil adjacent to the pile.
2. An increased penetration rate compensated for the effects observed when increasing the flow rate. This observation indicates an inverse relation between these parameters.
3. The drill cutting transportation depends on both the flow and penetration. This finding is highlighted by the almost linear relationship between normalized flow rate, Q_{norm}, and normalized mass of drill cuttings, $M_{c,norm}$.
4. For high normalized flow rates, Q_{norm}, the soil in front of the drill bit fluidized which reduced the penetration resistance. This response is comparable to observations during pile jetting. The fluidization may lead to considerable ground settlements. For the tests with too low flow rate (e.g. W-1) or too high penetration rate (e.g. W-9), opposite behavior was observed, and the soil resistance increased.
5. The introduced framework of normalized flow rate, Q_{norm}, and normalized mass of drill cuttings, $M_{c,norm}$, could provide a first effective mean to derive ideal drilling parameters. The experimental data reveals that a normalized flow rate between 10 to 20 results in an "ideal" drilling in terms of drill cuttings balance, i.e. $M_{c,norm}$ equal or close to 1.0. The effect of more representative soil stresses is, however, an area that requires further research.

The air flushing tests were limited by modelling constraints; thus, no clear conclusions can be drawn from these tests. However, a notable reduction of pore pressures adjacent to the casing was measured. This finding may indicate that air flushing causes a behavior equivalent to an air-lift pump effect which could lead to considerable erosion, soil loss and resulting ground movements. Similar observations were reported in case studies (Lande et al. 2020; Ahlund and Ögren 2016; Bredenberg et al. 2014).

The presented experimental data provide a new insight into the mechanisms of overburden drilling on the surrounding ground. Refined model tests were recommended which should focus on more representative stress conditions and flushing parameters. In addition, full-scale tests should be explored to further assess the introduced framework of normalized drill cutting transport and flow rate to evaluate the obtained data from the model tests.

The main findings from supplementary refined model tests are briefly summarized in the following section. Further, results from case studies of full-scale drilling are presented and assessed in section 7. One of the cases included drilling with a DTH water hammer and made it possible to assess the framework of normalized flow rate and drill cuttings transport.

6.7 **Refined model tests of pile drilling under realistic stress conditions**

Refined model tests for simulating pile drilling in sand under realistic stress conditions were carried out at the geotechnical lab at NTNU as part of the M.Sc. Thesis of Tyvold (2020). A detailed description of the modified test set up, including the model concept, soil preparations, instrumentation, and test procedures, as well as test results are presented in the thesis. The model had the option of increasing the total stresses and pore pressures in the sand to a desired amount, and the reported tests were carried out under stress conditions simulating a soil depth of 20 meters. Figure 6-19 shows a cross section of the model set up including the model tank (cylindrical), the reaction frame and custom-made piston and pressure plate to supply the desired compressive force, and hence, the stresses in the sand. The same load cell, linear actuator, rotation motor, and model pile as in the above reported tests were used. The soil model became very dense, reaching a relative density D_r, of about 1.0 (100%), which is significantly higher than the original model.

The tests used a variation of flushing flow rates and penetration rates to examine the effects on the penetration resistance, pore pressure and drill cuttings transport like the above reported tests. A large part of the study was used to improve the set up, and only 6 of the total 31 tests were considered reliable. Tyvold concludes that the successful series of tests were used to observe the effects of increased flow rate and normalised flow rate on the total drill cuttings transport. It was to some extent able to verify the expected scenarios of increased transport with increased flow, followed by a limit where no transport is achieved, as shown by the yellow symbols in Figure 6-20 presenting normalized flow rates (Q_{norm}) against normalized mass of drill cuttings ($M_{c,norm}$) for all tests that were considered successful. It did however not achieve severe "over-drilling" because of high flow rates, as the limit was reached earlier than expected. The tests with blue symbols were carried out with an air pressure valve and return tank with too low capacity which affected the pore pressure readings making the analysis of drilling effects difficult, and the results uncertain.

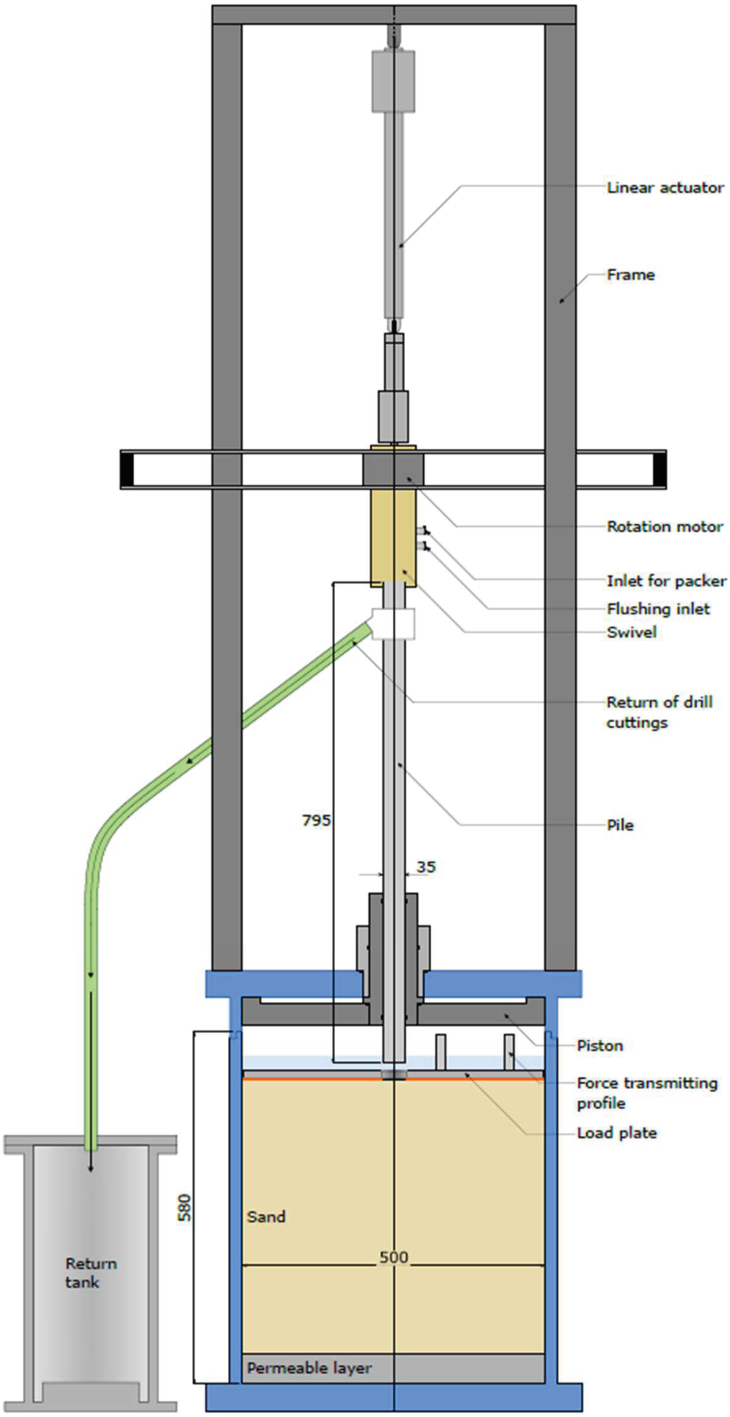


Figure 6-19. Illustration of refined model test set up (from Tyvold 2020). Units in mm.

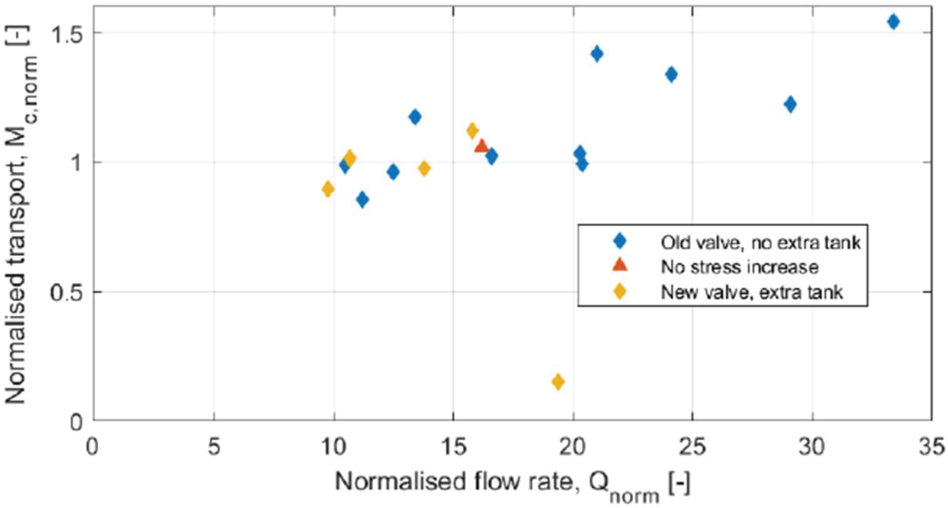


Figure 6-20. Normalized flow rate (Q_{norm}) *against normalized mass of drill cuttings* ($M_{c,norm}$) *for the successful model tests (from Tyvold 2020).*

The parameters that resulted in successful installations coincided well with the tests described in section 6. This included similar flow rates, penetration rates and a relative flushing pressure in the same magnitude. The absolute flushing pressure was however higher, to counter the back pressure of the system. Furthermore, the influence of these parameters on pore pressure levels and drill cuttings transport were also in the same magnitude. This indicated an independence from the stress conditions. Presuming that the refined model set up provided realistic stress conditions, Tyvold suggested that the deviations between full-scale prototype drilling and the above reported model tests were in fact not mainly due to their low stress conditions.

Although the modified test set up performed well, there were some aspects and components that should be improved to increase the functioning of the model and the quality of the results. This included having higher capacities of the air pressure valve controlling the pore pressure in the model, as well as the actuator and rotation motor for the pile, allowing pile installations under lower normalized flow and with a larger soil resistance. Improved instrumentation including an extensometer on the reaction frame would also provide more accurate information regarding the pile penetration rate during the drilling. The possibility to measure the soil resistance would help verify the soil homogeneity before drilling, and to assess the effects of drilling. And lastly, the model set up should include the possibility to perform compaction of the soil after the pre-installation of the pile.

7 Case studies of overburden drilling of foundation piles

7.1 Introduction

As part of the research projects *Limiting Damage* and *Risk Reduction of Groundwork Damage,* several construction projects involving overburden drilling of foundation piles have been studied. Results from instrumentation and monitoring have provided new insights and documented installation effects in a way and scale that has not been done before. This section gives a short description and presents the main results from the case studies.

7.2 Case 1

7.2.1 Project description

This project involved a 290 m long, and 28 m wide concrete bridge founded on nine end-bearing pile groups. Figure 7-1 shows a longitudinal profile of the bridge and the pile foundations. All piles were installed by drilling a continuous permanent casing (OD = 711 mm, 12.5 mm thickness) through the soil and with 1.5 m embedment into bedrock. The casings were reinforced and cast with concrete after the drilling was completed and the borehole was cleaned. The following sections focus on the drilling of pile groups 3, 4, and 5.

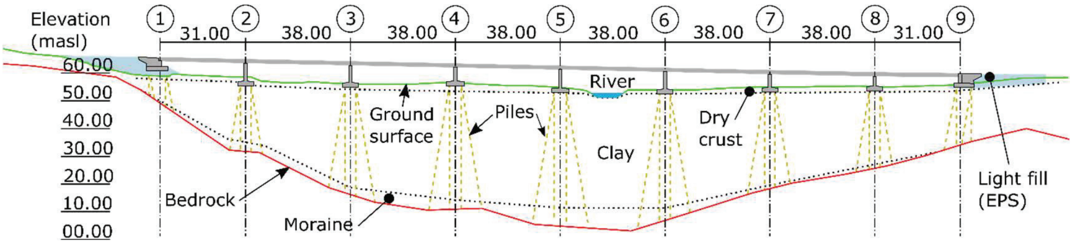


Figure 7-1. Longitudinal profile of the bridge founded on nine pile groups. Bedrock and soil stratigraphy based on ground investigations and pile drilling logs. Units in meter.

7.2.2 *Ground conditions*

Ground investigations (Rambøll Norway, 2012) indicate a thin layer of organic topsoil over 2-4 m of dry crust and about 7-48 m of medium stiff to stiff marine clay. Above bedrock a 1-9 m thick layer of compact glacial till with silty and sandy material was detected. The depth to bedrock varied from 10-55 m (Figure 7-1).

Laboratory investigations on undisturbed clay samples showed water contents (w) between 17-25% and saturated unit weights (γ) between 17.1-18.8 kN/m^3. Interpreted cone penetration tests (CPTu) showed that the undrained active shear strength (c_{uA}) in the clay increased from 40 kPa at 4 m depth to approximately 100 kPa at 40 m depth. Rotary-pressure soundings indicated that the clay is highly sensitive in the deeper parts just above the till or bedrock, but also in "pockets" within the clay deposit.

Pore-water pressure data showed that the ground water level typically varied from 1 m below ground surface by the river to about 4 m below ground at the bridge abutments where the ground elevation was 4-6 m higher than by the river. Piezometers installed in the till showed an artesian pressure approximately 15% higher than the hydrostatic pressure.

7.2.3 *Drilling method and procedures*

The rotary percussive duplex method with a DTH air hammer and a concentric drill bit was used, as shown in Figure 7-2. The five holes in the center of the pilot bit indicate where the compressed "exhaust" air exits after passing through the DTH hammer. The exhaust air guides the drill cuttings to channels between the casing and the pilot bit (red arrows in Figure 7-2) and transports them to the surface through the annulus between the casing and drill rod.

Typical air flushing pressures between 5-10 bars were used when drilling through clay, and 12-20 bars when drilling through till and into bedrock. Water flushing with a typical pressure of 10 bar was also combined with air flushing, to loosen the soil. This was followed by air flushing to remove the drill cuttings and water from the borehole. The water flow rate (Q) varied between 250 L/min during drilling through clay and up to about 350 L/min when drilling through the till. The penetration rate (V_{pen}) was generally kept between 70-100 cm/min through clay. Due to the high drilling resistance in the till, the penetration was mostly between 10-15 cm/min but in some cases as low as 3-4 cm/min. A rotational speed of 3-4 revolutions per minute (rpm) was used through the different soils.

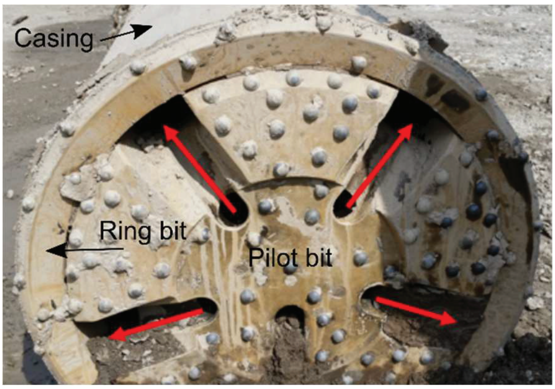


Figure 7-2. Concentric pilot and ring bit for a 711 mm diameter casing. (Photo by Einar John Lande)

7.2.4 *Instrumentation*

Figure 7-3 shows that five Borros type settlement anchors (Geokon, 2020) and four electrical piezometers (PZ) were installed adjacent to pile group 4. PZ1, PZ4 and anchor S5 were installed at the top of the till layer, while the other instruments were in the clay. Table 7-1 shows the depths of each instrument.

Vertical displacements were measured on top of each anchor rod using a total station theodolite. The accuracy of the measured values was approximately ±5 mm. Settlements were monitored regularly over a total period of about 9 months, starting about six weeks prior to the drilling of pile group 4 and lasting about six months after drilling in pile group 4 was completed. The measurements were generally carried out between 2-4 times each week during pile drilling in pile group 5, 3, and 4 to be able to document immediate effects. After drilling was completed in pile group 4, the subsequent measurements were carried out with longer time intervals (between 1-4 times each month) and only on anchor S4 and S5 since the others were destroyed by an excavator.

The piezometers were installed at the same time as the pile drilling started in pile group 5 which was approximately 8 weeks before the drilling started in pile group 4. The sensors had a general logging frequency of one-hour over a total period of 4 months, except PZ4 which logged data every 30 minutes during the period of drilling in pile group 4.

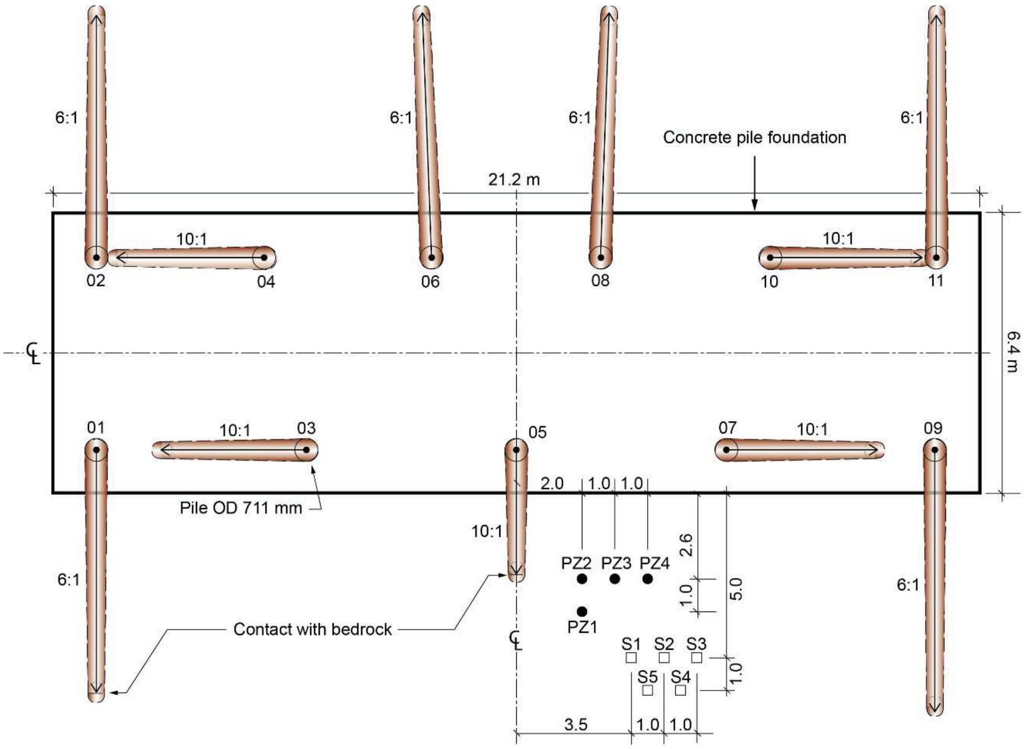


Figure 7-3. Foundation 4 with 11 piles (inclined 6:1 and 10:1) including settlement anchors (S1-S5) and piezometers (PZ1-PZ4).

Table 7-1. Instruments installed adjacent to pile group 4. (S = settlement anchor and PZ = piezometer).

Instrument	S1	S2	S3	S4	S5	Z1	Z2	Z3	Z4
Depth [m]	16	26	31	36	41	43	36	36	42

7.2.5 *Influence on pore-water pressure*

Figure 7-4 presents pore-water pressure changes (ΔU) against time during drilling of pile groups 5, 3, and 4. The pressure changes were related to the reference pressures (u_{ref}) at the given soil depth, based on measurements prior to drilling. The reference pressures for PZ2 and PZ3 installed in clay were chosen after the excess pressure from installation had dissipated and the pressure stabilized (approx. 1 week after installation).

The largest pore pressure changes were measured when drilling closest to the piezometers. A general trend of sudden pore pressure reductions was observed in the piezometers PZ1 and PZ4 when drilling through the till. Within a short time (1-6 hours) after drilling stopped, the pressure increased again. However, the pore pressure remained lower than before drilling of pile groups 5 (Figure 7-4 (a)) and 3 (Figure 7-4 (b)). Overall, a pore pressure reduction of about 25 kPa was observed at the top of the till layer.

The piezometers in clay (PZ2, PZ3) did not show any sudden pore pressure changes when drilling pile groups 5 and 3. After the excess pore pressures from the installation had dissipated (about 20th August), the pressure decreased gradually to about 5 kPa below the reference value during drilling in pile group 5. The pore pressure remained unchanged during drilling of pile group 3.

Figure 7-4 (c) shows that drilling of pile group 4 caused the greatest impact due to the close distance to the piezometers. Sudden pressure reductions were registered in both the till and the clay. The pre-drilling pressure in the till typically recovered within a few hours after drilling, indicating a high hydraulic conductivity and recharge of ground water in the till. The clay showed a delayed response. After drilling of piles 05 and 09, the pore pressures in PZ2 and PZ3 did not recover completely and were approximately 20 kPa lower than the reference pressure when the drilling was completed in pile group 4. Five weeks later the pressure remained 7-9 kPa below the reference pressure.

Due to the small distance to the piezometers (min. 1.7-2.4 m from pile 05 to PZ2 and PZ1), the most severe effects were observed when drilling for piles 05 and 09. Maximum pressure reductions of about 140 kPa (PZ2) and 115 kPa (PZ3) were registered in the clay, and about 180 kPa (PZ1) and 105 kPa (PZ4) in the till. The data also show rapid shifts between pressure reductions to pressure increase when drilling in the till for pile 05, 11, and 08 with a maximum excess pressure of 52 kPa registered in PZ1 during drilling of pile 05. These shifts were likely related to alternating water and air flushing. The logging frequencies of 30 and 60 minutes probably explains why these variations were not observed for all piles and that some piles located close to the piezometers (e.g. piles 01, 03) showed less impact than others further away (e.g. piles 08, 11).

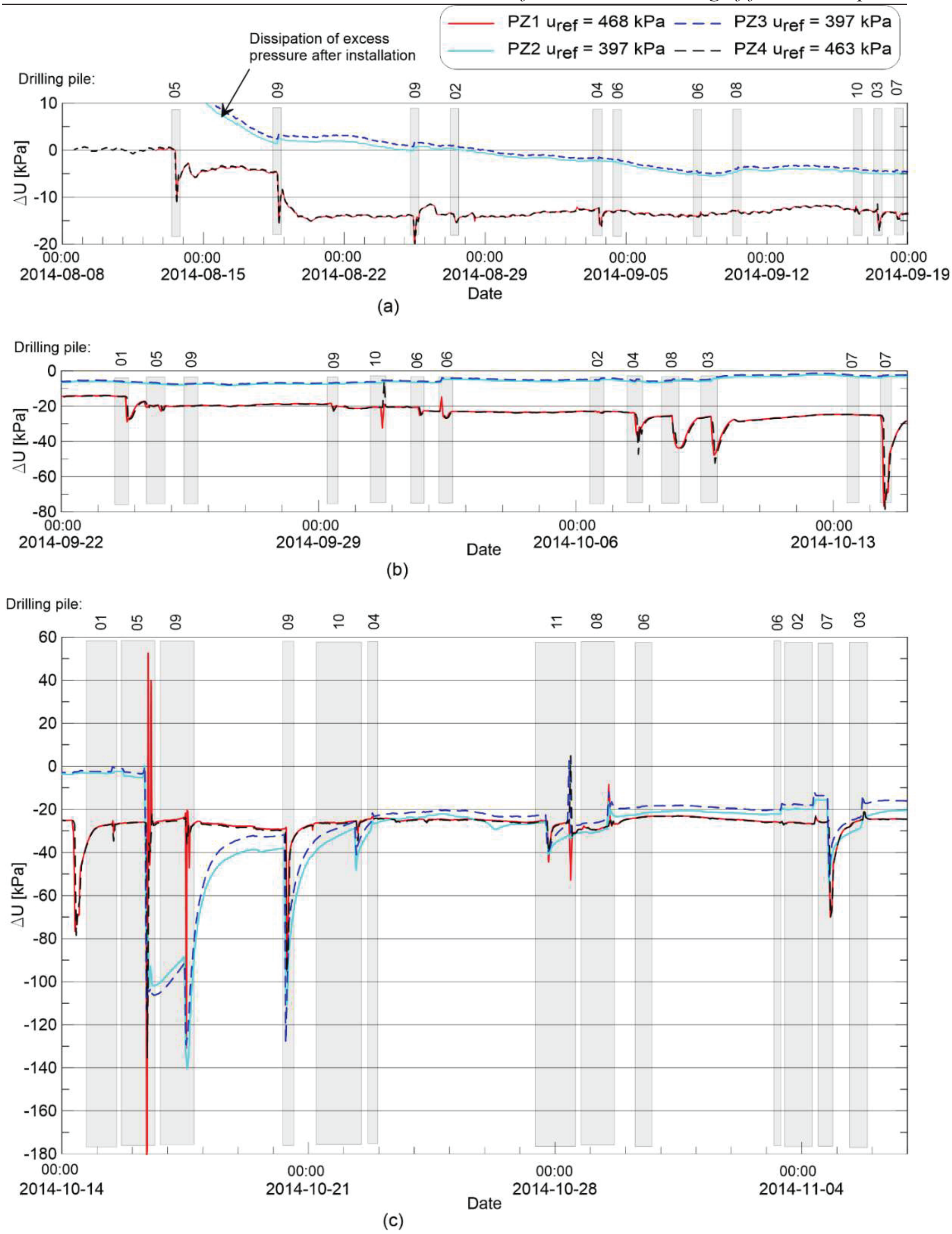


Figure 7-4. Pore pressure changes (ΔU) measured in piezometers installed adjacent to pile group 4 during pile drilling in: (a) pile group 5; (b) pile group 3; (c) pile group 4. The drilling time for each pile is marked with grey rectangles.

7.2.6 *Influence on ground settlements*

Figure 7-5 presents vertical ground displacements, δ_v, against time for the settlement anchors adjacent to pile group 4. The reference measurements were conducted on the 1st of September 2014, about 3 weeks after the drilling was started in pile group 5 and went on until 4th May 2015 (Figure 7-5 (a)). The data show a general fluctuation explained by ongoing ground works affecting the measurement accuracy. This probably explains the small heave (about 0 to 10 mm) registered during drilling of pile groups 5 and 3.

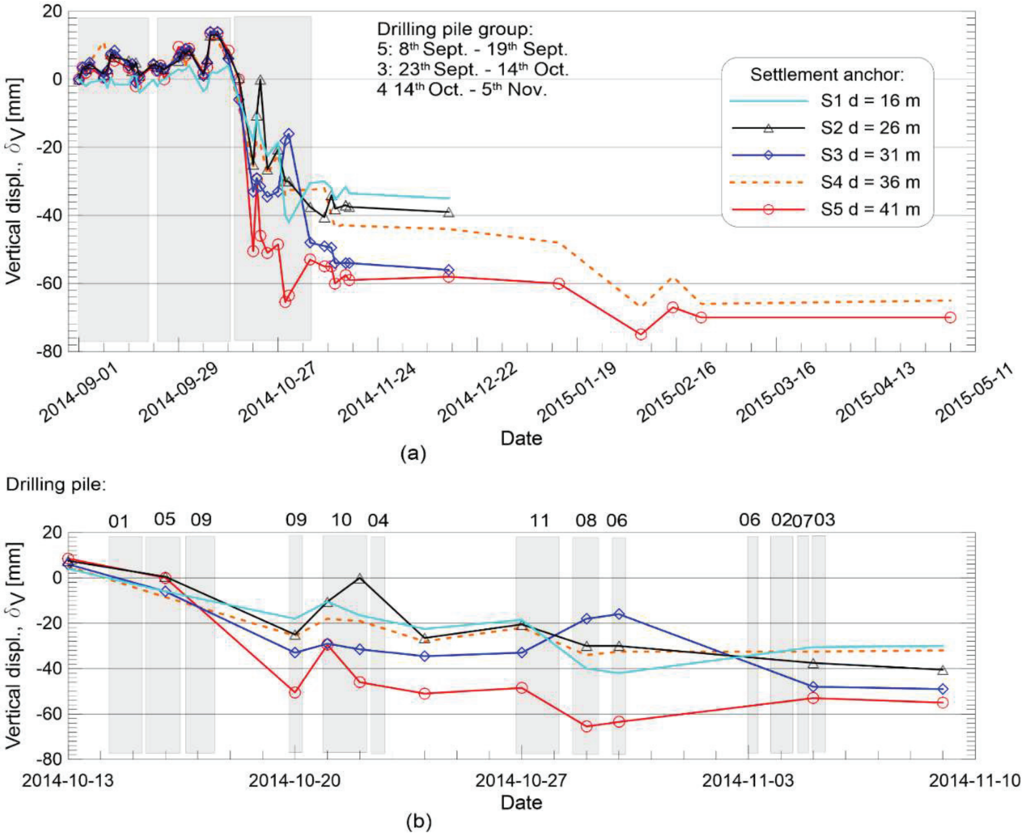


Figure 7-5. Vertical displacements (δ_V) of settlement anchors S1 to S5 for: (a) the entire surveying period; (b) during drilling in pile group 4. The drilling time is marked with grey rectangles. Negative values represent settlements.

Drilling of pile group 4 caused considerable ground settlements. Anchor S5 installed in top of the compact till layer at 41 m depth generally showed larger accumulated settlement than the shallower anchors. The greatest impact was registered when piles 01, 05, and 09 were drilled, as shown in Figure 7-5 (b). During this period (14th to 20th October) anchor S5 settled 45-50 mm while the other anchors settled between 15-30 mm. Despite some variations in the measurements, the settlements increased when drilling the remaining piles in pile group 4 and reached a maximum value between 55-60 mm in S5 and a minimum value of about 30 mm in S1. The minor settlements in S5 when drilling the piles at greater distance from the instruments indicates that settlement anchor S5 was "outside" the assumed influence zone from most piles. The immediate settlements in S5 imply that drilling in the till caused extensive soil volume loss. The smaller settlements in S1 to S4 indicate a delayed response in the clay.

Three months after drilling of pile group 4, additional settlements of approximately 10 and 25 mm were observed at anchors S5 and S4 (Figure 7-5 (a)). Later measurements indicated that the settlements subsequently stopped. The anchors S1 to S3 showed small settlements (0-5 mm) about six weeks after drilling was completed. At that time these anchors were accidentally destroyed by an excavator.

7.3 **Case 2**

7.3.1 *Project description*

This case involves the construction of a large building complex close to Oslo, Norway. The construction involved an approximately 4 m deep excavation within a 15-24 m deep sheet pile wall (SPW) supported by one level of horizontal tieback anchors connected to a secondary anchoring wall (5-8 m deep), as illustrated in Figure 7-6. After excavation to the final level, an approximately 0.2 m thick reinforced concrete slab was cast to support the SPW and act as a working platform for drilling more than 1,000 end bearing steel core piles with diameters between 150-230 mm. The steel cores were installed in permanent casings, which were drilled a minimum length of 1.0 m into bedrock using rotary percussive DTH drilling systems. More than 350 of the piles were installed within the building pit for the basement, which has a total footprint of about 13,000 m^2.

Figure 7-6 (a) presents a layout of the building pit including the pile positions in the central part. The building pit was divided into areas A-I; area E is discussed in detail below. Figure 7-6 (b) shows a cross-section through the areas E and F which visualizes the ground surface, soil layers, bedrock surface, SPW and positions of piezometers.

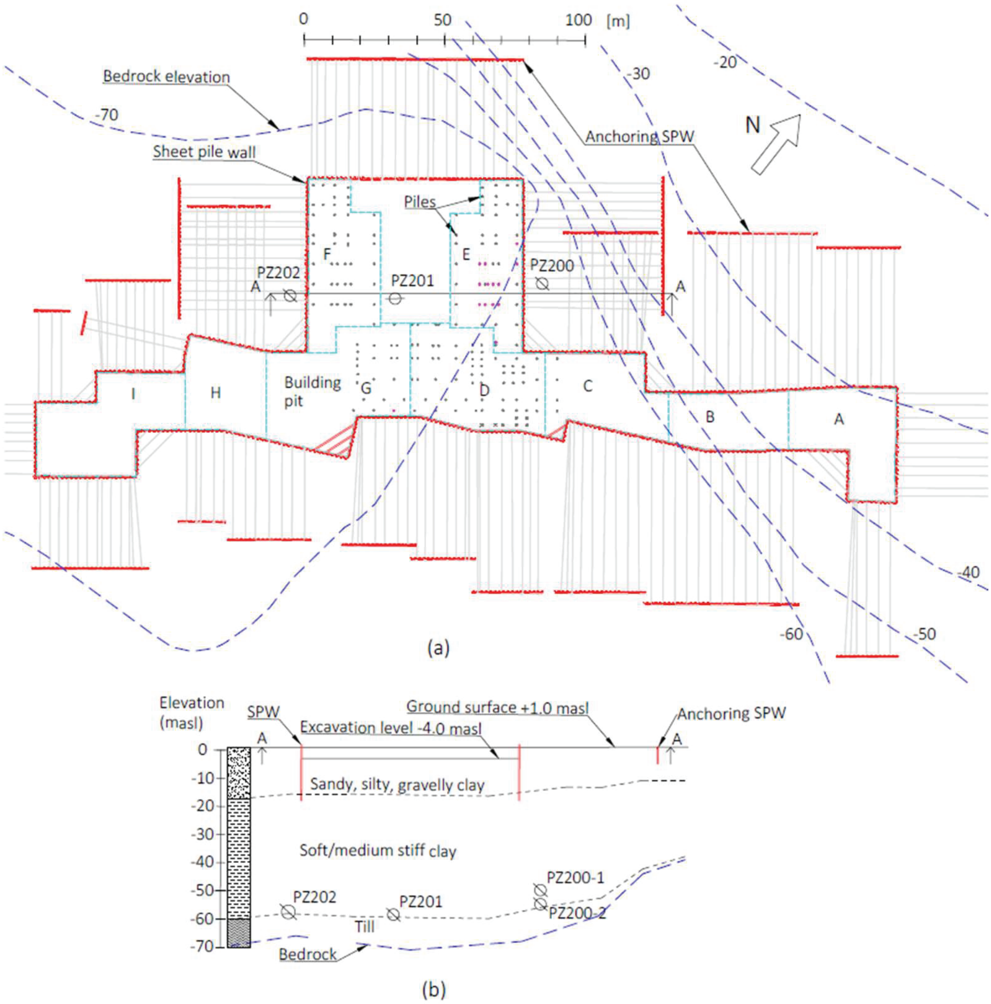


Figure 7-6. Overview of building pit: (a) layout with sheet pile wall (SPW) with tie-back anchoring system and (b) cross section A-A through area E and F. Area E indicates the area of interest for this study.

7.3.2 *Sequence of events*

The excavation and casting of the concrete slab was completed in May 2020. The pile drilling started on the 29th of May in Area A (Figure 7-6 (a)) where the thickness of the glacial till and depth to bedrock was modest. A DTH air hammer was used. The drilling continued further west to the deeper central part, starting in Area E on the 13th of July. In the beginning of September, considerable displacements occurred, causing some local damages on the SPW and anchoring system at the corner of Area E and Area D. The pile drilling continued in the western areas F, G, H, and I under close surveillance. However, on the 30th of October the pile drilling had to stop due to major ground displacements affecting the SPW, concrete slab and the pile casings. Alternative drilling methods were considered, and it was decided to carry out test drilling using a water driven DTH hammer. The test program involved drilling of 13 casings: nine with an outer diameter of 273 mm and two with 324 mm and 406 mm diameter.

7.3.3 *Ground conditions*

The ground elevation varies between +1.0 to +1.5 masl. Ground investigations mainly consisted of rotary pressure soundings to bedrock, cone penetration tests (CPTu) and core sampling (72 mm diameter) to a maximum soil depth of 27 m. The investigations showed layers of sandy, silty, and gravelly clays to depths between 10-15 m followed by a more homogenous soft to medium stiff silty marine clay deposit extending to about 60 m depth. Laboratory tests on undisturbed core samples showed that the clay deposit had sensitivities mostly between 5 and 20. Below this clay layer, rotary-pressure soundings indicated a compact layer of glacio-fluvial till (silty, sandy, gravelly soil) with thickness ranging from 5-15 m above bedrock. The bedrock depth varied from less than 30 m in the eastern area to more than 70 m in the central and western area (Figure 7-6 (b)).

The ground water table coincides with the sea level (i.e. 0 masl). Measurements showed an approximate linearly increasing artesian pore-water pressure with depth, and at the top of the till layer (i.e. a depth of approximately 60 m), the total head was 10 m higher (100 kPa) than the hydrostatic pressure.

7.3.4 *Drilling method and procedures*

Initially, a similar drilling system as in Case 1 using DTH air hammer and a concentric pilot- and ring bit was chosen. A so-called "telescope" solution was also adopted, see illustration in Figure 7-7. This involved first drilling a 406 mm diameter casing. For situations where the 406 mm casing could not be drilled through the compact till, a smaller casing (323 or 273) mm was then drilled deeper inside the 406 mm casing and into bedrock.

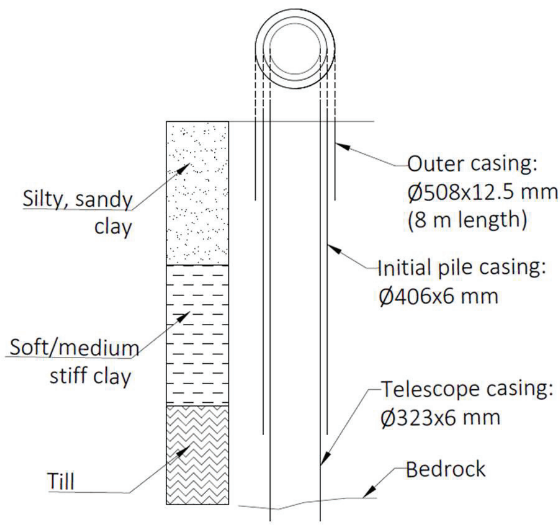


Figure 7-7. "Telescope" pile solution including an outer casing. Illustration not to scale.

Drilling through the clay deposit was mainly carried out with rotation and combined air and water flushing. A moderate amount of added water (Q ~ 100-200 L/min) loosened the clay which was then lifted to the surface by compressed air. A moderate penetration rate of about 100 cm/min was used to limit soil displacements in the clay. When the till layer was reached, the air pressure was increased to 12-15 bars to drive the DTH hammer. The drilling penetration rates in the till were low, down to 2-3 cm/min at the lowest. It was observed that the drill cuttings contained a considerable amount of natural rounded (i.e. uncrushed) stones from the till deposit which implies that the hammer did not perform as intended. This could be related to the high artesian pore-water pressures measured in the till (u_{ref} = 560-610 kPa) combined with significant ground water flow into the casings. This reduced the effective air pressure driving the percussive hammer as described by Halco Rock Tools (2021).

The later test drilling with the water hammer utilized a double head rotary drill rig. This enabled the casing to be rotated separately from the drill rod and the concentric pilot bit. Drilling through the clay deposit was carried out with a penetration rate between 1.2-4.4 m/min with an average of 2.2 m/min. The water flow rate varied between 380-1100 L/min for the Ø406 mm casings with an average value of approximately 700 L/min. In the till layer, substantially higher penetration rates (i.e. 0.25-1.37 m/min with an average of 0.9 m/min) were achieved compared to the air hammer.

7.3.5 *Instrumentation and monitoring*

Prior to the basement excavation, geodetic surveying points were established on top of the SPW and the secondary support wall. The measuring frequencies varied from a few days to a few weeks during the excavation work and were not followed up on a regular basis during the first seven weeks of pile drilling. When the large displacements occurred, supplementary surveying points were established on the SPW around the building pit and on bolts fixed to the concrete working platform cast directly on the ground. Prior to the test drilling, additional survey points were also established on top of 7 of the outer 508 mm casings in Area E. The monitoring frequency was increased to better assess the influence from the remaining pile drilling.

In addition to the surveying points, 10 electrical piezometers were installed. Five were installed at the top of the till layer at soil depths between 53-59 m, while five were installed in the clay at depths between 15-50 m. Figure 7-8 shows in more detail the locations of the piles in areas D, E, F and G, the location of the surveying points and piezometers in the central part of the building pit.

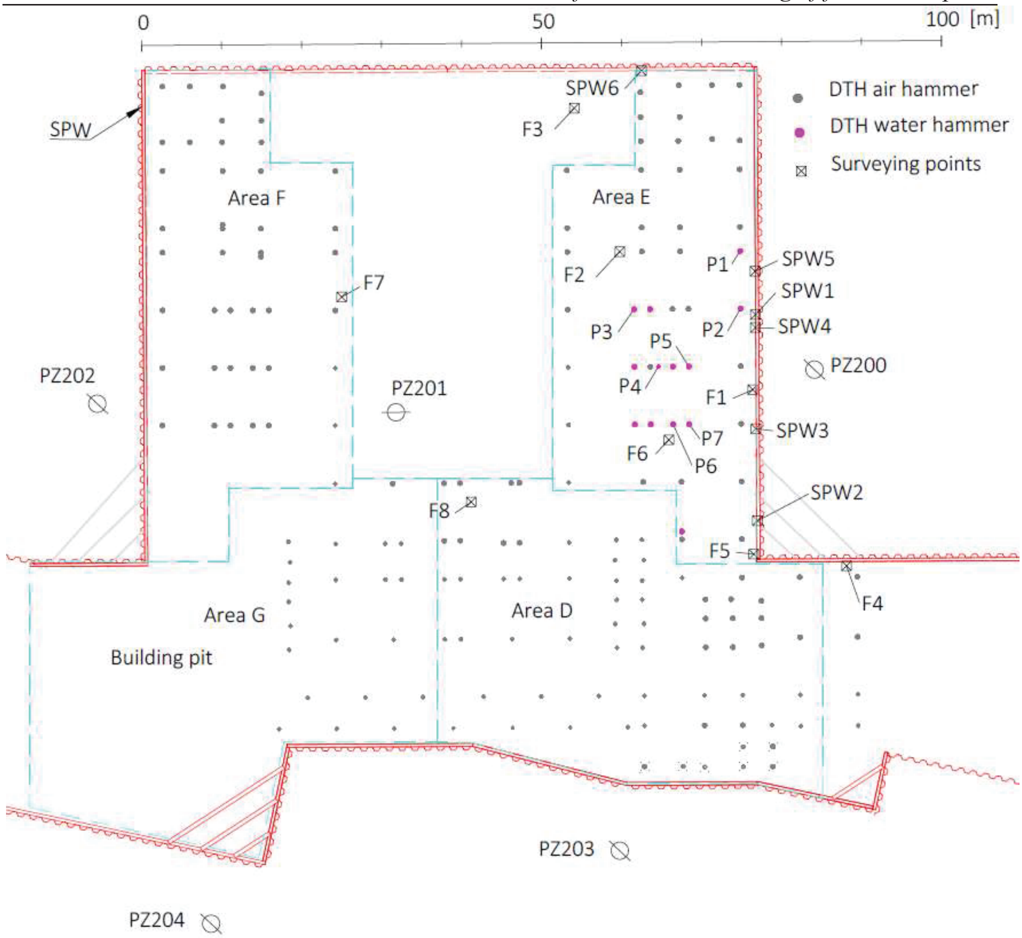


Figure 7-8. The central part of the building pit including the foundation piles, geodetic surveying points and piezometers.

7.3.6 *Ground settlements*

Figure 7-9 presents vertical displacements (δ_v) against time for the surveying points located in Area E. As can be seen from Figure 7-9 (a) the two initial surveying points on the SPW (SPW1 and SPW4) settled about 250 mm at the time the above-mentioned damage was observed on the SPW (2nd of Sept.) The settlements continued with varying rates and reached approximately 500 mm before all air hammer drilling stopped on the 30th of October. The varying settlement rates could be related to drilling activities in different areas. The supplementary point F1 on the concrete slab which was closest to the points SPW1 and SPW4 showed similar settlement rates, while the points F3 and F4 located further away settled less. An immediate reduction in the settlement rates was observed when the drilling stopped. Some ongoing settlements could be related to a delayed response in the clay deposit overlying the glacial till layer.

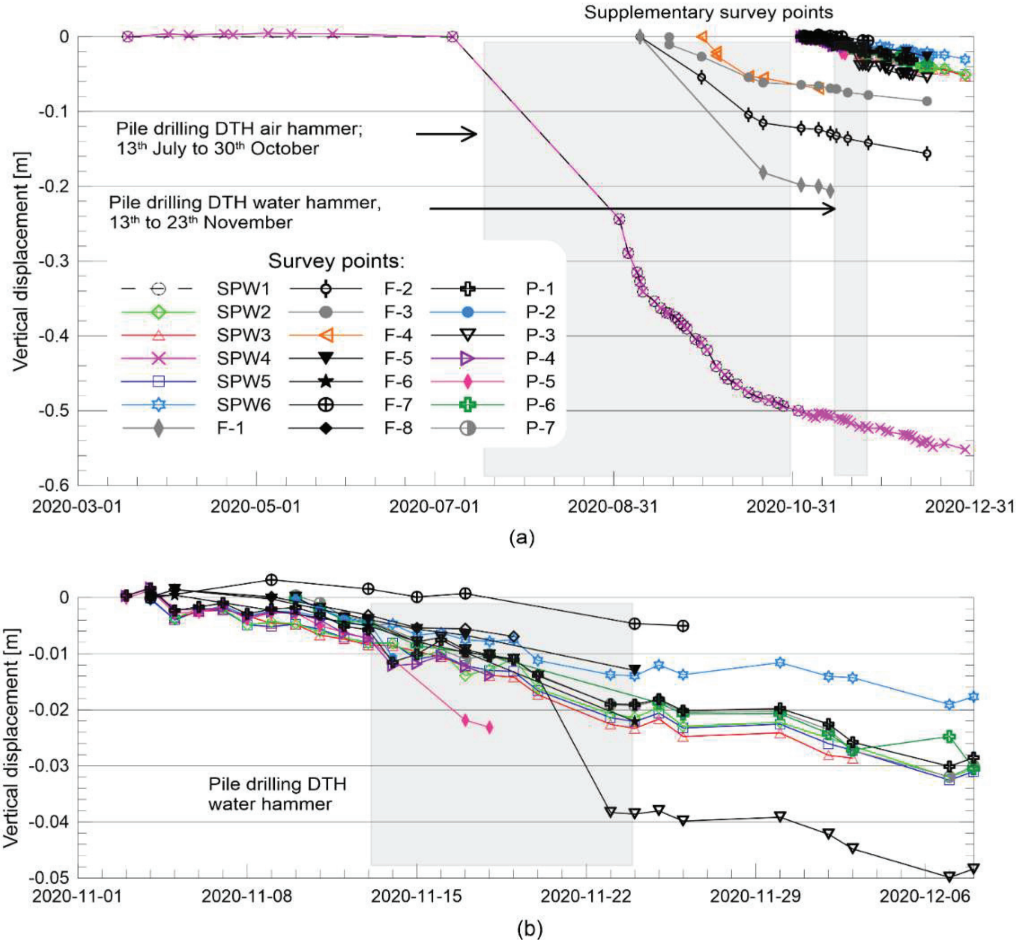


Figure 7-9. Vertical displacements against time on surveying points on top of the sheet pile wall (SPW), concrete slab (F) and 508 mm diameter outer casings (P).

The monitoring data and drilling protocols suggest that the substantial and immediate displacements were caused by two main effects:

1) erosion and loss of soil mass due to the air-lift pump effect when drilling in the till layer (as for Case 1)
2) hydraulic ground failure into the bottom of the 406 mm casings.

About 30% of all 406 mm casings drilled with the air hammer stopped in the till layer, thus requiring the telescope solution to reach bedrock. In many of these casings, hydraulic ground failure occurred at the bottom when the drill rod and pilot bit was pulled up, hence partly filling the casings up with fine grained soil from the till before drilling of the smaller casings continued. Measurements showed a maximum of about 13 m of soil at the bottom of the casing (equals approximately 1.55 m^3 of soil). The local ground conditions with artesian pore-water pressures and ground water flow (i.e. recharge in the aquifer) combined with the low drilling penetration rate probably enhanced the erosion effect from the drilling, and caused problems with hydraulic uplift at the bottom of the casings.

After having observed the above-described negative effect of the drilling works, the drilling procedure was changed to reduce further impact. The updated procedure included to stop drilling with the 406 mm casings if the penetration rate in the till became lower than 4-5 cm/min. The casings were also filled to the top with water before removing the drill rod to reduce the risk of hydraulic failure. Despite these changes the ground displacements continued to develop.

Figure 7-9 (b) presents the measured vertical displacements on the supplementary survey points. The points denoted P1 to P7 were located on top of the outer 508 mm diameter floating casings. The data indicates ongoing settlements between 4-8 mm over a period of ten days prior to the test drilling. Survey point F7 remained stable, likely due to the greater distance from the area most affected by the previous air hammer drilling.

During the test drilling, the settlements continued at nearly the same rate as prior to the test. For point P3 and P5, immediate and significantly larger settlements were measured compared to the other points. This was caused by the drill rig accidentally running over and pushing the outer casing (Ø508 mm) into the soft clay. When the test drilling was completed, the settlements stabilized for about a week before they continued with a rate of 5-6 mm/week. The points F7 and SPW6, which were furthest from the test piles, showed the least settlements but followed the same long-term trend as the other points. Monitoring point SPW4, which covered the entire period of drilling with both air and water hammer, showed similar settlement rates as the supplementary points located in close vicinity to the SPW during and after the test drilling. Overall, the results clearly show a minor influence on the surrounding ground when drilling with the water hammer compared to when drilling with the air hammer.

7.3.7 *Pore-water pressure response*

Figure 7-10 presents measured pore-water pressure (U) in the central part of the building pit against time. Locations of the piezometers are shown in Figure 7-8. The piezometers were installed to the top of the till layer.

The air hammer drilling caused sudden and temporary pressure reductions in PZ201 and PZ202. A maximum change of about 15 kPa in PZ201 (29th of October) was registered when drilling more than 50 m south from the piezometer. PZ202 and PZ203 were not affected despite being located closer to the pile. This was explained by the piezometers likely being installed in clay with a lower hydraulic conductivity than the underlying till. The observed pressure reductions support the hypothesis of air flushing causing an air-lift pump effect. Since the piezometers were installed just a few days before drilling with the DTH air hammer stopped, the results do not provide a complete picture of the influence from air drilling.

The water hammer drilling increased the pore pressure by about 5 kPa in PZ200. This negligible effect may be related to the relatively large in-situ pore pressure and distance between the piezometer and the casings. PZ203 showed a gradual pore pressure reduction of about 15 kPa which was likely caused by a significant leakage of ground water observed through several of the open casings in area D of the building pit (Figure 7-8).

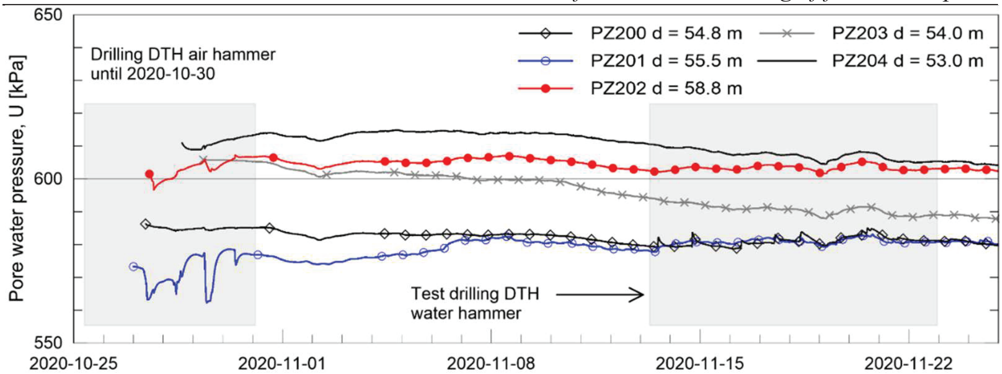


Figure 7-10. Measured pore-water pressure (U) against time in piezometers installed to the top of the till layer in the central part of the building pit. Depth (d) is related to the installation level.

7.3.8 *Drill cuttings volume balance*

An important part of the test drilling scheme was to assess the volume balance of the generated drill cuttings (V_c) and the theoretical volume of the installed pile (V_p). This was determined by measuring the total volume of the backflow (V_{BF}), i.e. flushing water plus drill cuttings of each pile. For the backflow in clay and till average densities (ρ) of 1120 kg/m^3 and 1220 kg/m^3 respectively were determined from representative samples (approx. 0.5 L) taken during drilling of every 12 m sections of the casings. The respective in-situ soil volumes were back-calculated from average water content (w) of 35% and clay in-situ total unit weight of 1800 kg/m^3 based on findings from the ground investigations. An in-situ total unit weight of 2000 kg/m^3 was chosen for the till. Potential volumetric changes in the soil (compression, dilation) caused by the drilling were ignored.

Table 7-2 presents an overview of the volume balance calculations for eleven of the thirteen casings drilled with the water hammer. The volume balance for drilling in clay ($M_{c,c}$) and till ($M_{c,t}$) are given as the ratio between the volume of drill cuttings (V_c) and the installed pile casing volume (V_p). The results indicate that the drill cuttings generated through the clay represent in-situ soil volumes of 57 to 111% of the theoretical gross volume of the installed casings. This implies that the soil was partly displaced during drilling, which seems reasonable considering the relatively high penetration rates reported by the drilling contractor. Through the till, the opposite was observed. In other words, the volume of the generated drill cuttings exceeded the pile volume (i.e. $M_{c,t}$ = 108-144%), suggesting a soil volume loss. It is possible that the soil displacement through the clay partly compensated for the volume loss through the till and to some extent contributed to the small ground settlements when using water drilling (Figure 7-9 b).

In addition to the volume calculations presented in Table 2, measurements of the total drill cuttings volume from the till layer were carried out for one of the test piles. This was done by collecting all the backflow generated during drilling in the till layer in a closed container. After a few days of sedimentation, the free water was pumped out and drained from the tank and the remaining soil volume was measured in a loose state. The results indicate that the total in-situ volume of soil from drill cuttings through the till represent approx. 100% of the gross volume of the casing. This assumes an in-situ soil density (ρ) of the till equal 2000 kg/m^3 and saturated density of the collected cuttings of 1500 kg/m^3 respectively. Despite uncertainties in the measured volume and assumed densities, the volume balance estimate indicates that water hammer drilling through the glacial till did not cause notable excess drill cuttings. The differences compared to Table 7-2 could be related to uncertainties in estimating the density of the backflow which was limited by a small number of samples.

Table 7-2. Drill cuttings volume balance based on measured density and volume of drill cuttings (backflow) from test drilling with a water hammer.

Pile	OD casing [mm]	$L_{p,c}$ [m]	$L_{p,t}$ [m]	$V_{p,c}$ [L]	$V_{p,t}$ [L]	$V_{BF,c}$ [m³]	$V_{BF,t}$ [m³]	$V_{c,c}$ [L]	$V_{c,t}$ [L]	$M_{c,c}$ [-]	$M_{c,t}$ [-]
1	273	42.5	12	2488	702	13.5	8.5	2025	935	0.81	1.33
2	324	53.8	3.7	4433	305	19.3	3.7	2895	407	0.65	1.33
3	273	48.8	8.6	2857	503	12	6	1800	660	0.63	1.31
4	406	42.8	15	5552	1946	27.5	25.5	4125	2805	0.74	1.44
5	406	47.9	9.3	6213	1206	45.8	11.8	6870	1298	1.11	1.08
6	273	54.1	9.6	3167	562	14.7	5.7	2205	627	0.7	1.12
7	273	47.2	9.2	2763	539	11.2	6.1	1680	671	0.61	1.25
8	273	47.8	8	2798	468	10.7	4.9	1605	539	0.57	1.15
9	273	49.3	9.2	2886	539	12.9	6.5	1935	715	0.67	1.33
10	273	48	9.1	2810	533	11.5	6.1	1725	671	0.61	1.26
11	273	49.2	8	2880	468	12.2	5.2	1830	572	0.64	1.22

$L_{p,c}$ – Length of casing in clay
$L_{p,t}$ – Length of casing in till
$V_{p,c}$ – Volume of pile casing in clay
$V_{p,t}$ – Volume of pile casing in till
$V_{BF,c}$ – Total volume of backflow from drilling in clay
$V_{BF,t}$ – Total volume of backflow from drilling in till
$V_{c,c}$ – Volume of in-situ clay from backflow
$V_{c,t}$ – Volume of in-situ till from backflow
$M_{c,c}$ – Soil volume balance in clay given by $V_{c,c}/V_{p,c}$
$M_{c,t}$ – Soil volume balance in till given by $V_{c,t}/V_{p,t}$

7.4 **Case 3**

7.4.1 Project description

This project involved construction of a pedestrian bridge over the Ring 3 road close to the national football stadium in Oslo. The bridge is founded on a total of 19 end-bearing pile groups, see layout in Figure 7-11. The piles were installed in similar way as with Case 1 and 2, by overburden drilling of casings with an outer diameter of 508 and 610 mm respectively and with an approx. 1 m embedment into solid bedrock. After drilling and cleaning of the boreholes, steel reinforcement was installed, and the casings were cast with concrete.

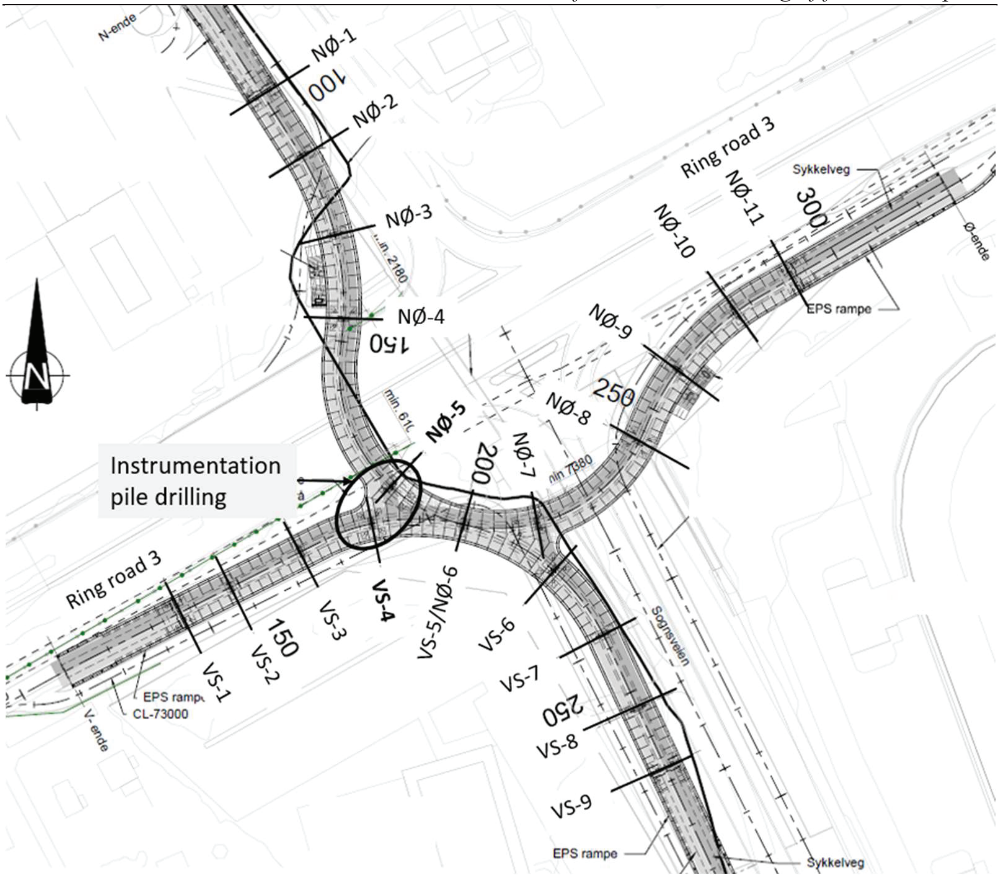


Figure 7-11. Layout of pedestrian bridge over Ring road 3 at Ullevaal in Oslo. Location of the foundation axis ("VS-X" and "NØ-X") are shown. Instrumentation of pile drilling next to axis VS-4 and NØ-5 are marked.

7.4.2 *Ground conditions*

The construction site is flat and lies in an area of thick marine clay deposits ranging in depth from about 20 m to more than 45 m along the bridge (Figure 7-12). Ground investigations shows that the soil consists of a top layer with 1-2 m of fill material and/or dry crust above a medium stiff to stiff clay, with some small layers of silty clay in between. There are also some fine sand layers close to bedrock.

Figure 7-13 presents a typical soil profile with index data based on investigations carried out on piston samples from the site. Laboratory tests on undisturbed clay samples showed quite large variations in water content (w) from below 20% to over 35%, but mostly between 25-35% which is common for medium stiff Norwegian clays. The soil unit weights (γ) are typically between 19.0-19.5 kN/m^3 down to about 25 m depth, from where it is about 18.5 kN/m^3. The clay is characterized as quick (sensitivity, S_t from 24 to over 300) from about 9 m soil depth and down to bedrock. Interpreted cone penetration tests (CPTu) show that the undrained active shear strength (c_{uA}) in the clay increases from approx. 40 kPa at 4 m depth to 100 kPa at 34 m depth.

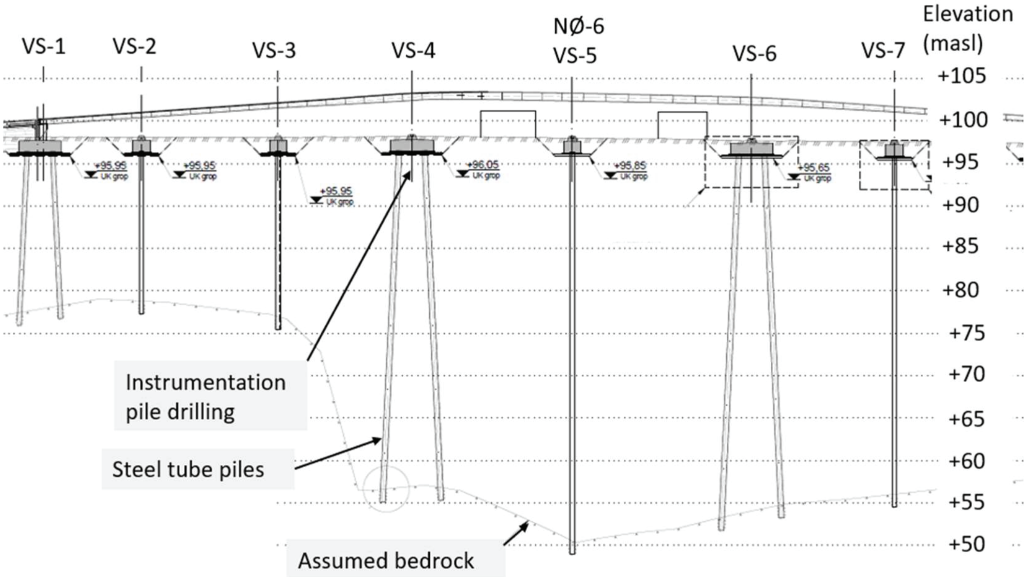


Figure 7-12. Profile of the southern part of the pedestrian bridge with foundation axis, steel tube piles and assumed bedrock surface. Instrumentation installed close to axis VS-4.

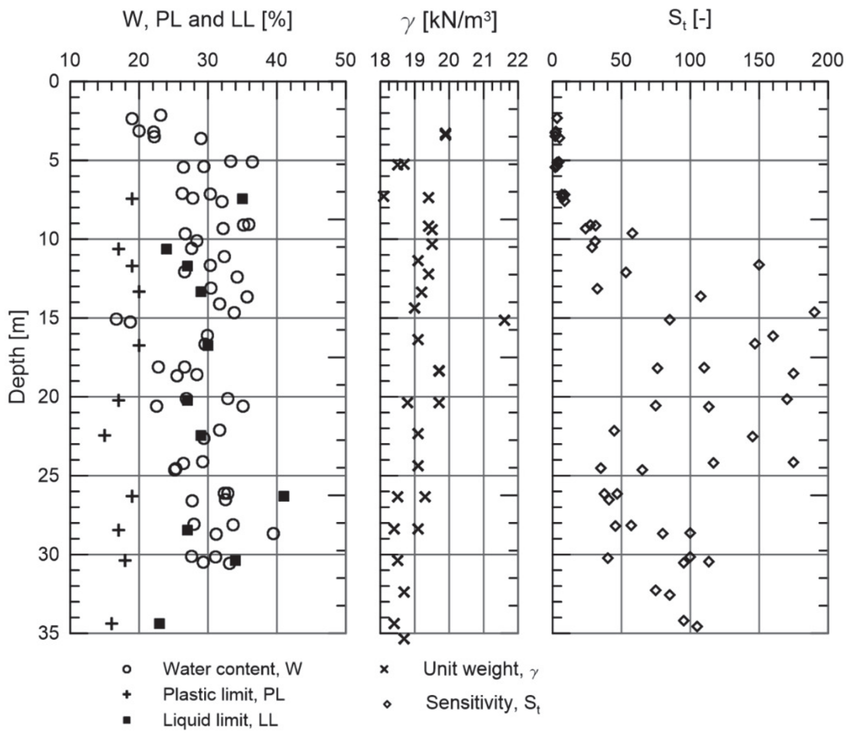


Figure 7-13. Soil profile with index data from the construction site in case 3.

7.4.3 *Instrumentation*

Four Borros type settlement anchors (Geokon, 2020) and four electrical piezometers (PZ) were installed adjacent to pile group VS-4 and NØ-5 to monitor and study the installation effects from overburden drilling. Layout of the instrumentation are shown in Figure 7-14. Table 7-3 shows the installation depths of each instrument.

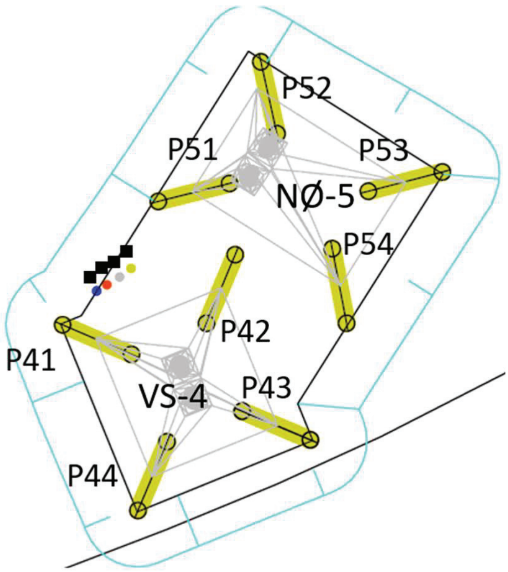


Figure 7-14. Layout of pile foundations VS-4 and NØ-5 including piezometers PZ1 to PZ4 (black symbols) and settlement anchors (S1 = blue, S2 = red, S3 = grey, S4 = green).

Table 7-3. Instruments installed adjacent to pile foundations VS-4 and NØ-5. (S = settlement anchor and PZ = piezometer).

Instrument	S1	S2	S3	S4	Z1	Z2	Z3	Z4
Depth [m]	25	20	13	3	6	16	26	37

Vertical displacements were measured on top of each anchor rod using a total station theodolite. The accuracy of the measured values was approximately ±2 mm. Settlements were monitored over a total period of about 5.5 months, starting after drilling of casings for pile P52 and P51 had been carried out. The measurements were carried out twice a week during drilling in foundation NØ-5 and VS-4 to be able to document immediate effects. The following measurements were carried out with longer time intervals.

The piezometers were installed about one week before the pile drilling started in foundation NØ-5. The sensors had a general logging frequency of 20 minutes during drilling in foundation NØ-5 and VS-4. During drilling of pile P43 the logging frequency was set to one minute.

7.4.4 *Influence on pore-water pressure*

Figure 7-15 presents the measured pore-water pressures (U) against time for a period of about 6 months. The time of drilling in pile group NØ-5 and VS-4 are marked. Figure 7-16 presents in more detail the measurements during drilling of each casing in NØ-5 and VS-4. Reference pressures (u_{ref}) at the given soil depth are based on the measurements prior to drilling.

The measurements show similar influence as with Case 1, with immediate and temporary changes in pore pressures during drilling. The piezometers show rapid variations between significant excess pressures and reductions.

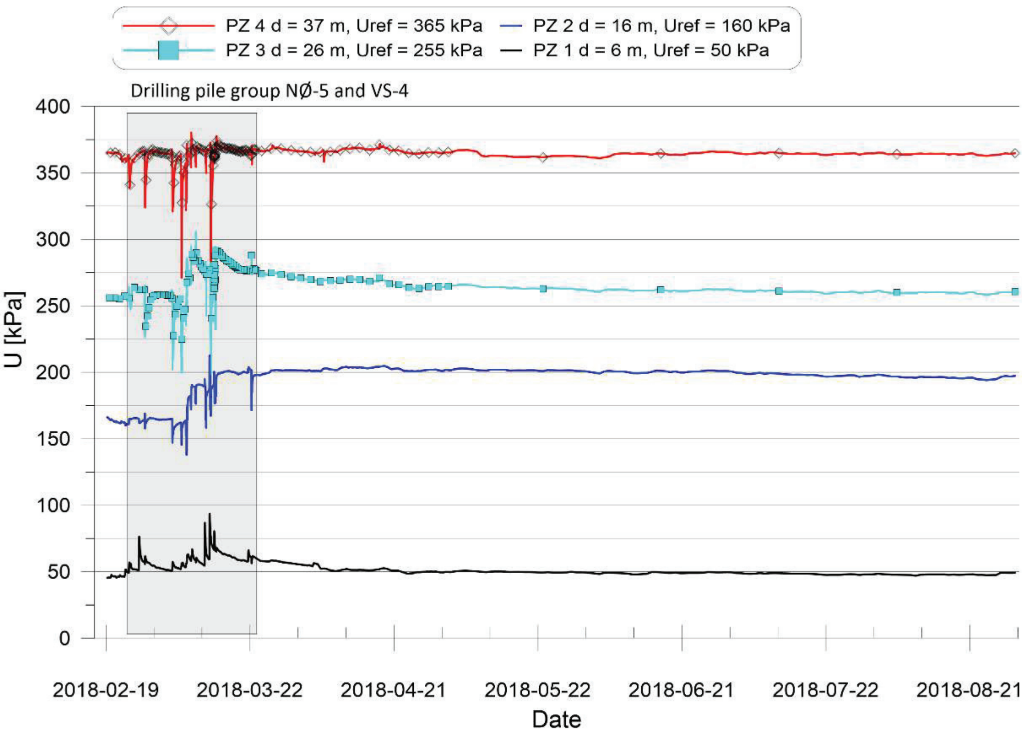


Figure 7-15. Pore-water pressures (U) measured in piezometers installed adjacent to pile group VS-4 and NØ-5 over a period of about 6 months.

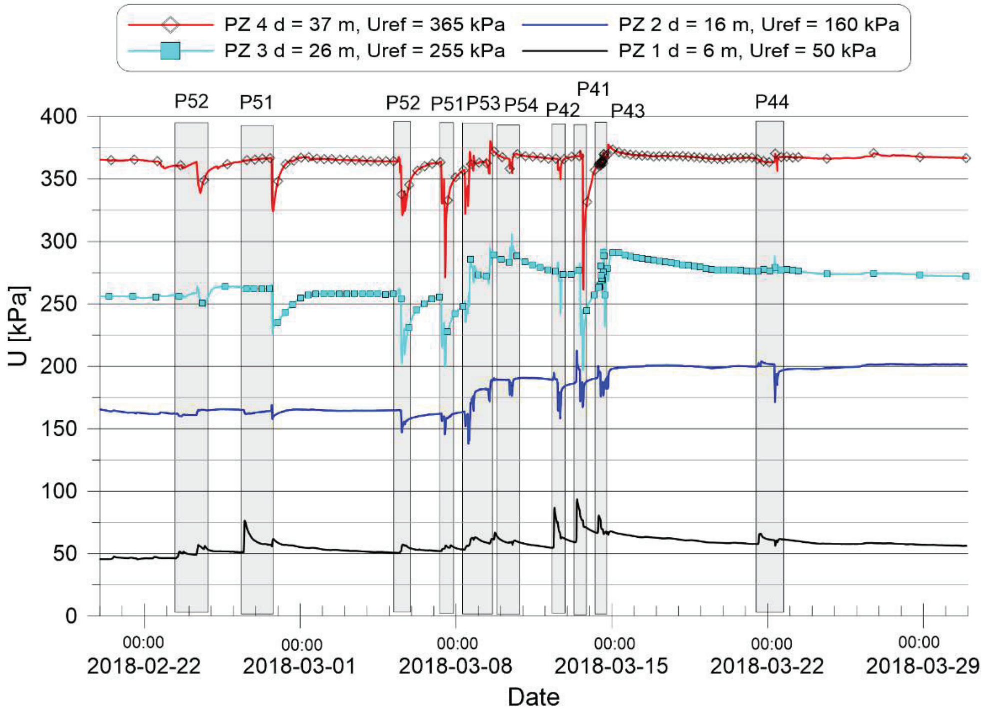


Figure 7-16. Pore-water pressures (U) measured in piezometers installed adjacent to pile group VS-4 and NØ-5 during pile drilling. The drilling time for each pile is marked with grey rectangles.

7.4.5 *Ground displacements*

Figure 7-17 presents vertical ground displacements, δ_v, against time for the settlement anchors adjacent to pile group VS-4 and NØ-5. The time of drilling is marked with a grey rectangle. The reference measurements were done on the 27th of February 2018, after drilling of pile P52 and P51 had been carried out. The last measurement was carried out about 4.5 months after the drilling in VS-4 and NØ-5 was completed.

The measurements show a considerable vertical displacement on all the settlement anchors during drilling of the pile casings in pile group VS-4 and NØ-5 with values between 0.22 and 0.26 m. This immediate response indicates a significant loss of soil mass around the casings causing the surrounding soil to settle. The results show a further increase of more than 0.1 m on all anchors except S3 in the following months after drilling.

The settlement measurements show values which are much larger than what has previously been documented and experienced, e.g. Case 1. Despite the large settlements on the anchors, there were no visible signs of the settlements on the ground surface. Taking into consideration the ground conditions with highly sensitive clay, there is a chance that local remoulding and possible liquefaction of the soil around the casings have caused the anchor rods to “sink” into the soil.

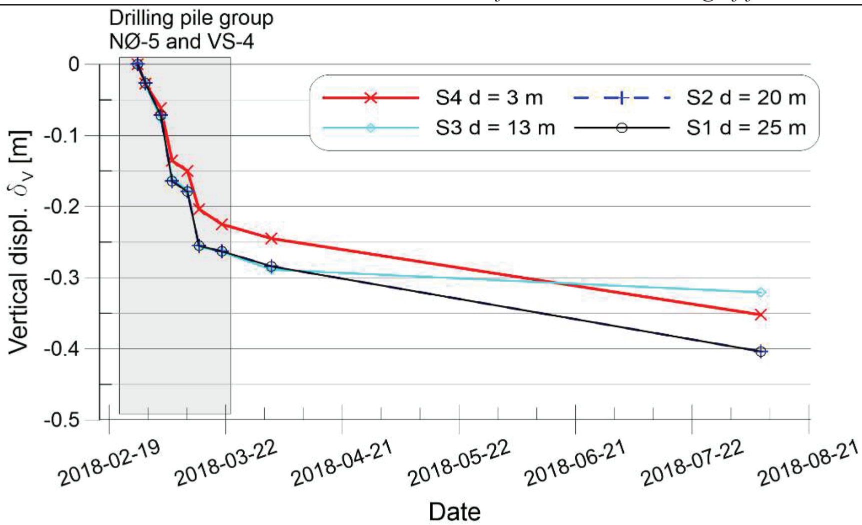


Figure 7-17. Measured vertical displacements on settlement anchors adjacent to pile group VS-4 and NØ-5.

7.5 **Case 4**

7.5.1 Project description

This project involved the construction of a new 7 story residential building with one and two levels of underground basements in Oslo, Norway. The total excavation depth varied from 1 m to about 4.6 m, where the deepest part of the building pit was supported by sheet pile walls (SPW) and temporary tieback anchors drilled into bedrock. Where the depth to bedrock was greater than 15 m, every second sheet pile was driven to bedrock to withstand the vertical forces from the tieback anchors. The building was founded on 68 steel core piles with dimensions from 90 mm to 180 mm. The steel cores were installed in permanent casings drilled to bedrock using rotary percussive DTH air hammer and an eccentric drill bit.

The ground conditions on the site consisted of a top layer of fill material (gravel, rocks) and dry crust to a depth of 3-3.5 m, followed by a soft to medium stiff silty clay which was characterized as quick from about 9 m depth and down to bedrock. The depth to bedrock varied from 8 m to about 40 m in the building pit. In the area where the depth to bedrock was largest, there was a layer of dense granular soils (assumed glacial till) above bedrock. Measured pore-water pressures indicated ground water level at 3 m soil depth.

7.5.2 Instrumentation and monitoring

Supplementary instrumentation was installed to monitor the installation effects from drilling of the pile casings. Two piezometers were installed directly on the outside of the SPW, at 20 and 34 m soil depth respectively, and between the sheet piles that were driven to bedrock. The contractor performed measurements of ground settlements on the roads adjacent to the SPW. Displacements on top of the SPW were also measured during the ground works.

The measurements on the SPW showed considerable vertical displacements during the ground works, and particularly during drilling for the piles closest to the SPW in the area with a layer of till above bedrock. The results showed between 50 to 100 mm settlement on the SPW, and similar ground surface settlements on the outside of the building pit. This observation confirmed that the sheet piles were not driven to bedrock as intended but had stopped in the dense till instead.

Figure 7-18 presents results from the piezometers installed in the same area as the above-mentioned settlements were observed. Drilling with the air hammer through the till layer and into bedrock evidently caused major reductions in the pore-water pressures at the top of the till layer (i.e. PZ 1), with maximum values up to approx.75 kPa. The effects were temporary and the data show that the pressure returned to its in-situ reference level within a few hours after the drilling was completed. This indicates that the recharge of ground water in the till layer over bedrock was relatively high. PZ 1 installed in the clay deposit at 20 m depth showed excess pressures with a maximum of about 40 kPa during drilling. Figure 7-18 show that the pore pressure in PZ 1 had not stabilized after the installation and before the drilling started. Measurements in the following months showed a slow dissipation to the in-situ reference pressure of about 170 kPa.

The monitoring verified that the SPW was not installed to bedrock as intended, and consequently experienced large vertical settlements during pile drilling. The use of air flushing most likely caused erosion and a loss of soil volume around the drill bit and pile casings.

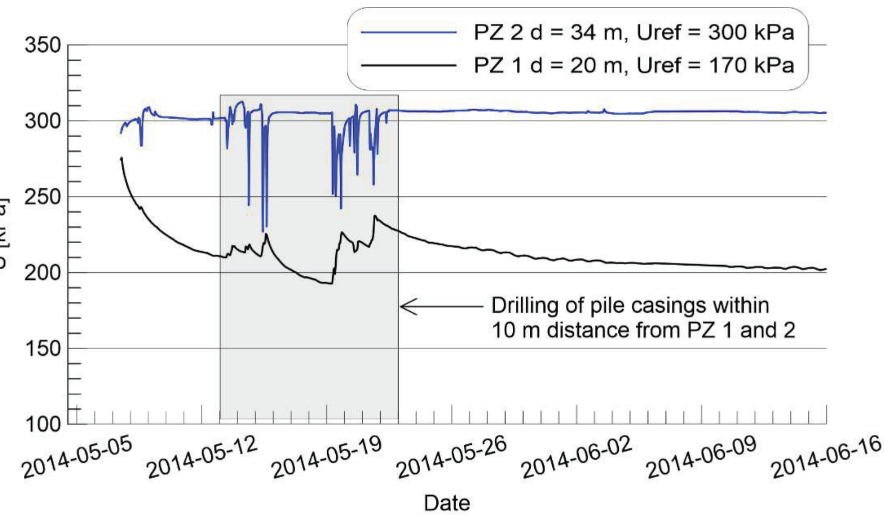


Figure 7-18. Measured pore-water pressures (U) in piezometers installed close to the SPW in Case 4. Time of pile drilling within 10 m distance from the piezometers is marked with a grey rectangle.

7.6 **Case 5**

7.6.1 Project description

This project involved the construction of seven new railway tracks leading into the central station in Oslo, Norway. The railway tracks were laid in a new 600 m long concrete tunnel under the historic Medieval Park. The tunnel was constructed within excavations up to approx. 10 m deep, supported by stiff SPW, internal struts, and soil stabilization with deep dry soil mixing and jet-piling within the building pit. A part of the tunnel was constructed with the "top down" method, while the main part was built with the "cut and cover" method. The tunnel was founded on several hundred end-bearing steel core piles installed in casings drilled into bedrock using rotary percussive duplex drilling. The pile casings typically had an outer diameter of 273 mm.

The results presented and discussed in the following focus on two separated areas along the deep excavation for the tunnel, named the "*South*" and "*North*" area. In the South area the drilling was carried out with DTH air hammer while in the North area a double head drill rig and DTH water hammer as with case 2 were used. Both drilling methods used concentric ring bit and pilot bit.

The ground conditions on the site were varying. In the South area the soil was characterized by a 2 m thick top layer of fill material (sand, silt) and dry crust. Below was a soft to medium stiff silty clay with unit weights (γ) from 18.5-19.0 kN/m^3 and water contents (w) between 30-40%. The undrained active shear strength (c_{uA}) in the clay varied from approx. 30 kPa in the top to about 70 kPa at 17 m depth. The clay was characterized as highly sensitive and quick from 15 m depth. Below the clay was a thin layer (1-2 m) of compact sandy and gravelly glaciofluvial soils (interpreted as till) over bedrock. The depth to bedrock was about 20 m in the South area.

In the North area the soil conditions were similar to the South area to about 20-25 m soil depth, with soft to medium stiff clay. Below this upper clay, deposits of compact glaciofluvial soils were found in between layers of silty clays, and with an up to 15 m thick deposit over bedrock. The depth to bedrock was over 50 m in the North area.

7.6.2 Instrumentation and monitoring

As part of the present Phd-study, supplementary instrumentation was installed to monitor the installation effects from drilling of pile casings and assess the impacts on the surroundings. The initial instrumentation involved three piezometers (PZ) in the South area. PZ1 and PZ2 were installed close to the inside of the SPW, in the till at 21.6 m depth and in clay at 17.0 m soil depth respectively. PZ3 was installed close to the outside of the SPW in the till above bedrock at 20.6 m soil depth.

Figure 7-19 presents measured pore pressures against time during drilling with the DTH air hammer in the South area. The results include measurements during soil stabilization works directly next to the piezometers.

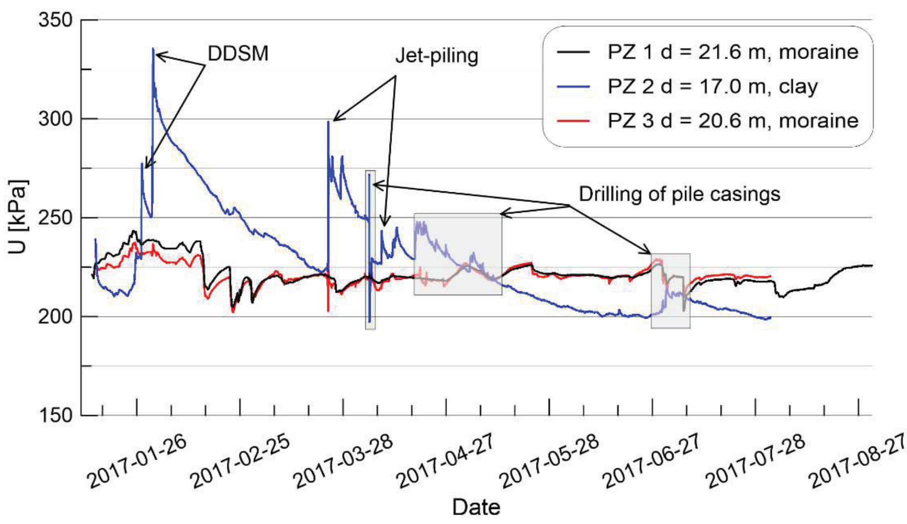


Figure 7-19. Measured pore-water pressures (U) in piezometers installed adjacent to the inside (PZ1, PZ2) and outside (PZ3) of the SPW in the South area of case 5. The time of pile drilling within 15 m radial distance from the piezometers are marked with grey squares.

Both the deep dry soil mixing (lime-cement stabilization) and jet-grouting inside the building pit had major impacts on the pore pressures in the clay causing temporary excess pressures in PZ2 up to about 120 kPa and 90 kPa respectively. Smaller variations in the pore pressure were also observed in the till (PZ1 and PZ3) during the same period as the soil stabilization works. Due to missing information on the drilling protocols, it was not possible to decide what caused these changes. However, it is likely that the temporary pore pressure reductions in the till were related to drilling of the casings and anchoring depth in bedrock for piles in the surrounding area, as observed later shown in the grey squares. The pile drilling executed later than the soil stabilization caused smaller excess pressures in the clay, but at a considerably longer distance to the piezometers.

At a later stage in the project the North area was instrumented with piezometers to investigate the effects on the pore pressures from water hammer drilling. Two piezometers were installed in the glaciofluvial soils at 30.6 m and 34.7 m depths respectively. Figure 7-20 shows the measured pore pressure changes (ΔU) against time during drilling of pile casings with the DTH water hammer in the North area. Significant excess pressures with a rapid response were observed, with a maximum value of almost 150 kPa. The large excess pressures were mainly explained by the continuous water flushing and the water head levels in the drill rod and casings being much higher than the in-situ pore-water pressure (up to 12 m higher).

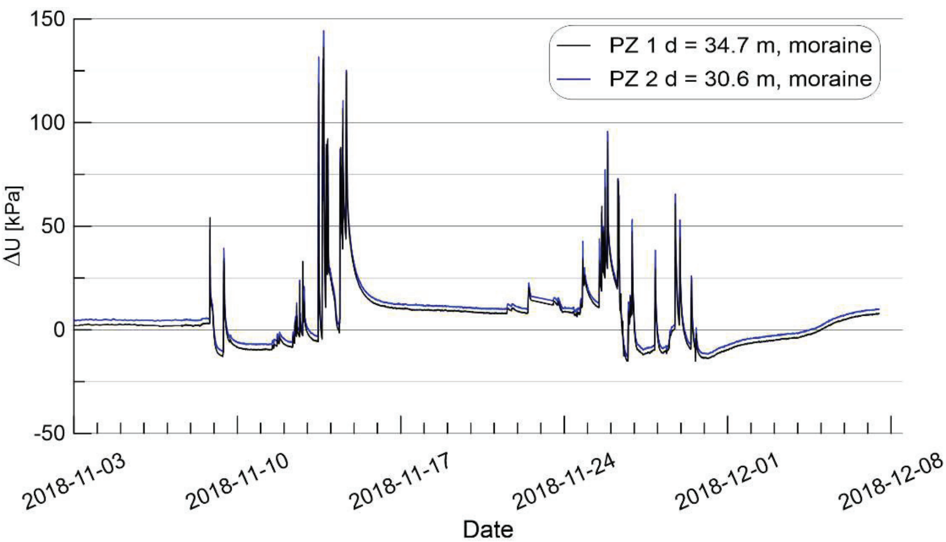


Figure 7-20. Measured pore-water pressure changes (ΔU) during drilling of pile casings with the DTH water hammer in the North area of case 5.

7.7 **Discussion**

7.7.1 *Effects on ground settlements*

Case studies 1 to 4 show that overburden drilling with the DTH air hammer caused significant ground settlements during and/or shortly after the drilling was completed. The monitoring data clearly indicates that these immediate settlements were mainly caused by excessive loss of soil mass (i.e. volume loss) as described in Section 4.4. The displacements registered on settlement anchor S5 in case 1 (Figure 7-5) verifies this effect when drilling through the glacial till (moraine) layer over bedrock. Results from the other case studies support the findings from case 1.

For case 1 the settlement anchors installed at different depths in the clay (i.e. S1-S4) showed smaller settlements than anchor S5 at the top of the till. This can likely be related to both a delayed response in the clay, and gradual decrease of the volume loss effect with closer distance to the soil surface. These lesser settlements are likely a result of soil arching effects (Terzaghi 1936).

The settlement anchors in case 3 showed almost the same values at different soil depths. This may be explained by local disturbance and liquefaction of the sensitive quick clay due to the drilling, causing the anchors to sink in the clay under their own weight.

The results from settlement anchor S5 in case 1 indicate that the influence from local soil loss around individual pile casings can be approximated by an inverted cone around each casing with an apex angle of 45 degrees, when drilling in the till layer. An influence zone of such extent is in line with previous numerical modelling and back analysis of soil volume loss (Lande 2009; Sandene et al. 2023).

Cases 1, 2 and 4 all showed that it was challenging to drill through the silty and sandy glacial tills with the DTH air hammer. Similar ground conditions were also related to the excessive settlements caused by anchor drilling in studies by Konstantakos et al. (2004), Küllingsjö (2007) and Sandene et al. (2021). However, the massive, accumulated settlements in case 2 due to air hammer drilling exceed the other cases and previous studies by far. Factors which likely magnified the negative impacts from drilling with the air hammer are: i) large number of piles combined with large depth to bedrock, ii) thick glacial till layer, iii) artesian pore-water pressures, and considerable ground water recharge in the permeable till. The combination of these factors contributed to longer duration of drilling in the problematic till layer, increasing the soil erosion and caused a larger influence area at ground level. The main factor was most likely the ground water recharge amplifying the soil erosion effect by "feeding" erodible soil towards the drill bit. The hydraulic ground failures occurring at the bottom of approximately 30% of the 406 mm diameter casings most likely contributed to the large soil loss and ground settlements.

The test drilling for case 2 showed a definite difference in performance between air and water hammer drilling. The possible benefits of using water flushing to reduce settlements in the surrounding ground became evident. While the air hammer struggled to drill through the till with high pore-water pressures and ground water recharge, average penetration rates from 25 to 137 cm/min were achieved with the water hammer. Hydraulic ground failure was not observed when water drilling due to the casings being constantly filled to the top with water. In addition, considerable drilling stops could be avoided in the till and the casing could be drilled at least 1 meter into bedrock. Asplind (2017) have however, reported ground failure occurring in some casings installed by use of water hammer drilling through granular soils, however without discussing the reasons for it.

Measurements from cases 1, 2, and 3 all show that the ground settlements continued after the pile drilling was completed, but at a substantially reduced rate. The settlements of S4 and S5 in case 1 could likely be explained by a combination of the following effects: 1) Soil closing the presumed gaps/cavities created adjacent to the casings (illustrated in Figure 4-3 b). 2) Consolidation settlements in the clay deposit due to permanent pore pressure reduction (about -25 kPa) in the till layer (Langford and Baardvik, 2016). 3) Re-consolidation of remolded clay along the casings, as described by Lande et al. (2020). However, due to limited settlement data for the anchors S1-S3 this was not verified. The same effects most likely explain the long-term settlements of case 2, where the large thickness of the till layer and large number of piles magnified the influence. Due to previous placement of fill at the case 2 construction site, creep settlements may also have contributed to the settlements around this site.

7.7.2 *Effects on pore-water pressure*

The monitoring data and experiences obtained from construction projects studied in the *Limiting Damage* projects provide valuable insights to the installation effects from drilling. Results from pore pressure measurements provide essential information when assessing the impacts of drilling and the potential for causing excessive settlements.

Figure 7-21 presents the maximum changes in pore pressure (ΔU_{max}) measured during drilling of individual pile casings in the case studies summarized in Section 7 plus two additional unreported cases. The pore pressure changes are presented against normalized distance r/r_0 (Figure 7-21 a) and radial distance r in meters (Figure 7-21 b) from the piezometers to the respective casings. r_0 represents the radius of the casings. For comparison, Figure 7-22 presents similar measurements from reported studies summarized in Section 3.

The impact on pore pressure from drilling generally reduces with increasing distance. However, the data show that a significant impact can be observed at distances of 30 to 40 m from the drilling, equal to approx. 100 times the radius of the casings (r_0). Such influence areas are larger than experienced from other geotechnical installation works, like pile driving and ground improvement. The piezometers installed in till/moraine generally show larger changes in pore pressures at greater distances from the drilling than the piezometers in clay. This is explained by the higher hydraulic conductivity in the till/moraine compared to the clay.

Based on the compiled field data some general trends are observed. Drilling in clay typically cause significant excess pore pressures (positive values for ΔU_{max}) in the surrounding clay (open black circles). This response is mainly related to high penetration rates leading to a varying degree of soil displacements comparable to driven closed ended piles. Continuous flushing with water or air during drilling in clay seems to increase the excess pressures. The effects are temporary and most of the excess pressure dissipates within hours or a few days after drilling. These field observations correspond well with the full-scale field tests described in Section 5 and the test drilling reported by Ahlund and Ögren (2016). Results from Case 1 and Case 3 show that drilling with the air hammer caused both excess pressures and pore pressure reductions (negative values for ΔU_{max}) in the clay. The delayed re-establishment of pore pressures following sudden pressure reductions in the clay as witnessed for Case 1 and Case 3 could be explained by the low hydraulic conductivity.

Pore pressure reductions (i.e. negative values) were observed in the piezometers installed in glacial till (moraine material) when drilling with the DTH air hammers (black circles). This response verifies the air-lift pump effect and can be used as an indicator for the possible erosion and loss of soil mass. This is most likely the main effect causing the rapid and considerable ground settlements during and right after drilling as observed in cases 1 to 4 and corresponds well with the studies by Sandene et al. (2021) and Kullingsjö (2007). The results from drilling with DTH water hammers mainly show excess pore pressures, implying that the method does not cause notable pump effects as when using the air hammer. The excess pressures are partly explained by the water level in the casings affecting the pressures at drill bit level during drilling.

Results from case studies with both air and water hammer drilling showed that the pore pressures in the moraine layer typically re-established almost to the pre-drilling levels within a few hours after drilling was stopped. This response is probably related to a combination of ground water recharge in permeable granular soils and the procedure of filling the casings with water directly after it is drilled into bedrock.

The more long-lasting pore pressure reductions in the till layer in case study 1 and 2 (Figure 7-4 and Figure 7-10) were explained by observed drainage (i.e. leakage) of ground water into the bottom of some of the open casings before they were filled with cement mortar. This is comparable to drainage effects from bored piles at the bottom of deep excavations well below the ground water pressure head at bedrock as reported by Langford and Baardvik (2016) and Langford et al. (2015).

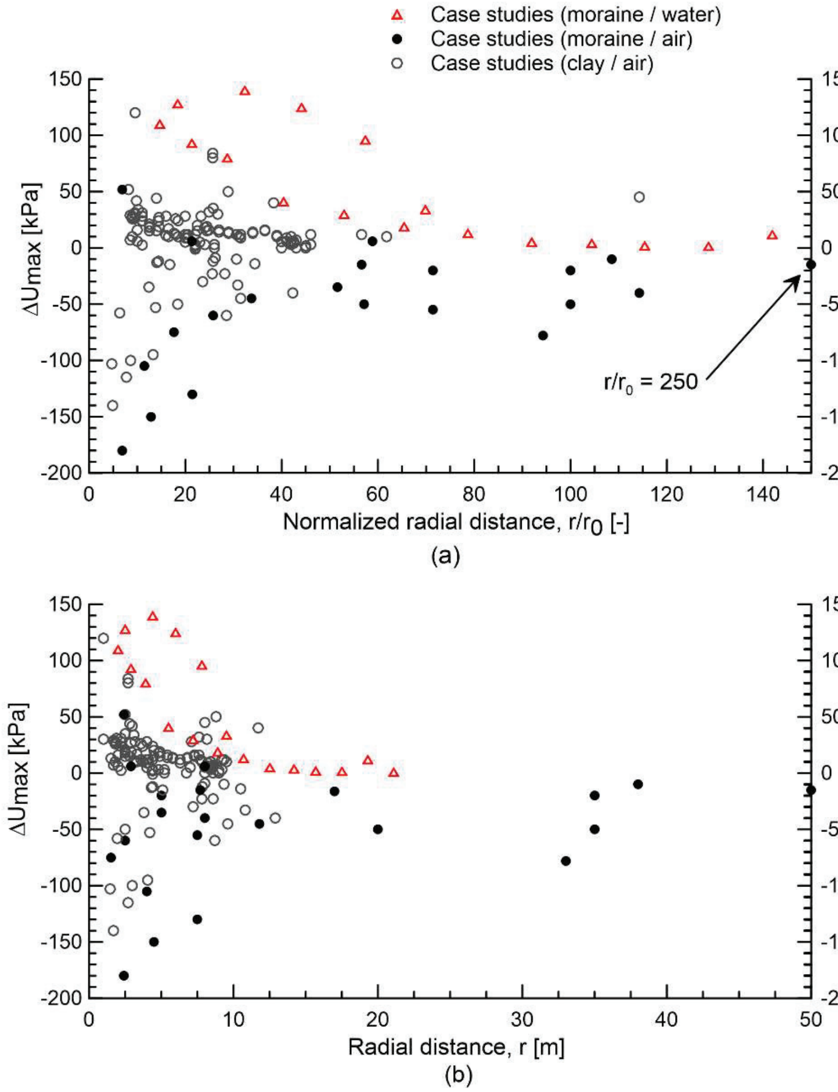


Figure 7-21. Maximum pore pressure changes (ΔU_{max}) against (a) normalized radial distance; and (b) metric radial distance between piezometer and pile casing. r_0 is the radius of the respective pile casings. Data from present case studies.

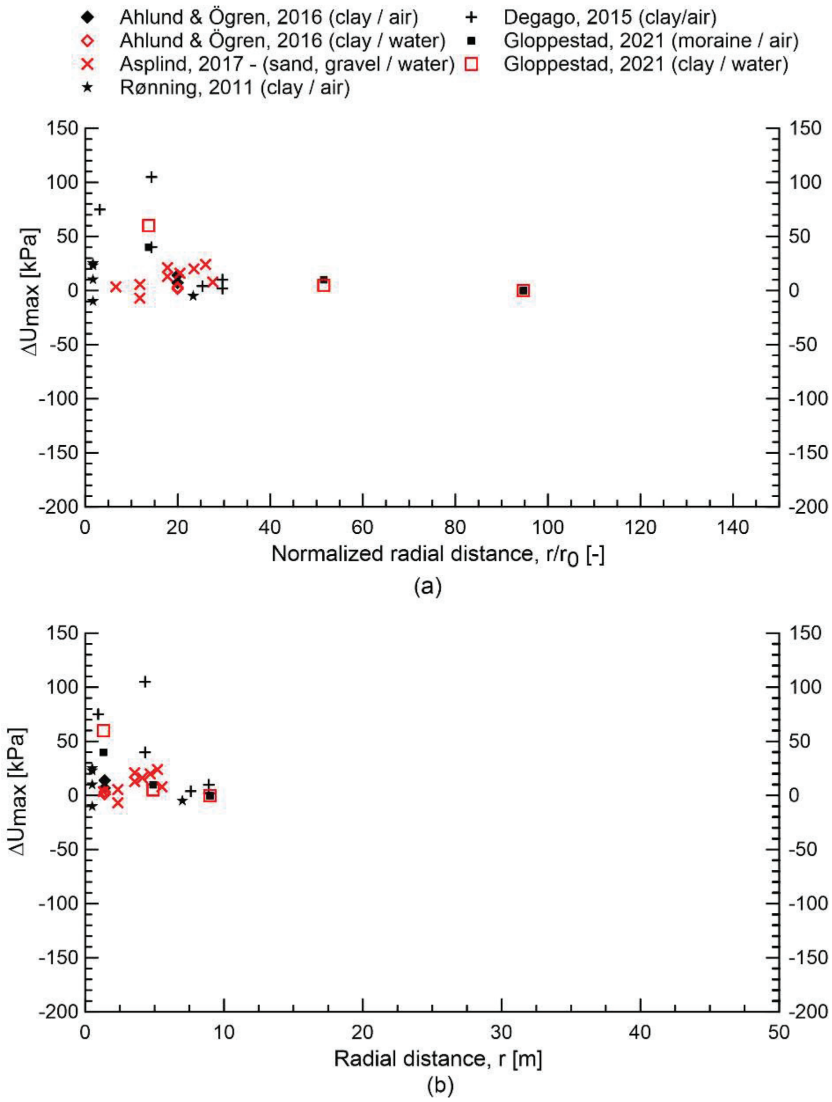


Figure 7-22. Maximum pore pressure changes (ΔU_{max}) against (a) normalized radial distance; and (b) metric radial distance between piezometer and pile casing from reported case records. r_0 is the radius of the respective pile casings.

8 Recommendations to reduce impacts on the surrounding from overburden drilling of piles and anchors

8.1 Introduction

During the past 10 years the research programs *Limiting Damage* and *Risk reduction of Groundwork Damage* have provided a substantial amount of monitoring data and experiences from numerous case studies related to deep supported excavations and foundation works (piling) in soft clays (Sandene et al. 2023; Langford et al. 2015; Lande et al. 2024). As part of this study new full-scale field tests with drilling for anchors (Lande et al. 2020) were undertaken, and also experimental model tests in the laboratory to study effects of drilling with water flushing in sand (Lande et al. 2021).

The following sections provide recommendations to reduce negative impacts on the surrounding ground from overburden drilling based on the available data and experiences, including the published studies in Section 3. Despite all the data and experiences, it is imperative to highlight that the quality of workmanship during drilling is of great importance to achieve results that limits the potential for settlements and damage induced in the surrounding areas.

8.2 Drilling methods and procedures

- Overburden drilling without the use of compressed air flushing generally reduces the risk of negative impacts on the surrounding ground. Whenever technically possible, it is therefore recommended to use a drilling method that enables drilling with water flushing through both soft soils, dense granular materials (e.g. glacial tills, sand, gravel, rocks), as well as into bedrock. Hydraulic top hammers can drill casings with an outer diameter up to about 200 mm using water flushing to remove the drill cuttings. The largest DTH water hammer currently available can drill casings up to 500 mm diameter.

- Concentric drill systems with a ring- and pilot bit designed to reduce the risk of compressed air or flushing water evacuating into the surrounding ground are generally preferred compared to eccentric systems. It should still however be noted that concentric drill systems also can cause excessive soil erosion and ground settlements as documented in case studies.

- Drilling in soft soils (clays, silts, fine sand) should always be performed by the rotary duplex method (section 2.2) with continuous water flushing to remove the drill cuttings, and to keep the flushing channels in the drill bit open. The water flushing flow rate (Q) is crucial to maintain a satisfactory velocity enabling the transport of the drill cuttings up through the annulus between the casing and the drill rod. Water pressures should typically be between 3-15 bars, and flow rates between 60-350 L/min depending on the casing dimension and depths.

- If drilling is carried out with a DTH air hammer, the air pressure from the compressor should be limited to a minimum to run the hammer and maintain acceptable penetration. The pressure should not be more than about 8-10 bar, and never more than 15 bar when drilling in soils.

- To achieve a satisfying velocity of the backflow inside the casing without the need of excessive flushing it is imperative that the diameter of the drill rod is proportional to the casing diameter. This applies to both air and water flushing methods,

- Drilling with reversed circulation systems (RC-drilling) offers a possibility to collect all the drill cuttings and flushing water into closed containers and is recommended to keep control of the volume of cuttings.

- The drilling penetration rate should be adapted to the ground conditions and casing dimensions. Results from field tests (Section 5) show that the rate of penetration when drilling in soft clays should be no more than 1 m/min to achieve a more "ideal" drilling, thus reducing soil displacements and large excess pore pressures in the surrounding soils. For large diameter pile casings, the penetration rate may need to be lower than 1 m/min. When drilling in sensitive clays or erodible soils from a level well below the ground water level (e.g. in deep excavations), too low penetration rates may increase the risk of soil collapsing around the drill bit causing excessive loss of soil mass and settlements.

- Field results generally show that the air and/or water flushing seem to increase the excess pore pressures caused by the drilling penetration itself. This additional effect should be considered when it is important to avoid large excess pore-water pressures, for instance in areas with poor stability of adjacent slopes.

- To reduce the risk of hydraulic ground failure and soil particles flowing into the bottom of casings and the drill bit, the casings should always be filled to the top with water or a suitable drilling fluid before the drilling process is stopped or paused (e.g splicing of casings).

- To avoid leakage of ground water up along the casings drilling for piles and anchors should preferably be performed from a level above the ground water pressure level at bedrock. If that is not feasible, mitigating measures should be adopted to minimize the risk of leakages. Temporary packers could for instance be installed at the top of the casings until they are permanently grouted to avoid leakages up along the inside of the casings. To reduce leakages on the outside of the casings, systematic injection grouting through the bottom of the casings after drilling of embedment into bedrock can be used. However, the quality of such grouting is difficult to control and verify. Another option may be to drive down an outer larger casing first with a limited depth (e.g. 5-6 m) before the pile casing is installed by drilling through it. The annulus between the two casings shall then be grouted with cement mortar to seal against leakages up along the outside of the internal casing. As it often takes time between drilling of casings and insertion and grouting of anchors or piles into bedrock, the internal casing should be temporarily plugged.

- Monitoring with piezometers and settlement anchors (extensometers) are recommended to document installation effects from overburden drilling and assess the suitability of the drilling method and procedure.

8.3 **Drill cuttings mass balance**

Measuring the volumes and density of the backflow from water drilling as with case 2 (section 7.3) can be used to assess the drill cuttings mass balance. However, accuracy is highly dependent on the numbers and representativeness of the backflow samples. An efficient alternative that seems to be suitable is to use the non-dimensional methodology based on model tests described in Section 6.3.5 (Lande et al. 2021). This method allows comparison of the generated drill cuttings with the drilling parameters by introducing the parameters for normalized flow rate (Q_{norm}) and normalized mass of drill cuttings ($M_{c,norm}$). The normalized flow is defined as:

$$Q_{norm} = \frac{Q}{A_{pile} \times V_{pen}} \tag{4}$$

where Q is the flushing flow rate in m^3/min, A_{pile} is the cross-sectional area of the pile in m^2, and V_{pen} is the penetration rate in m/min. The normalized mass of drill cuttings, $M_{c,norm}$, is the same as $M_{c,c}$ and $M_{c,t}$ presented in Table 7-2 where a value above 1 indicates volume loss and below 1 indicate soil displacement.

Figure 8-1 present results from the soil volume balance calculations for the test drilling in case 2. The black solid symbols represent results from drilling with different casing diameters in the till deposit. The open symbols represent drilling in the clay. The black line shows a linear polynomial fit to the results for the till. Results from the small-scale model tests replicating drilling in sand described in section 6 (Lande et al. 2021) are included for comparison. The results indicate that the volume (mass) of drill cuttings in both clay and till generally increases with the normalized flow rate, a trend also observed in the model tests. However, it seems that the magnitude of normalized drill cuttings in till consistently lies above the results from the model tests for the same normalized flow. This could be related to larger dimensions of the pilot drill bit (including channels for the backflow) for the full-scale drilling system compared to the model test. Differences in the soil properties (density and grain sizes), soil stresses and water flushing pressures may also explain the deviation in results.

The results indicate that the drilling parameters flushing flow rate (Q), and penetration rate (v_{pen}), needs to be adapted to the pile (casing) diameter (A_{pile}) to obtain an ideal ratio between the volume of drill cuttings and casing volume installed in the ground. Normalized flow rate Q_{norm} between 6-8 seem to represent optimal design values for drilling in soils characterized as moraines, as indicated by the grey box in Figure 8-1. This method requires careful logging of the drilling parameters flow- and penetration rates, preferably with sensors on the drill rig. More field data are needed to validate the method.

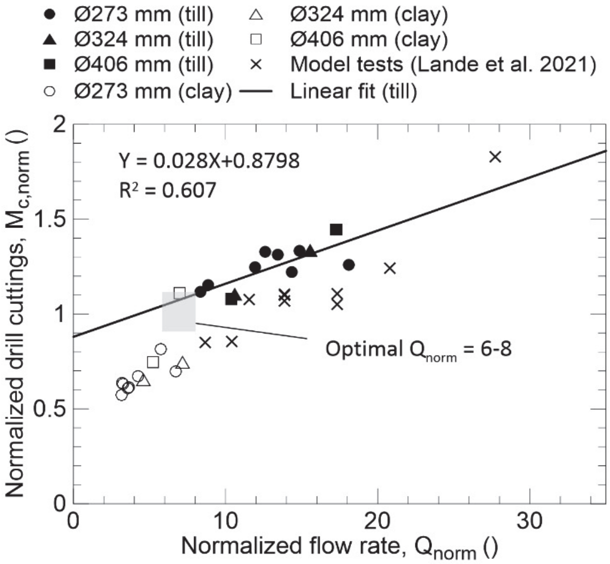


Figure 8-1. Design chart: normalized flow rate (Q_{norm}) against normalized mass of drill cuttings ($M_{c,norm}$) for test piles drilled with DTH water hammer in case 2.

8.4 A framework for selecting overburden drilling method

Figure 8-2 presents steps in a proposed simple framework which can be used to guide practitioners when selecting overburden drilling method in an urban setting to reduce the risk of excessive ground settlements. The framework is meant for situations where vulnerable structures are located within an assumed influence zone of drilling which could be described by a 45-degree inverted cone from the point where the anchors or piles enter bedrock.

Step 1 is related to undertaking sufficient ground investigations. The presented case studies 1, 2, and 4 as well as previous case records (Sandene et al. 2021; Kullingsjö 2007; Konstantakos 2004) show that it is imperative to map potential layers of granular silty and sandy soils (e.g. glacial tills) susceptible to erosion during drilling. As a minimum, information about the soil types grain size distributions and hydrogeological conditions (i.e. pore-water pressures, aquifers overall groundwater regime) should be determined. In ground conditions with artesian pore-water pressures and/or confined aquifers with a recharge of ground water, there is an increased risk of soil erosion during drilling.

If erodible silty and sandy soils and a confined aquifer are encountered, drilling with water flushing is generally recommended (step 2). Top hammers can drill casings up to approx. 170 mm diameter while DTH water hammers can drill up to 500 mm diameter. If water hammers are chosen, the proposed design chart in Figure 8-1 may be used to derive optimal drilling parameters. For such challenging ground conditions, it could be necessary to re-design and reduce the pile dimensions so that water hammers could be used (i.e. casing diameter < 500 mm).

If the ground investigation shows that there are no significant soil layers of granular silty and sandy soils combined with confined aquifers with high ground water recharge, the less expensive drilling with air hammers could be acceptable.

Regardless of which drilling method is chosen in step 2, a monitoring program should be established to assess the field performance and suitability of the drilling method and procedures (step 3). The monitoring should include electrical piezometers and settlement anchors (extensometers) at different soil depths and distances from where the drilling is carried out. In addition, the amount of drill cuttings should be measured for some piles to be able to more accurately assess if excessive soil loss occurs. Preferably, test-drilling should be carried out close to the instrumentation to enable assessment of the impacts of drilling as early as possible.

If the performance of the selected drilling method and procedures is not within acceptable limits, the drilling method should be re-assessed, or the drilling parameters adjusted (step 4). The limits for what are acceptable in terms of performance of the drilling method and possible impacts on the surroundings need to be defined in each project.

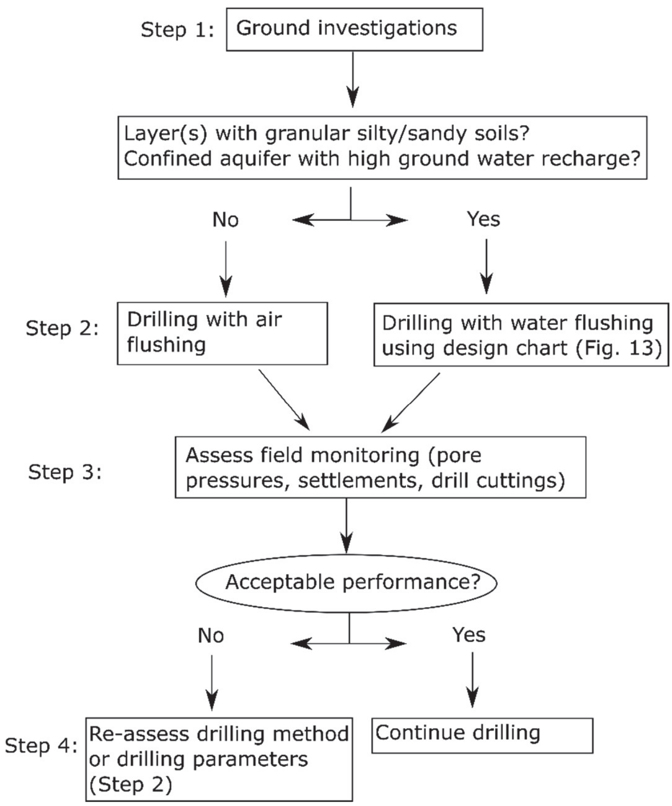


Figure 8-2. Flow chart illustrating design framework to choose drilling method in urban settings.

9 Summary

An extensive review and assessment of effects of drilling for piles and tieback anchors from within deep-supported excavations in soft marine clays in Norway show that direct and secondary effects can cause ground settlements that are significantly larger than 2% of the excavation depth. That is much larger than reported in historical studies. For excavations in urban areas, ground displacements of such magnitudes imply a large potential for causing damage to surrounding buildings, structures, and is associated with large liability potentials.

Although installation effects related to overburden drilling have been known and recognized in the past, there has been a lack of systematic and comprehensive studies of such special foundation works and their impact on the surrounding ground.

The PhD-study presented in this thesis was carried out as part of the *Limiting Damage* research projects funded by the Norwegian Research Council and the project partners. The main objective of the projects was to reduce the risk of damage to neighboring structures caused by groundwork in construction projects. The study has focused on the effects of overburden drilling for foundation piles and ground anchors on the surrounding ground.

The study was divided in the following main parts:

1) State of the Art of overburden drilling.
2) Literature review and installation effects on the surrounding ground.
3) Full-scale field test with drilling of tieback anchors through soft clay and into bedrock.
4) Small-scale model test of pile drilling in sand.
5) Case studies of overburden drilling of foundation piles
6) Recommendations to reduce impacts on the surrounding ground from drilling of piles and anchors.

In this thesis overburden drilling is characterized by drilling permanent continuous casings for piles or anchors to support the borehole through varying soils (i.e. overburden) and further into bedrock. The study mainly focused on the “rotary percussive duplex” drilling methods which involves a combination of rotation and percussion on the drill bit. The percussion on the drill bit is provided by either a top hammer acting on top of the drill rod or a down-the-hole (DTH) hammer located just above the drill bit. These drilling methods require efficient flushing with a suitable fluid to cool the drill bit and transport drill cuttings up from the borehole. Top hammer drilling can be carried out with either air or water flushing through the drill rod. The DTH hammers on the other hand require flushing with compressed air or high-pressure water to run the hammer.

Full-scale field test – drilling of anchors in soft clay and into bedrock

A full-scale field test with drilling of anchor casings was conducted at a site at Onsøy, Norway. The main objective was to investigate and compare the impacts on pore-water pressures and ground surface settlements from drilling with five different drilling methods/systems including DTH hammers driven by air or water flushing. Eight anchors were installed with each drilling method in separate areas on the test site. Three piezometers and eight settlement anchors were installed in each area to monitor the installation effects.

The results show that the drilling penetration rate is of great importance for the impact on pore pressures in the ground adjacent to the casings. Two methods, including the DTH air hammer, caused much higher excess pore pressures than what is previously experienced with driven, closed-ended piles with the same dimensions. Most of the excess pore pressures dissipated during the first days after drilling, while the remaining part took several months to completely dissipate as part of re-consolidation of remolded clay around the anchor casings. The results indicate that a penetration rate of approx. 1 m/min reduces soil displacements and the excess pore pressures significantly.

The field test documented the potential detrimental effects of drilling with DTH air hammer (Area B, Method 2). The method clearly stands out with significantly larger ground settlements compared to the other methods with a maximum of 12 mm. Settlement anchors 2 to 5 in Area B settled the most with 11-12 mm which is about twice the settlements in Area E with top hammer drilling and water flushing. This shows that the area directly above where the anchors hit the sand/moraine layer and entered bedrock were most affected. The fact that most of the settlements measured in all the test areas occurred after drilling with Method 2 (and 4), indicates that drilling with air flushing had a major impact and influence on the area. Between 40 to 50% of the final settlements were caused by long-term consolidation.

The compressed air needed to run the hammer and to drill the casings into bedrock caused uncontrolled blowouts of air up along the outside of the casing and in the surrounding ground. The monitoring data clearly indicated that air flushing with both DTH air hammer and with the top hammer (Area B, Area D) affected the entire test field, most likely due to the air-lift pump effect which was documented.

The test indicates that drilling with only water flushing will not cause the same pumping effects as with air flushing, thus reducing the risk of excessive ground settlements.

Model tests of pile drilling in sand

Novel experimental model tests with drilling of a miniature pile in fully saturated medium dense sand with simultaneously rotation, penetration and flushing were carried out as part of the Phd study. The test set-up made it possible to investigate the main mechanisms of rotary flush drilling and the effects of different drilling parameters on the surrounding soil in a more controlled manner compared to field experiences. The accurate measurements of drill cutting volumes and mass to assess the potential soil volume loss was important.

The tests were conducted with either water flushing or with air flushing. Different water flushing flow rates and penetration rates were tested. The results from water flushing showed that increased flow rates caused larger excess pore pressures, generated more drill cuttings, and reduced the soil resistance significantly close to the pile. Increasing penetration rates compensated for the effects from increasing the flow rate. This indicated an inverse relation between these drilling parameters.

A framework with normalized flow rate (Q_{norm}) and normalized mass of drill cuttings ($M_{c,norm}$) are introduced to assess the soil mass balance and derive optimal drilling parameters when drilling with water flushing. Q_{norm} includes the flushing flow rate (Q), the cross-sectional area of the pile (A_{pile}) and the penetration rate (V_{pen}). High values of Q_{norm} (above 15) fluidized the soil in front of the drill bit and reduced the penetration resistance, a mechanism comparable to observations during pile jetting. The fluidization may lead to considerable ground settlements. For tests with too low flow rate (1.5 l/min) or too high penetration rate (4 mm/s), opposite behavior was observed, and the soil resistance increased.

Different air flushing pressures were also tested but were not successful in transporting the drill cuttings up through the pile, resulting in piping on the outside of the pile. The air flushing tests were limited by modelling constraints; thus, no clear conclusions can be drawn from these tests. However, a notable reduction of pore pressures adjacent to the casing was measured. This finding may indicate that air flushing causes a behavior equivalent to an air-lift pump effect which could lead to considerable erosion, soil loss and resulting ground movements.

Case studies – overburden drilling for piles

Several construction projects involving overburden drilling of foundation piles have been studied. Results from instrumentation and monitoring combined with observations have provided new insights and documented installation effects when drilling with rotary percussive duplex methods under different ground conditions. Based on the case studies some general trends are observed:

Monitoring of pore-water pressures shows that drilling with DTH air hammers may cause significant pressure reductions in the ground verifying the air-lift pump effect. The pressure reductions are temporary and the natural in-situ pressure levels are typically reached within hours after the drilling is completed.

Excess ground settlements immediately after drilling. These findings are likely explained by a loss of soil volume around the casings and was often observed when drilling through silty and sandy soils, or granular material (i.e. a moraine layer) above bedrock. The soil loss might be related to the so-called air-lift pump effect (Behringer, 1930; Kato et al. 1975) as silt and sand particles are eroded and transported to the ground surface. By contrast, the studies reported by Lande et al. (2020), Asplind (2017), and Ahlund and Ögren (2016) indicate that drilling with water driven DTH hammer caused less settlements and excess pore pressures compared to air flushing.

Recommendations on choice of drilling methods and procedures are provided based on results from full-scale field test, experimental modelling, and several case studies with pile drilling in Norway. These recommendations can be used as guidance for practitioners for design (planning), execution and evaluation of overburden drilling to help reduce the risk of damage, thus realize potential cost savings in the building, construction, and property sector. The results should also be implemented in cost-benefit and risk assessments.

The technical risk, time delays and economic risk can be reduced by raising the competence of the stakeholders on the effects of groundwork on, e.g. settlements.

10 Conclusions

The present PhD study shows that drilling for foundation piles and anchors represents rather complex processes, which often contribute to unacceptably large ground movements. The drilling involves continuous rotation and penetration of the drill string, combined with flushing with air or water to remove drill cuttings from the borehole. The natural variations in ground conditions (geological and hydrogeological), quality of workmanship, drilling systems and procedures used make it very difficult to foresee the installation effects and impacts on the surrounding ground.

The main findings and conclusions based on full-scale field tests, small scale model tests and construction projects are summarized in the following.

- Rotary percussive duplex drilling with air flushing (top hammers and DTH air hammers) causes an air-lift pump effect at the front of the drill bit that may lead to significant local erosion and loss of soil mass (i.e. cavities). This effect seems to be particularly problematic when drilling is carried out in erodible soils like silt and sand, typically for glacial tills (e.g. moraine material) often found underlying marine deposits in Norway. Drilling through confined aquifers with artesian pore-water pressures and a high recharge of ground water (as observed in case studies) further increase the risk of excessive erosion which may result in considerable ground displacements and damage in surrounding areas.

- Rapid pore pressure reductions measured in glacial till deposits (moraine materials) during drilling confirm the air-lift pump effect and can be used as an indicator for potential soil volume loss, and as a result, ground settlements.

- Drilling with water-driven DTH hammers greatly reduces the risk of excessive soil volume loss and ground displacements compared to drilling with air hammers. The benefits of this method seem to be prominent in ground conditions with artesian pore-water pressures and aquifers with high ground water recharge as experienced in case study 2.

- Overburden drilling in soft clays is often performed with high penetration rate, thus causing significant soil displacements, and excess pore pressures. Results from field tests and several case studies show that the excess pore pressures in the surrounding clay can become much higher than what is expected for driven, closed-ended piles with the same diameter. However, most of the excess pore pressure seems to dissipate during the first days after drilling, while the remaining dissipates as part of re-consolidation of remolded clay. That may take several months. The soil displacements caused by high penetration may to some extent compensate for other effects that may cause ground settlements, i.e "over-coring" and loss of soil volume, but the effect of disturbance of the clay will cause some settlements due to re-consolidation of this disturbed clay.

- The results from Area C (i.e. DTH water hammer) in the anchor drilling field test give reason to assume that a drilling penetration rate of about 1 m/min reduces unwanted soil displacements and excess pore pressures in the surrounding clays. This is assumed valid for flushing with water and may be different if air flushing is used in soft soils. However, when drilling in sensitive clays from a level well below the ground water level (e.g., in a deep excavation), lower penetration rate may increase the risk of soil collapsing around the drill bit and causing enhanced loss of soil mass.

- Drilling with air flushing may cause uncontrolled outbursts of compressed air along the casing and into the surrounding ground, as observed in Area B (Method 2) in field test with the DTH air hammer. Such outbursts can lead to "over-coring" and cavities around the casings, as well as significant remolding of surrounding clay.

- The proposed method of using normalized flow rate (Q_{norm}) for water drilling can provide guidance to derive optimal drilling parameters. Field results indicate that Q_{norm} between 6-8 seem to represent an ideal drilling in glaciofluvial deposits (e.g. till/moraine).

- Experience from the presented case studies indicate that it's imperative to carry out investigations to map the properties of dense granular soils (e.g. glacial tills) as basis for the choice of piling method. As a minimum information about grain size distributions and hydrogeological conditions should be obtained.

- Monitoring with piezometers and settlement anchors (extensometers) at different levels in the ground is vital for documenting installation effects from drilling and assess the suitability of the drilling method adopted.

- When feasible, drilling of casings for piles should be performed from above the total head ground water level at the transition to bedrock. This is to avoid leakage of ground water up inside open casings and along the outside of the casings through possible leakage paths created during drilling. Injection grouting through the bottom of the casings after drilling of the embedment in bedrock can be used as a mitigating measure.

Further research on the topic of overburden drilling for piles and anchors are recommended to further improve the performance of drilling in the future, and to reduce the risks of damage on third parties from new construction projects.

11 Recommendations for further research

With respect to the topic of overburden drilling for piles and ground anchors, this Phd-study has shown that there has been very limited research on installation effects from drilling in the past (i.e >10 years ago). Despite the great efforts and resources put in the work in the *Limiting Damage* projects, there are still some research questions to address and to follow up on. Recommended future research and work are shortly described in the following.

11.1 Numerical modelling of installation effects of drilling

Today's "best practice" in design of deep excavations or pile foundations does not consider the installation effects from overburden drilling, nor how they may cause ground settlements. However, the empirical data from deep excavations (Figure 1-2) and case studies reported in this thesis clearly suggest that the effects should be addressed in risk assessments of technical solutions and the design.

Future work should investigate in more detail the possibilities of numerical modelling of drilling effects in soft clays. This could involve using large deformation FE-analysis to study the effects on stresses, pore pressures and shear strains in the soil under different degrees of soil displacements during drilling. The results could then be used to assess possible impacts related to stability of nearby slopes or supported excavations due to displacements and strain-softening. The analyses would also provide results to assess long-term re-consolidation of remolded clays around anchors or piles. Based on the studies, a "best practice" for modelling of installation effects from overburden drilling could be developed.

Piciullo et al. (2021) developed a GIS-based tool for rapid assessment of potential building damage due to excavation-induced displacements, named the "GIBV method". This methodology includes Ground-work Impact (GI), in terms of induced greenfield settlements, and the Building Vulnerability (BV). Both short- and long-term displacements are considered in the impact evaluation. The short-term displacements however only accounts for undrained shear deformation of a defined support wall, missing additional effects from installation of tieback anchors if they are used. Future work should aim at implementing effects of soil volume loss due to anchor drilling in the GIBV-method. This would help illustrate the impacts and to carry out risk assessments of different technical solutions. Given the uncertainty in quantifying soil volume loss, it would be reasonable to account for it by including three levels of volume loss, e.g. "low", "medium" and "high". The amount of volume loss could be based on previous back-analysis reported by Konstantakos (2004), Lande (2009) and Sandene et al. (2021 and 2023).

11.2 Verification of drilling methods and procedures

Drilling for piles and anchors is truly complex specialty geotechnical work. It involves several factors such as geological and hydrogeological conditions, different drilling methods and procedures, and not to forget the workmanship which all play an important role in the quality and how it affects the surroundings.

It is highly recommended to continue to document and analyze field data from drilling in different ground conditions and with recommended methods and procedures described in Section 9. It is particularly important to investigate further the effects of drilling with water flushing to enrich the existing empirical database and experience with this method, which at the time being is limited. All new data and experience will help practitioners (project owners, consultants, and contractors) in early-stage risk assessments and choice of technical solutions, as well as planning and execution of overburden drilling to minimize the impacts on the surroundings. Drilling should be carried out under controlled manner and by monitoring the drill parameters so that the feasibility of normalized design framework presented in Figure 8-1 can be validated.

It would also be of great interest to test the Double-head duplex drilling method (as in Case 2) and DTH air hammer under different ground conditions. This setup would enable drilling with the pilot bit positioned somewhat (30-50 cm) above the ring bit on the casing, thus possibly reducing the negative effects as soil erosion from air flushing.

Other drilling methods like the Rotary vibratory drilling, also known as Sonic drilling, should be tested out for installation of smaller casing dimensions. This method would not require air flushing, thus reducing the risks related to such flushing.

It is strongly encouraged to further test out and document the impact on leakage and pore pressure reductions in the ground when alternative methods for sealing against water leakage up along the drill casings are being used.

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Paper I

Lande, E.J. and Karlsrud, K. (2015). ***Full scale field test – drilling of anchors to bedrock in soft clay***. In Proc. of the XVI European Conference on Soil Mechanics and Geotechnical Engineering, Edinburgh 13-17th September 2015. ICE Publishing, London, pp. 625-630.

Paper I

Full scale field test – drilling of anchors to bedrock in soft clay

Tests de terrain à grande échelle : installation d'ancrages dans le roc à travers des argiles molles

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ABSTRACT Recent experiences suggest that drilling for tie-back anchors through soft clays and into bedrock can cause unexpected ground movements and damage to neighbouring buildings. This is an aspect that have been recognized by some researchers in the past, but there has been a lack of specific data. As part of the ongoing R&D project called "BegrensSkade" ("Damage Limitation") a full scale field test program was undertaken to better understand the physical effects of drilling through soft clays, and to investigate the potential for causing ground movements. Five different drilling methods were tested at a test site with soft normally consolidated clay to a depth of 20-25 m. For each method, 8 anchors were drilled from ground level and into bedrock at 45 degrees angle. The anchors were placed in two rows 3 m apart and with a spacing of 2 m between the anchors. Three piezometers and six settlement anchors were installed close to each of the five test areas to document the effects of drilling. Measured results are presented and assessed in relation to type of drilling method applied. Some tentative recommendations are given for how drilling for- and installation of tie-back anchors may be improved in the future to limit the potential for causing ground movements.

RÉSUMÉ Les forages et installations d'ancrage de type tie-back, pour les murs de soutènement, dans le roc et à travers des argiles molles, semblent être l'un des principaux facteurs occasionnant des tassements imprévus et des dégâts aux bâtiments environnants. Les effets des forages dans les argiles ont été documentés par le passé, mais il perdure un certain manque de données spécifiques sur le sujet. Dans le cadre du projet de recherche intitulé "BegrensSkade" (littéralement, "limiter les dégâts"), une étude à grande échelle a été menée afin de comprendre les effets des forages sur les argiles molles, et de déterminer les causes possibles des tassements dans les sols à proximité. Cinq méthodes différentes de forage ont été testées sur un site situé à environ 100 km au Sud-Est d'Oslo. Pour chaque méthode, huit ancrages ont été installés, depuis la surface et jusqu'au socle rocheux, avec un angle de 45 degrés. Les ancrages étaient placés à un intervalle de 2 m selon deux lignes distantes de 3 m. Pour chaque méthode, et afin de suivre les effets des forages, des capteurs de pression ont été placés à trois profondeurs différentes ainsi que 8 capteurs de tassement à une profondeur de 2 m. Les principaux résultats de ces tests ainsi que leur interprétation sont présentés dans cet article.

1 INTRODUCTION

Neighboring buildings or structures often experience unexpectedly large settlements and damage due to foundation works in soft clays. In connection with deep excavations in soft clays in Norway, tie-back anchors drilled into bedrock are commonly used to support sheet pile walls or other retaining structures. It is also common to install bored piles from the bottom of an excavation to support buildings to be erected. Drilling for such anchors and bored piles seems to be a main reason for unexpectedly large ground movements. This aspect has been recognized by some researchers in the past, but the problem has not been addressed in depth (Kempfert & Gebreselassie 1999; Kullingsjö 2007). Some case studies with unexpectedly large settlements are presented and discussed by Langford et al. (2014). The following presents results from a full scale field test program to better understand and document the ground response to drilling for tie-back anchors to bedrock in a soft clay deposit. The main objective was to better understand the physical impact of drilling on the surrounding clay, and the potential for causing ground movements.

2 TEST SITE

2.1 *Ground conditions*

The test site is located on a nearly flat farm field at Onsøy, about 100 km southeast of Oslo. The total area of the test site is around 6000 m^2. The ground elevation is approximately +6 to +7.

At the test site the ground consists of about 13-25 m of homogenous soft normally consolidated marine clay, overlain by 1-2 m of dry crust at the top. The bedrock is partly covered with a thin layer of dense sand/moraine. Typical index parameters for the Onsøy clay is presented in Table 1. The values are based on a summary of results from other research projects conducted, ref. NGI (2011).

The depth to the ground water level is about 0.5 m depth. Measured pore pressures show a slight artesian pressure at bedrock.

Table 1. Typical index parameters, Onsøy clay

Soil parameter	Unit	Value
Density, γ	kN/m^3	15.5-17.5
Water content, w	%	45-70
Plasticity index, Ip	%	30-45
Organic content	%	1-4,3
Undrained shear strength, c_u	kN/m^2	10-40
Sensitivity, St	-	2-30
OCR	-	1.25-1.7
Shear modulus, G_{50}	MPa	0.61-8.13
G_{50}/ c_u	-	100-200

2.2 *Instrumentation*

To measure and document the effects of drilling through soft clay and into bedrock, the test site was extensively instrumented with electrical piezometers (PZ) and Borros settlement anchors. A total of 17 piezometers and 40 settlement anchors were installed in late august 2013.

Figure 1 shows the location of each of the five areas A-E were different drilling methods were tested. The drilling areas and drilling directions were oriented such that potential direct overlapping effects would be minimized. Figure 2 shows a cross sectional illustration of the instrumentation in each test area, and Figure 3 a plan view of the instrumentation at a typical location.

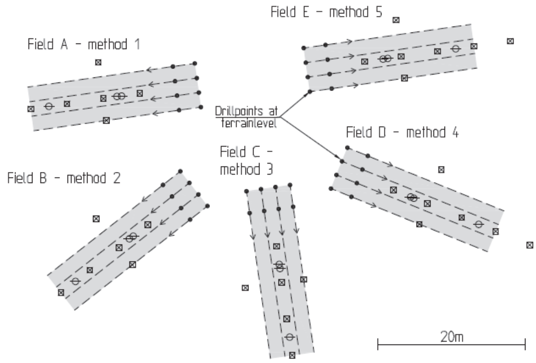

Figure 1. Layout of the test site with different fields and instrumentation for each drilling method.

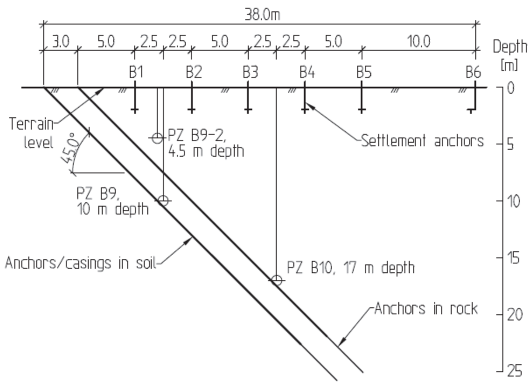

Figure 2. Cross section illustrating instrumentation in each area.

The instrumentation included three piezometers with automatic logging in each test area, and two extra as reference points in field A and E. The piezometers were installed to 4.5 m, 10 m and 17 m along a centerline in the middle of the drilling direction, and with different distance from the borehole, see Figure 2. Due to some variation in depth to the sand/moraine layer three of the piezometers were not installed as deep as planned. Ground movements were monitored by means of 8 Borros type settlement anchors installed at 2 m depth in each test area.

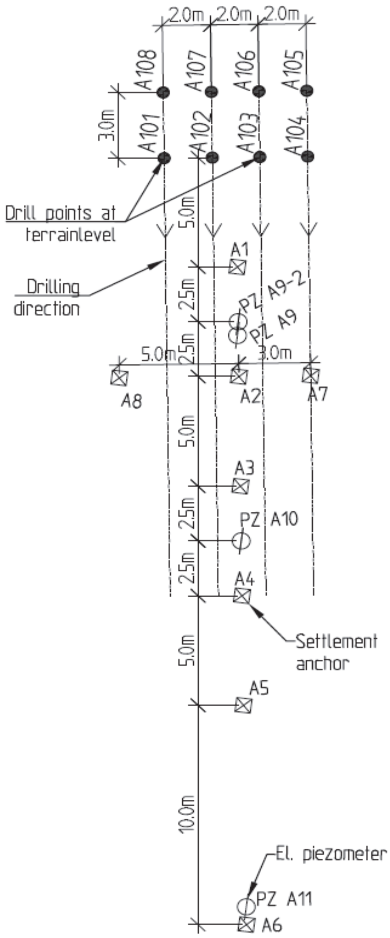

Figure 3. Plan view of instrumentation at each test area/drilling method (example from field A).

3 FIELD TEST – DRILLING OF ANCHORS TO BEDROCK IN SOFT CLAY

3.1 Drilling methods and procedures

The five different drilling methods used at the test site are listed in Table 2. Methods 1, 2 and 4 are most commonly used in Norway for installing anchors and bored steel-core piles into bedrock. There was less experience with methods 3 and 5. It was assumed that method 3 would be "gentlest" when it comes to the potential for disturbance of the clay and creating erosion and cavities up along the drill string.

The eight anchors installed with each drilling method were generally set with 45 degrees inclination. The anchors were placed in two rows with a distance of 3 m between the rows and center distance of 2 m between anchors in a row (Figure 3).

Table 2. Overview drilling methods

Field	Drilling method	Period of drilling
A	1 – Self-drilling injection anchors, dimension 40/16	19.09.2013 –24.09.2013
B	2 - Odex 115 DTH air hammer	17.10.2013 – 22.10.2013
C	3 - Odex 115 DTH water hammer (Wassara)	27.11.2013 - 02.12.2013
D	4 - Odex 90/76 with top hammer	16.10.2013 – 17.10.2013
E	5* - OD 114.3 centric ring bit Ø120 with top hammer	30.10.2013 – 31.10.2013

*Drilling method where the borehole was grouted with cement (V/C = 0.4) up to terrain and casings pulled up afterwards.

The drilling procedures applied were consistent with what the drilling contractors defined as "best practice". The methods are briefly described as follows:

- When drilling through the soft clay only water flushing was used with all methods. Pressure 5-20 bars, 60 l/min.
- Compressed air was only used with method 2 when drilling though the dense sand/moraine and further in to bedrock.
- Typical penetration rates through clay were 10-20 sec/m for method 1, 5 sec/m for method 2, 4 and 5, and 30-60 sec/m for method 3.
- The rotation speed of the drill bit was around 60 rpm for method 1, 2 and 3, and 120 rpm for method 4 and 5.

All the anchors were drilled into bedrock to simulate real projects, except two anchors in field A. In field D only 2 out of 8 anchors (D104 and D103) were installed because drilling into moraine/bedrock was not possible. The main reason was probably a combination of depth and that the top hammer used could not supply sufficient energy.

Figure 4 shows examples of drill bits that were used for drilling method 1 (self-drilling Ischebeck anchors) and method 2/3 (Odex 115 system). The drill bit for Odex 90/76 used with method 4 are the same as for Odex 115, only with less outer diameter (OD).

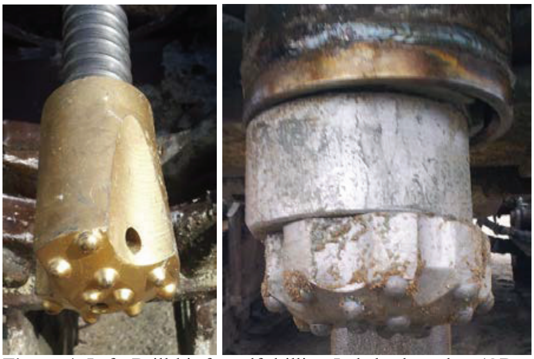

Figure 4. Left: Drill bit for self-drilling Ischebeck anchor (OD = 70 mm, 40 mm for the anchor rod). Right: Casing (OD = 139.7 mm) and eccentric drill bit (OD = 151 mm) for Odex 115 system.

4 RESULTS

4.1 Pore pressure measurements

Pore pressure were measured continuously at the test site for about 8 months. Figure 5 a) and 5 b) present measured pore pressures versus time when drilling in field B with Odex 115 DTH air hammer. The data shows that drilling of anchor B104 caused pore pressure changes also in field A, at distances of approximately 30 m from anchor B104. This was most likely caused by flushing with high air pressure when drilling through moraine and into bedrock.

During drilling of anchor D103 in field D a decrease of around 15 kPa was measured in PZ D10. This was most likely caused by some minutes of flushing with high pressure air when trying to penetrate in to bedrock.

Figure 6 summarizes maximum pore pressure changes (ΔU_{max}) recorded during drilling. The pore pressure changes go up to 70 kPa about 1m from the drill string (drilling method 5, PZ E9), and the influence zone extends more than 5 m in some cases.

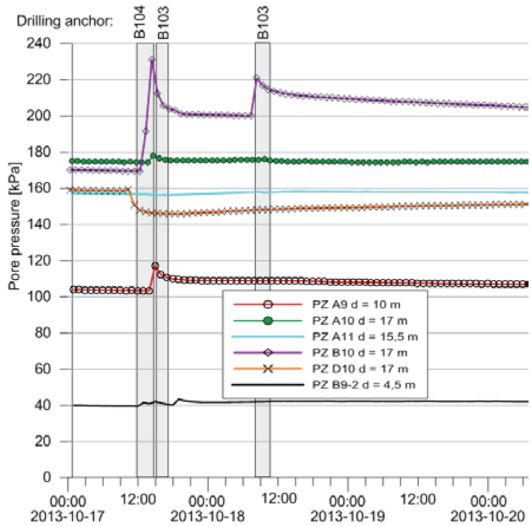

Figure 5 a). Pore pressure measurements during drilling in field B with Odex 115 DTH air hammer.

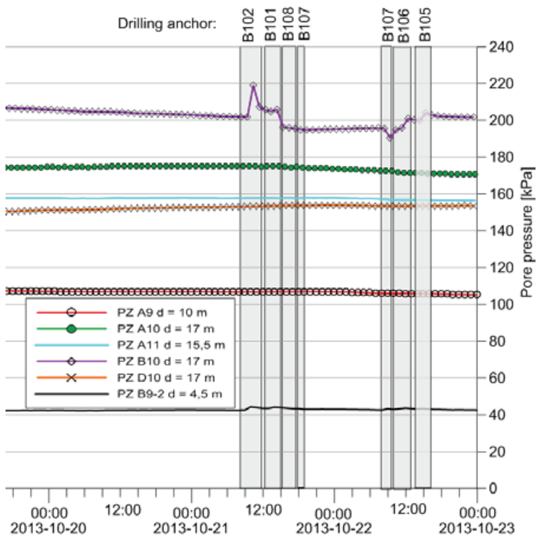

Figure 5 b). Pore pressure measurements during drilling in field B with Odex 115 DTH air hammer.

Most of the generated excess pore pressures in PZ B10 and PZ E9 dissipated rapidly during the first days after end of drilling. However, it still was about 5-10 kPa higher than the in-situ pore pressure about 5 months after drilling.

The results in field A are effected by a change of inclination from 45° to 56° for six of the anchors Hence, the theoretical minimum distance between the piezometers and the anchors became larger than originally planned.

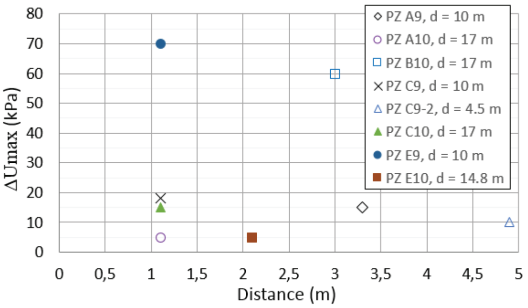

Figure 6. Measured maximum pore pressure change (ΔUmax) versus minimum theoretical distance to drill string.

4.2 Measured settlements

All 40 settlement anchors at the test site were regularly monitored over a period of 9 months, starting about 2 weeks prior to drilling in field A and ending in the beginning of June 2014. Measuring intervals were shortest during and right after drilling to document any immediate effects. Figure 7 a) and 7 b) presents settlement versus time for anchor no. 4 (Figure 3) in each field. The grey lines marked A-E shows the times when drilling took place in each field.

The results generally show rather small settlements for all drilling methods, with a maximum value of 12 mm in field B. There is also some variation most likely due to measurement accuracy of ±1-2 mm. The data clearly indicate that drilling in field B (and D) with Odex DTH air hammer caused almost immediate settlements in the range of 2-7 mm over the entire test site, see Figure 7 a).

After installation in field B was completed, subsequent measurements until June 2014 showed no clear further settlements in field A, C, D or E. The exception was field B, where settlements increased by 2-6 mm over a period of 3 months (between 7th January and 4th April 2014), see Figure 7 b). During the same time period some of the remaining excess pore pressure (about 5-10 kPa) dissipated at location of piezometer PZ B10.

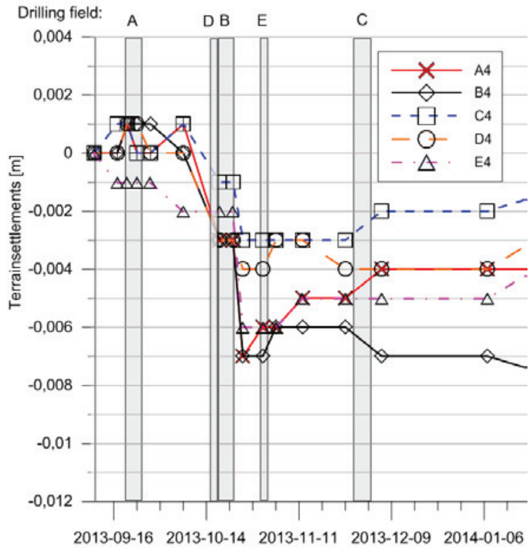

Figure 7 a). Measured settlements at anchor no. 4 for all drilling methods on the test site. Time period 2013-09-10 to 2014-01-07.

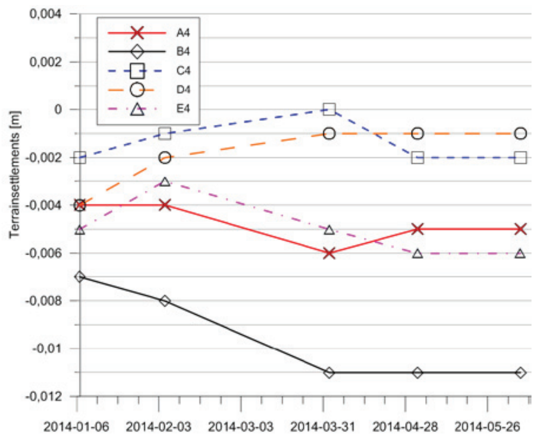

Figure 7 b). Measured settlements at anchor no. 4 for all drilling methods on the test site. Time period 2014-01-07 to 2014-06-06.

4.3 Measured volume of cuttings during drilling

Based on measurements and observations during drilling at the test site, the total volume of soil cuttings per anchor was estimated for each drilling method. Compared to the theoretical gross volume of casings in the ground, the volume of soil taken out was for drilling methods 2 (field B) and 5 (field E) generally smaller. This may partly explain why the excess pore pressures were largest for these two drilling methods (Figure 6). The cuttings generated with drilling methods 1 (field A) and 3 (field C) tended to be larger than the volume represented by the casings, (e.g. net volume loss or "over-coring") which may explain the smaller excess pore pressures (Figure 6).

5 SUMMARY

Both measured pore pressures and settlements suggest that drilling with Odex 115 DTH air hammer as with method 2 in field B, caused significantly larger excess pore pressures (up to 60 kPa) and surface settlements than for all the other methods. That the surface settlements in field B continued to increase over several months after drilling indicates that settlements were mostly associated with dissipation of excess pore pressures and re-consolidation of possibly partly disturbed clay around the drill string. The settlements must still be considered rather small (maximum 12 mm). The other 4 methods had quite similar and smaller impact on pore pressures and settlements, and the settlements stopped shortly after drilling in these areas.

Variations in measurements between the different methods are considered to be mainly related to penetration rate and use of flushing medium, especially air flushing. Apart from some variations in the depth to bedrock, the marine clay deposit across the site is considered very homogeneous. Variability in soil conditions are therefore, not likely to explain the difference in settlements observed for the different drilling methods.

The relatively small observed settlements generated in this test area stand in strong contrast with the large settlements (up to 40 cm) that were reported by Langford et al (2014) around excavations in soft clays supported by tie-back anchored sheet pile walls. The two main reasons for this difference are probably:

i) When drilling from a level below ground level as within an excavation, the unbalanced pressure between vertical ground stress and water or air pressure at the head of the drill string, will increase as compared to when drilling from ground level. This will enhance the potential for over-coring, disturbance of the surrounding clay and erosion up along the drill casing. To limit such effects this study suggests that the rate of penetration should be about 60-90 sec/m. Avoiding use of air during flushing also seems essential.

ii) If gaps are created between the drill casing and the surrounding soil, it will effectively act as a drainage path for groundwater, causing reduced pore pressures and consolidation settlements within the surrounding clay. This has been documented to cause wide spread pore reduction extending several hundred meters away from excavations and large settlements (Langford et al. 2014). Using driven rather than bored casings or grouting through the end of the drill string after completion of drilling may be ways to reduce this problem.

ACKNOWLEDGEMENT

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Paper II

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Case Study

Effects of Drilling for Tieback Anchors on Surrounding Ground: Results from Field Tests

Einar John Lande[1]; Kjell Karlsrud[2]; Jenny Langford[3]; and Steinar Nordal[4]

Abstract: A full-scale field test program was carried out to investigate the effects of drilling for tieback anchors on the surrounding ground. The test anchors were drilled from the ground surface through a soft clay deposit and into bedrock. Five different drilling methods were compared. All methods caused excess pore pressures in the surrounding clay, up to 70 kPa, extending several meters away from where drilling took place. This impact on pore pressures was for most drilling methods significantly larger than what has been observed for driven piles in clay. High penetration rate combined with water flushing during drilling through soft clay is the main reason for the effects on the pore pressure. Drilling with a down-the-hole hammer and air flushing through a layer of moraine and into bedrock in one of the test areas (Area B) caused significantly larger excess pore pressures and ground settlements than the other drilling methods. Approximately half of the maximum resulting settlements of 12 mm in Area B was most likely caused by reconsolidation of remolded clay around the casing tubes. Drilling with water-driven hammer in Area C had less effect on both pore pressures and ground settlements. **DOI: 10.1061/(ASCE)GT.1943-5606.0002274.** *This work is made available under the terms of the Creative Commons Attribution 4.0 International license, https://creativecommons.org/licenses/by/4.0/.*

Author keywords: Drilling; Piles; Anchors; Settlements; Pore pressure; Remolding; Flushing.

Introduction

It is well established that deep supported excavations in soft clay deposits can cause significant ground settlements in the areas surrounding the excavation, ranging from approximately 0.5% to 2% of the final excavation depth H (e.g., Peck 1969; Mana and Clough 1981; Karlsrud and Andresen 2008). Recent experience, however, shows that ground settlements caused by initial and secondary effects from the installation of drilled tieback anchors and bored piles from inside an excavation can be significantly larger than 2% of the excavation depth (Langford et al. 2015). For excavations in urban areas, deformations of such magnitudes imply a large potential for causing damage to neighboring buildings and structures and for the associated large liability potentials. A recently completed research project in Norway focused on understanding and identifying the causes of excessive settlements associated with excavations and foundation works (Baardvik et al. 2016).

Although negative installation effects related to drilling for ground anchors and piles in varying soil conditions is recognized in some literature, e.g., Kempfert and Gebreselassie (1999), Kullingsjø (2007), Konstantakos et al. (2004), and Bredenberg et al. (2014), the problem has not been systematically addressed and studied. There is specifically a lack of knowledge related to the effects of drilling on the surrounding ground and the extent to which it may cause ground movements. Disregarding some general guidelines related to the design and implementation of drilled piles (Finnish Road Authorities 2003; FHWA 2005), the authors have not found specific guidelines for selecting appropriate drilling methods or installation procedures to reduce the risk of excessive ground movements.

Kempfert and Gebresellassie (1999) reported excessive settlements and damage on an adjacent building due to drilling and pulling of casings for tieback anchors as support for an up to 7.0-m-deep excavation in soft lacustrine clay. Konstantakos et al. (2004) reported a case study from an up to 23-m-deep excavation in Boston. The excavation was supported by a 0.9-m-thick diaphragm wall embedded in bedrock and four to six levels of poststressed tieback anchors into bedrock. A maximum of 65-mm ground surface settlements was recorded on the outside of the excavation. The excessive settlements were explained by local cavities and loss in soil volume around the anchors during drilling through sand and silt layers. The hypothesis was confirmed by finite-element analyses, corresponding to a loss in soil volume of approximately 0.36–0.50 m^3/linear meter of the supported diaphragm wall. Results from the analyses agreed well with monitoring results. Details regarding drilling method and execution were, however, not presented.

Kullingsjø (2007) presented monitoring data for a deep supported excavation in soft clay in Gothenburg, Sweden. Results from inclinometers on the sheet pile wall and extensometers installed in the ground behind the wall clearly indicate that drilling of casings for tieback anchors caused loss of soil volume (cavities) in a silty sand layer just above bedrock. The volume loss resulted in significant large ground settlements of up to 40 mm, approximately 0.4% of the excavation depth. Bredenberg et al. (2014) describes a case record from Stockholm, Sweden, where casings with an outer

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diameter of 168 mm (OD = 168 mm) were drilled for steel core piles through soft marine clay and into bedrock. Drilling was carried out with a so-called down-the-hole (DTH) air hammer and a concentric drill bit designed to reduce the risk of cavities due to high-pressure air flushing. Monitoring data showed that drilling with the new concentric drill bit caused from 10- to 15-mm settlements on nearby basement floors. That was approximately 70% less settlement compared to an adjacent construction project with similar ground conditions where a conventional eccentric drill bit was used. Rønning (2011) gives a brief description of monitoring results from a test installation for a bored steel pipe wall (OD = 610 mm) in quick clay in Trondheim, Norway. This field study showed that it is possible to drill through sensitive and quick clay causing only a limited mechanically remolded zone close to the pile wall. Total pressure sensors installed at the pile tip showed maximum excess pressure of approximately twice the effective overburden stress during drilling. Piezometers installed 0.5 m from the pile wall at depths of 6 and 13 m showed excess pore pressures of 23 and 25 kPa, respectively.

The main scope of the full-scale field test program described herein was to identify the main so-called installation effects and better understand the ground response to drilling for tieback anchors through a soft clay deposit and into bedrock. The study seeks to investigate the difference between five drilling systems, including with or without casing and with air-driven and water-driven hammers. The field trials and the primary results were briefly presented by Lande and Karlsrud (2015) but are assessed in more detail in this paper.

Drilling Methods

In Scandinavia the use of tieback anchors and piles (both micropiles and large-diameter steel pipe piles) that are drilled through soils and into bedrock has increased significantly during the last decades. There are several reasons for this. First, typical ground conditions with soft clay overlying solid bedrock favor anchors and piles to bedrock due to the considerably larger capacity compared to soil anchors and friction piles. Second, contractors often prefer tieback anchors instead of internal struts for deep excavations due to more efficient excavation and construction processes. Third, installation of piles by drilling can be performed efficiently using relatively small, lightweight drill rigs. Finally, piles installed by drilling and grouting into bedrock can resist both axial compression and tensile forces.

Many drilling methods and systems are available for drilling tieback anchors and piles in diverse ground conditions. The drilling method has traditionally been selected on the basis of efficiency and cost of construction, and often with less focus on minimizing soil disturbance and damage to the surroundings. According to the Federal Highway Administration (FHWA 2005), drilling can be divided into two main categories: open hole drilling, i.e., without casing, or with a continuous casing supporting the borehole. The latter is referred to as overburden drilling in this paper. Drilling in soft and sensitive soil often requires the use of a casing to support the borehole. Fig. 1 illustrates three systems for overburden drilling using the rotary percussive duplex drilling method (FHWA 2005), where the drill bit is both percussed and rotated. Fig. 1(a) shows a hydraulic powered top drive (top hammer) with an eccentric drill bit where both rotation and percussion are applied at the top of the drill rod by the drill head of the rig. Figs. 1(b and c) show examples of DTH hammers where a percussion hammer is located just above the drill bit, and the drill rod is rotated by the drill head. DTH hammers are driven by compressed air or water with high pressure. Both top-hammer and DTH drilling methods use continuous flushing with compressed air or water to remove soil cuttings from the front of the drill bit and transport them up to the ground surface through the annulus between the casing and the drill rod. Fig. 1(c) illustrates a reversed circulation (RC) drilling system with a double-tubed drill rod (dual wall) where the cuttings and flushing returns along the inner tube.

Many different drill bits are in use for different ground conditions and applications, most of them available for both top hammers and DTH drilling. The traditional eccentric drill bits are the most commonly used for overburden drilling. The system consists of a concentric pilot bit in the front followed by an eccentric reamer with slightly larger diameter than the casing, illustrated in Figs. 1(a and b). During penetration, a guide device on the drill bit acts on a casing shoe that is welded to the bottom of the casing, pulling down the casing. A disadvantage with the eccentric system is that the reamer may cause a gap between the casing and the borehole wall. This gap increases the risk of compressed flushing air escaping up through the gap, resulting in excessive erosion and disturbance of the surrounding soil.

To mitigate some of the shortcomings of eccentric systems, concentric systems have been developed. The concentric system consists of a pilot bit in the center, a casing shoe that is welded to the casing, and a symmetrical ring bit that is locked onto the pilot bit, illustrated in Fig. 1(c). The ring bit drills a borehole slightly larger than the outer diameter of the casing, allowing the casing to advance. The face of the pilot bit is placed almost in line with the ring bit, which facilitates keeping the borehole in its desired alignment and also in entering an inclined bedrock surface. During the past 10 years or so, several manufacturers have developed new drill bits to minimize overcoring effects. The main concept with these concentric drill bits is to redirect the air flow at the front of the bit, to limit compressed air from evacuating into the ground, and creating unwanted cavities, thereby reducing the risk of settlements.

Field Test—Drilling of Anchors through Soft Clay and into Bedrock

Test Site

The field test was carried out on a nearly flat agricultural field at Onsøy, approximately 100 km southeast of Oslo, Norway. The ground elevation varied from 6 to 7 m above sea level (masl) within the site, which had a total area of approximately 6,000 m^2. The site was approximately 150 m from where pile load tests had been carried out previously and where the ground conditions already were well known and documented (Karlsrud et al. 2014).

Fig. 2 presents a layout of the test site with the location of the five areas (A–E) where different drilling methods were tested. The layout gives an overview of boreholes and drilling directions for each anchor, as well as instrumentation installed to document the effects of each drilling method. The directions of drilling were for each method oriented such that the potential overlapping effects would be minimized.

Ground Conditions

The ground at the test site consists of approximately 0.5 m organic topsoil over 1.0–1.5 m dry crust. Underneath the dry crust is a layer of homogeneous soft, normally consolidated marine clay. The thickness of the clay deposit increases from approximately 13 m in Area E (northeast) to approximately 23 m in Areas B and C (southwest). Fig. 3 presents a typical soil profile with index data and in situ stress conditions for Onsøy clay, based on soil investigations carried out at the site for pile load tests (Karlsrud et al. 2014).

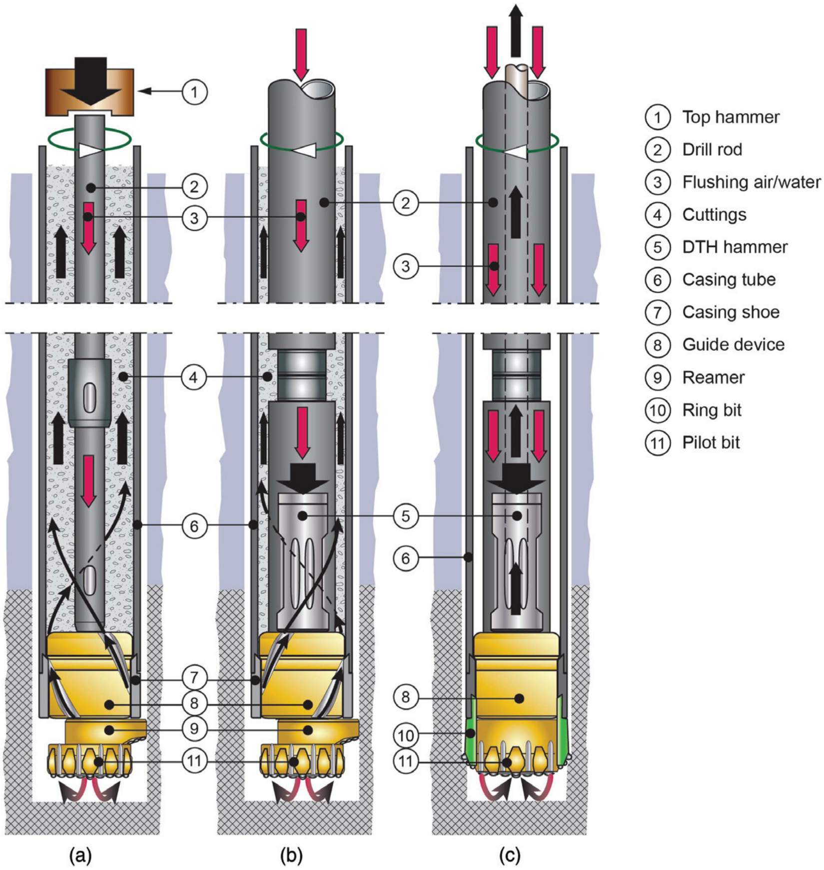

Fig. 1. Rotary percussive duplex drilling methods and drill bits used for overburden drilling: (a) top drive (top-hammer) eccentric; (b) DTH hammer eccentric; and (c) reverse circulation (RC) DTH hammer concentric.

With clay content in a range between 44% and 66% combined with plasticity index data, the clay is classified as medium to highly plastic. The bedrock is partly covered with a thin layer of dense sand/moraine. Observations during drilling of the anchors showed that the thickness of sand/moraine is 200–300 mm in Areas A, C, and E and up to approximately 2 m at some of the anchors in Areas B and D (Fig. 2). The groundwater level is registered at a depth of approximately 0.5–1.0 m below the ground surface. Measurements show that there is a slight artesian pore water pressure of 10–20 kPa at bedrock.

Instrumentation

To be able to measure and document the effects of drilling, the test site was instrumented with electrical piezometers (PZ) and settlement anchors. A total of 17 piezometers and 40 settlement anchors were installed approximately 3 weeks before the first tests started. Fig. 4 shows the typical layout and cross section of the instrumentation installed at each test area, here represented by an example from Area B. Three piezometers with automatic logging were installed at depths of 4.5, 10, and 17 m in each test area. All were placed along the middle section of each test area, and at different distances from the boreholes (Fig. 4). Two extra piezometers were installed as reference points in Areas A and E. Due to the smaller depth to bedrock in Area E (between 13 and 16 m), Piezometers E10 and E11 (reference point) were installed to depths of 14.8 and 13.2 m, respectively, both with the tip just above the bedrock.

Ground settlements were monitored by means of eight Borros-type settlement anchors (Geokon 2019) installed at a depth of 2 m within each test area. Settlements of the anchors were measured using a total-station type theodolite. A bedrock outcrop approximately 100 m east of the test site was used as reference point.

An attempt was made to measure the volume of drill cuttings during drilling of some of the boreholes (anchors) and to compare this to the theoretical volume of the casings installed in the ground. In practice, this turned out to be very difficult and it was not possible to get accurate measurements. However, based on observations during drilling, it was possible to estimate whether drilling caused loss of soil volume or soil displacement.

Drilling Methods and Procedures

Table 1 presents details regarding the five drilling methods used in the field test, including the time when drilling was carried out. While Methods 2, 3, and 4 are commonly used in Scandinavia for overburden drilling through soft clays for both ground anchors

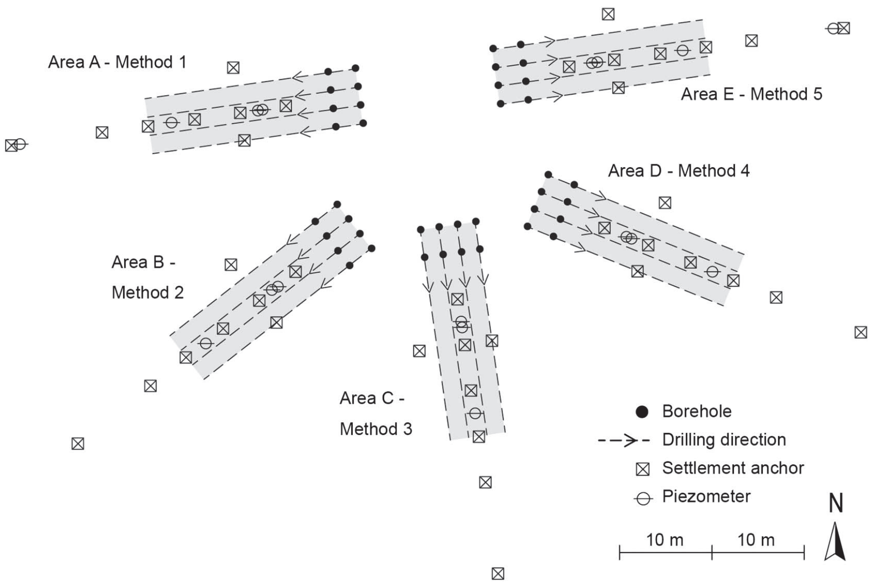

Fig. 2. Layout of test site with Areas A–E and related drilling methods.

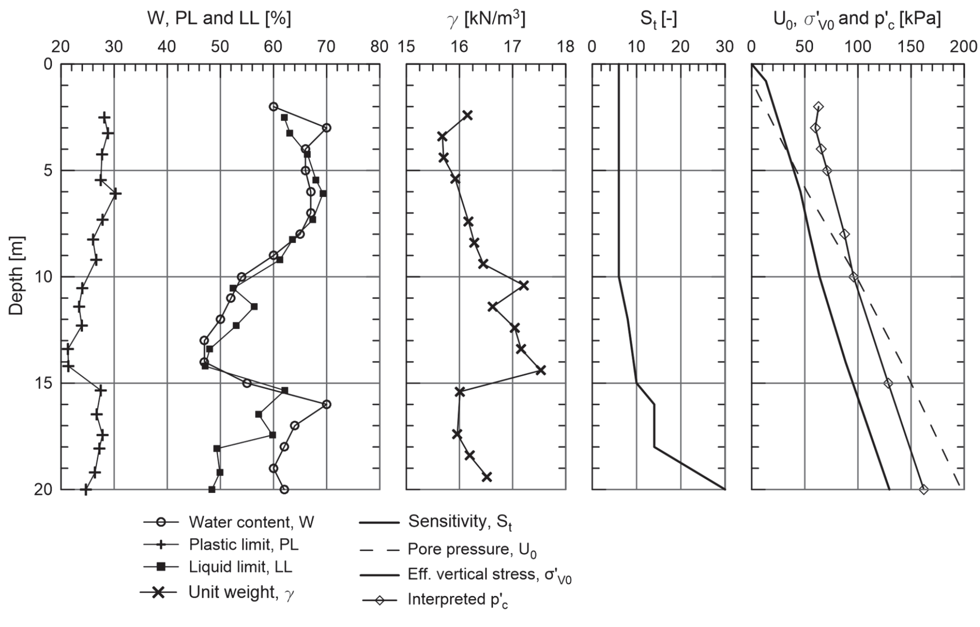

Fig. 3. Index data and interpreted in situ stress conditions for Onsøy clay from pile load test site. (Adapted from Karlsrud et al. 2014.)

and micropiles, there is limited experience with the other methods. Fig. 5 show pictures of the different drilling systems and drill bits that were tested. Drilling in Area A was carried out with an uncased system using 40-mm hollow-core steel bars with a 70-mm rock drill bit [Fig. 5(a)]. The same type of eccentric drill bit was used in Areas B, C, and D [Fig. 5(b)], however with a smaller dimension in Area D. In Area E, a system with a concentric drill bit and a ring bit was used [Fig. 5(c)].

For each drilling method, a total of eight "anchors" were drilled from the ground surface at a 45° inclination, through the soft clay,

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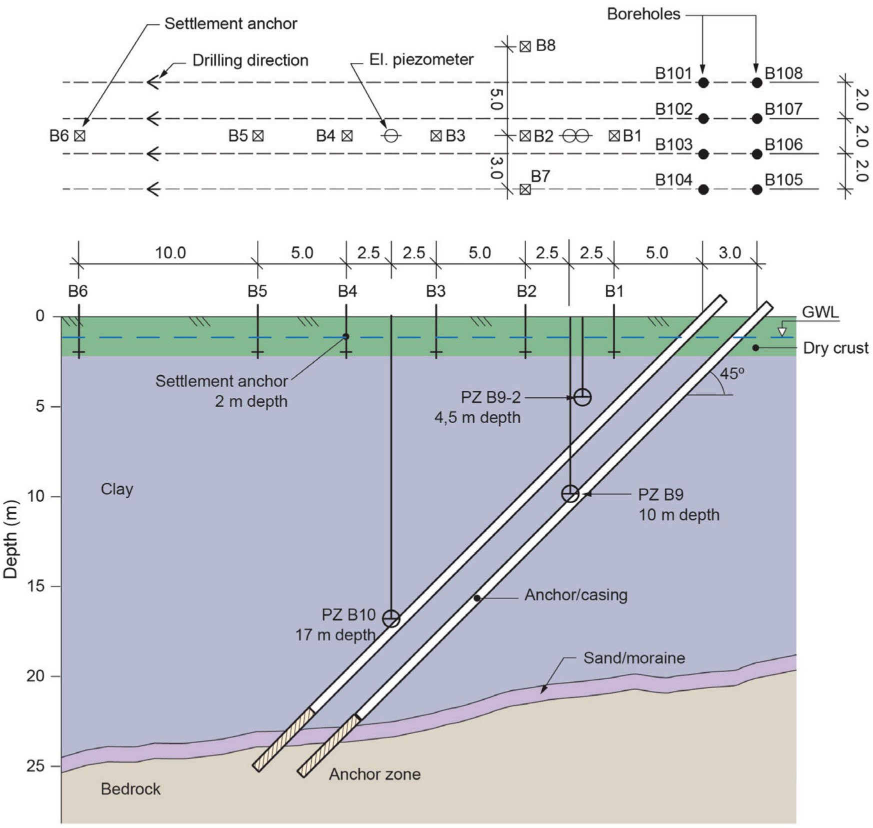

Fig. 4. Layout and cross section of instrumentation for each drilling method (example from Area B).

Table 1. Overview of drilling methods used in field test

Area	Drilling method	OD (mm)		Period of drilling
		Casing (mm)	Reamer (mm)	
A	1—Top hammer with hollow core steel bars	—	—	September 19–24, 2013
B	2—DTH air hammer with eccentric drill bit	139.7	151.2	October 17–22, 2013
C	3—DTH water hammer with eccentric drill bit	139.7	151.2	November 27, 2013–December 2, 2013
D	4—Top hammer with eccentric drill bit	114.3	123.0	October 16–17, 2013
E	5—Top hammer with concentric drill bit	114.3	120.0	October 30–31, 2013

Note: OD = outer diameter.

the thin layer of dense sand/moraine, and into bedrock. The anchors were placed in two rows 3 m apart and with a spacing of 2 m between the anchors (Fig. 4). The test program did not include installation or poststressing of any anchor tendons/strands in the casings since the focus was purely on overburden drilling.

Table 2 presents typical values for the main drilling parameters for the different methods that were tested. The drilling length in bedrock for each method is also given. All drilling with Methods 1, 3, 4, and 5 was carried out using continuous water flushing. With Method 2, however, air flushing with approximately 1,200–1,500 kPa (12–15 bar) air pressure was used to run the DTH hammer and to penetrate through the layer with sand/moraine and into bedrock. With Methods 1 and 5 the top hammer was used to drill through the moraine and into bedrock. After the drilling was completed, the boreholes were filled (grouted) with a cement suspension (water-to-cement ratio of 0.4–0.7). The grout was pumped at low pressure through the drill rod, filling the borehole from the bottom up to the ground surface. With Method 5 the casing was pulled up directly after grouting, leaving the borehole supported only by the grout.

To replicate a typical production drilling scenario, the penetration rate through clay was generally high except with Method 3, where the rate was reduced significantly compared to the other methods (Table 2). The intention was to minimize excess pore pressures due to soil displacement, as observed with the other methods.

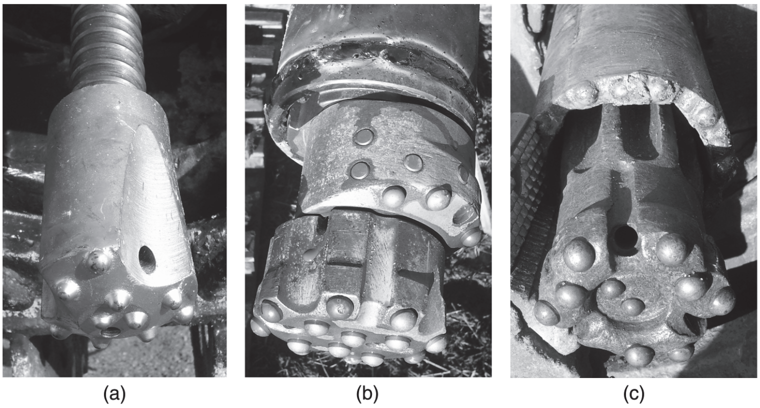

Fig. 5. Pictures of different drilling systems and drill bits used in field test. (Images by Einar John Lande.)

Table 2. Typical drilling parameter values in field test

Drilling parameter	Unit	Drilling method/Test area				
		1/A	2/B	3/C	4/D	5/E
Water pressure (clay)	kPA (bar)	500 (5)	2,000 (20)	9,000 (90)	500 (5)	500 (5)
Water flow rate (clay)	L/min	60	60	150–200	60	60
Penetration rate in clay	m/min	3–6	12	1–2	12	12
Rotation speed	rpm	60	60	60	120	120
Drilling length in bedrock	m	0–2.5	4	0.4–1.4	0	1.85–4.2

The objective was to drill all boreholes for the anchors 4 m into bedrock and to provide a casing for the first 0.5 m (Methods 2, 3, 4, and 5). However, for practical reasons, and to save time, the actual length of drilling into bedrock was reduced. The first two drill rods (anchors) with Method 1 broke trying to enter bedrock. To mitigate this problem, it was decided to drill the remaining six anchors steeper at approximately a 56° inclination. The first three anchors with Method 1 were not grouted because of problems with drill rod clogging and because one of the rods that broke sank into the borehole. With Method 4, it was decided to abort the test after drilling only two out of eight casings (D104 and D103). This was because drilling through the dense moraine layer and into bedrock was not possible with this drilling system. The main reason was probably a combination of depth to moraine (approximately 18 m) and that the top hammer and drill rod that were used could not supply enough energy. Drilling through the soft clay was carried out with water flushing, but additional flushing with pressurized air was used when trying to improve drill cutting transport and penetration into the dense moraine.

Results

Pore Pressure

This section presents the main results from pore pressure measurements. All piezometers were logged continuously over a total period of approximately 8 months. To establish representative reference values, the piezometers were installed approximately 4 weeks before the drilling in Area A commenced. Data were logged at 1-h intervals during the whole test period and changed to one per day when all drilling was completed.

Monitoring data show that all the drilling methods caused excess pore pressures in the surrounding clay. The observed response on the pore pressure was generally much the same with all methods. However, Method 2 in Area B (DTH air hammer) and Method 5 in Area E (top hammer and concentric drill bit) resulted in significantly higher excess pore pressures than the other methods. Table 3 gives a summary of maximum excess pore pressures registered at each test area. The largest observed excess pore pressure was 70 kPa in PZ E9 at a depth of 10 m while drilling of anchor (casing) E107 in Area E with a minimum distance of approximately 1.1 m to the piezometer. The main reason for the large excess pressure was likely the high penetration rate, approximately 12 m/min, combined with pressurized water flushing (Table 2). This resulted in the highest ratio of maximum excess pore pressure ($\Delta U_{\max}$) to the effective overburden stress (σ'_{V0}) of all the piezometers with a value of 1.14 (PZ E9). The minor changes in PZ E9-2 and PZ E10 (ΔU between 0 and 5 kPa) was most likely due to the much greater distance between the casings and the piezometers as well as the relatively small dimension of the anchors (OD $=$ 114 mm).

Measurements in Area A showed relative moderate changes in pore pressures with a maximum value, $\Delta U_{\max} = 15$ kPa in PZ A9. The results were likely affected by the relatively small dimensions of the drill bit (OD $=$ 70 mm) and the change in inclination from 45° to 56° for the last six anchors, thereby increasing the theoretical minimum distance between the piezometers and the anchors.

Fig. 6 shows changes in pore pressure (ΔU) with respect to time during drilling in Area B [Fig. 6(a)], Area C Fig. 6(b)], and Area D [Fig. 6(c)], respectively. Time of drilling for each individual anchor is indicated with gray bars in the figures. Fig. 6(a) show that drilling of Anchor B104 caused an immediate excess pore pressure of approximately 60 kPa in PZ B10 at a depth of 17 m, while PZ B9-2 at a depth of 4.5 m showed only minor change ($\Delta U = 1$–2 kPa). No data were available from Piezometer B9, which was out of function during the field tests. Drilling of Anchor B104 also caused an increase of approximately 13 kPa in PZ A9 (10 m depth) and 4 kPa in PZ A10 (17 m depth) in Area A, at a distance of around 30 m from Anchor B104. The excess pressures were most likely caused by flushing with compressed air [1,200–1,500 kPa (12–15 bar)] when drilling through the sand/moraine layer above bedrock. Small outbursts of air, water, and remolded clay were observed up along the

Table 3. Summary of pore pressure data

Area	PZ No.	Depth (m)	U_{ref} (kPa)	σ_{V0} (kPa)	σ'_{V0} (kPa)	ΔU_{max} (kPa)	$\Delta U_{max}/\sigma'_{V0}$	$\Delta U_{max}/U_{ref}$
A	A9	10	98	162.5	64.5	15	0.233	0.153
	A9-2	4.5	38	73.5	35.5	4	0.113	0.105
	A10	17	171	278	107	3	0.028	0.018
	A11	15.5	155	—	—	—	—	—
B	B9	10	—	162.5	—	—	—	—
	B9-2	4.5	40	73.5	33.5	4	0.119	0.100
	B10	17	170	278	108	60	0.556	0.353
C	C9	10	101	162.5	61.5	18	0.293	0.178
	C9-2	4.5	42	73.5	31.5	10	0.317	0.238
	C10	17	175	278	103	18	0.175	0.103
D	D9	10	101	162.5	61.5	8	0.130	0.079
	D9-2	4.5	40	73.5	33.5	0	0.000	0.000
	D10	17	163	278	115	−17	−0.148	−0.104
E	E9	10	101	162.5	61.5	70	1.138	0.693
	E9-2	4.5	40	73.5	33.5	3	0.090	0.075
	E10	14.8	150	245	95	5	0.053	0.033

outside of the casing as well as along the previously installed Anchor Rods A104 and A103 in Area A. This shows that drilling with compressed air caused pneumatic fracturing, not only along the casing wall but through the moraine layer.

Despite the closer proximity to PZ B10, drilling of Anchor B103 and B102 had less impact on the excess pressures than Anchor B104. The results indicate that some of the flushing air evacuated through the moraine and joints/fissures in bedrock and up into the casing for Anchor B104, rather than building pressures in the ground as with Anchor B104. This mechanism was also observed for some of the other anchors in Area B.

Drilling of Anchors B101, B108, and B107 reduced the excess pore pressure in PZ B10 with approximately 15 kPa in total. This reduction could be caused by groundwater that was sucked into the casings with the backflow (drill cuttings) when drilling into bedrock. Based on visual observations, the amount of water is roughly estimated to be between 20 and 30 L/min.

Piezometer B9-2 at a depth of 4.5 m showed insignificant changes, with a maximum accumulated excess pore pressure of approximately 4 kPa during drilling in Area B. The longer distance between PZ B9-2 and the anchors compared to PZ B10, combined with lower soil stress at shallow depth, may explain this difference in response.

Fig. 6(b) shows that drilling with the DTH water hammer in Area C resulted in considerable lower excess pore pressures in the surrounding clay compared to Areas B and E. The major difference is reasonable, considering the lower penetration rate when drilling through the soft clay in Area C (Table. 2). Drilling of the first four anchors in Area C (C104 to C101) had minor influence on the piezometers except PZ C10, which showed an accumulated increase to a maximum value of 18 kPa after drilling of Anchors C102 and C101. The excess pressure then decreased and was almost unaffected during drilling of Anchors C108 to C105 because of the greater distance to the casing. PZ C9 showed, however, excess pressure of approximately 18 kPa while drilling of Anchors C107 and C106 with a minimum distance of approximately 1.1 m from the casings. Piezometer C9-2 at a depth of 4.5 m showed an approximately 10-kPa increase in pore pressure during drilling of Anchor C105, even with a minimum distance of approximately 5 m to the casing. This was—two to three times higher compared to the piezometers at a depth of 4.5 m in the other test areas. The difference from the other drilling methods could be related to the significantly higher water pressures and flow rates used during drilling in clay [150–200 L/min at 6,000–8,000 kPa (60–80 bar) from the water pump]. The flushing might have caused some hydraulic fractures in the upper part of the clay, extending the influence zone.

The measurements in Area D (Method 1) are not directly comparable with those obtained using the other methods since the test was aborted after drilling of the first two anchors (casings). The results are, however, interesting with respect to the installation effects from drilling. Fig. 6(c) shows that drilling of Anchor D104 resulted in a pore pressure reduction of approximately 3 kPa in PZ D10 (17 m depth) and PZ E10 (14.8 m depth), which decreased to approximately 5 kPa during the following 24 h. The pore pressure in PZ D10 reduced further to a minimum value of approximately 17 kPa right after drilling of Anchor D103, still being approximately 10 kPa below the reference pressure 4 days later. Piezometer D9 (10 m depth) showed a temporary pressure reduction of approximately 2 kPa during drilling of Anchor D103 before it increased evenly to a maximum excess pressure of approximately 8 kPa in the following 4 days. Some minor temporary increase in pressure between 2 and 4 kPa was also observed in PZ E10 and E11 during drilling.

The pore pressure reductions observed in both Area D and E were likely caused by some minutes of air flushing during drilling in Area D when trying to improve the transport of drill cuttings and penetrate through a layer of dense sand/moraine encountered at a soil depth of approximately 18 m. Flushing with air probably caused a so-called air-lift pump effect (Behringer 1930; Kato et al. 1975) in front of the drill bit when the water and drill cuttings inside the casing was flushed up to the surface by pressurized air. This caused a lower pressure inside the casing compared to the pore pressure in the surrounding sand/moraine, creating a gradient, i.e., flow of groundwater, toward the drill bit like a pumping well. Owing to the higher permeability (hydraulic conductivity) in the sand/moraine layer compared to the clay, the effects of air flushing were noticeable in Area E over 20 m from the anchors. It is reasonable to assume that the recovery time for the pore pressure was increased since the two casings in Area D were not filled manually with water again after the drilling was aborted. The amount of drill cuttings generated from Anchors D104 and D103 indicates that the drilling formed a cavity, i.e., volume loss, around the casings in the moraine layer. The volume loss is likely the main reason for

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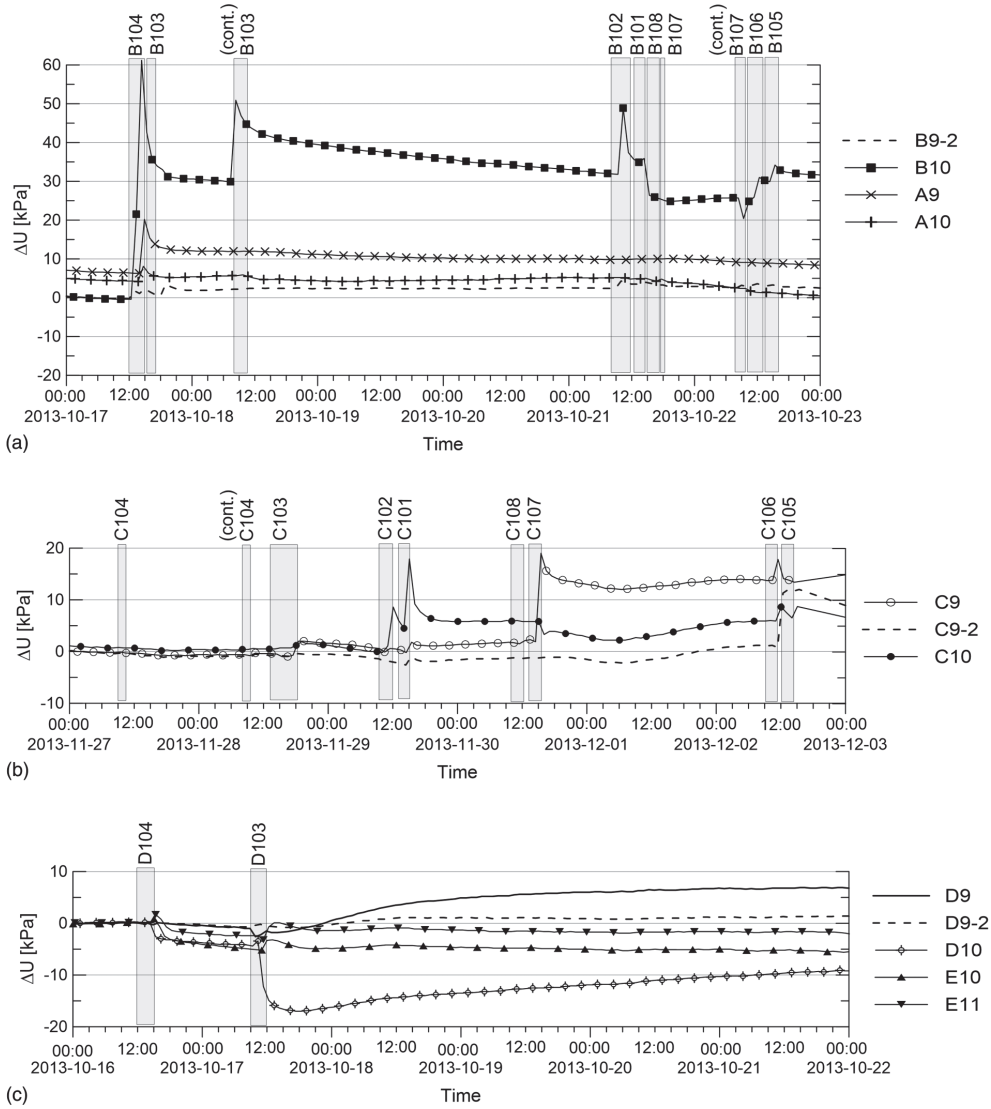

Fig. 6. Change in pore pressure (ΔU) with respect to time during drilling in (a) Area B; (b) Area C; and (c) Area D.

the pressure reduction in PZ D10, caused by suction in the clay above the cavity in the sand/moraine layer.

Fig. 7 shows dissipation of excess pore pressures against time for some piezometers in each test area. Most of the excess pore pressures dissipated rapidly after drilling was completed. This behavior coincides well with results from pile driving in clay reported by, among others, Li et al. (2019), Karlsrud (2012), and Edstam and Küllingsjö (2010). A similar dissipation trend is also observed with pile drilling (e.g., Ahlund and Ögren 2016; Veslegard et al. 2015). In Area E only 10 kPa of the maximum excess pressure of 70 kPa remained 2 days after drilling of Anchor E107, and approximately 30 days later it had completely dissipated. Pulling of the casings in Area E might have made it easier for the excess pressure to dissipate to the grouted borehole, thereby speeding up the process compared to the other test areas. PZ B10 showed a slower trend with approximately 8 kPa excess pressure remaining 150 days after drilling was completed in Area B. The longer dissipation time indicate a more severe influence from drilling on the surrounding clay.

Fig. 8 presents the maximum change in pore pressure ($\Delta U_{\max}$) at different depths in each test area against the distance from the anchors (casings). Fig. 8(b) shows results against the ratio of radial distance from casing r to the radius of casing r_0, and Fig. 8(b) shows results against metric radial distance. The data represent the maximum values obtained from the drilling of single anchors with each method. For comparison, typical excess pore pressure

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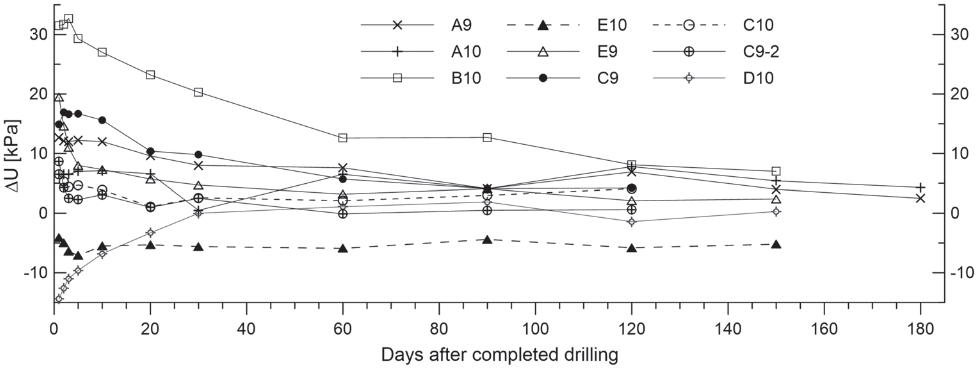

Fig. 7. Dissipation of excess pore pressure (ΔU) over time.

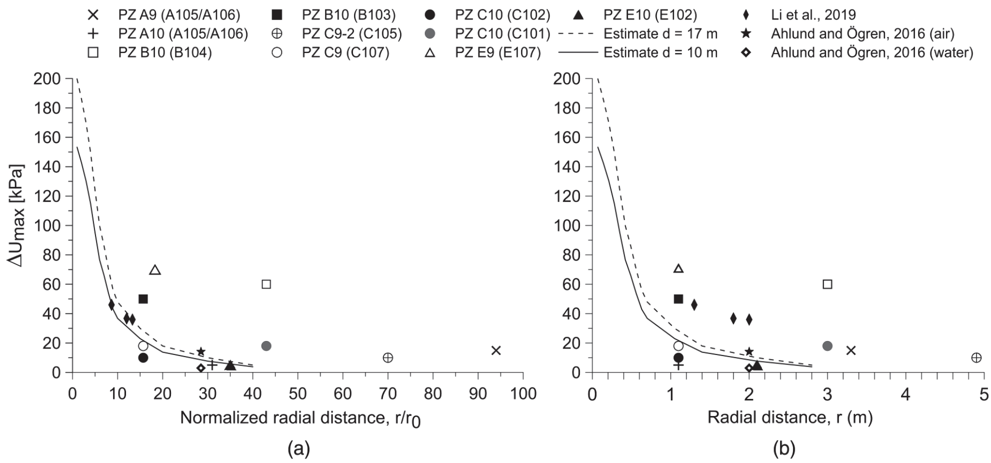

Fig. 8. Maximum change in pore pressure (ΔU_{max}) against (a) normalized radial distance; and (b) metric radial distance from anchors (casings).

curves are also shown for a driven closed-ended pile, based on the strain path method (SPM) theory (Baligh 1985) and the procedure described by Karlsrud (2012). The estimated excess pore pressure is for a single pile with a diameter equal to the casings in Areas B and C (OD = 140 mm), at depths of 10 and 17 m below ground level. Driven closed-ended piles are here considered to represent a worst-case scenario in terms of generating soil displacements and excess pore pressures in the surrounding ground. Drilling should ideally represent the opposite, i.e., a method by which the soil volume of the casing being installed is removed, thereby limiting soil displacements. Fig. 8 includes some results from a recent field test with the jacking of open-ended concrete piles (OD = 300 mm, wall thickness = 70 mm) in soft organic clay (Li et al. 2019). It also include results reported by Ahlund and Ögren (2016) from a field test comparing air- and water-driven DTH hammers for the drilling of piles (OD = 139.7 mm). The effect of air flushing was much higher than that of water flushing, but both methods are in the lower range compared to the results presented in this paper.

Results from this field trial show that drilling may cause much higher excess pressures than previously reported for driven closed-ended piles. The main reason for the excess pressures is likely related to the high penetration rate used for all drilling methods except in Area C (Table 2). The results indicate that the flushing process enhances the effect on soil displacements.

Ground Settlements

All 40 settlement anchors at the test site were monitored over a period of approximately 9 months. The first measurements (baseline) were made September 9, 2013, 10 days before the field tests started in Area A. Settlements were measured more frequently during the field test (between one and three times a week) so that any immediate effects from drilling could be documented. The frequency was reduced to every 4–5 weeks after drilling was completed in Area C. Accuracy of the measurements was specified as ±1 to 2 mm by the surveying company.

Fig. 9 presents the vertical ground settlements (δv) measured on Settlement Anchor 4 (Fig. 4) in each test area from September 9, 2013, to January 7, 2014. The gray bars show the time when drilling took place in each area. The monitoring data show that drilling

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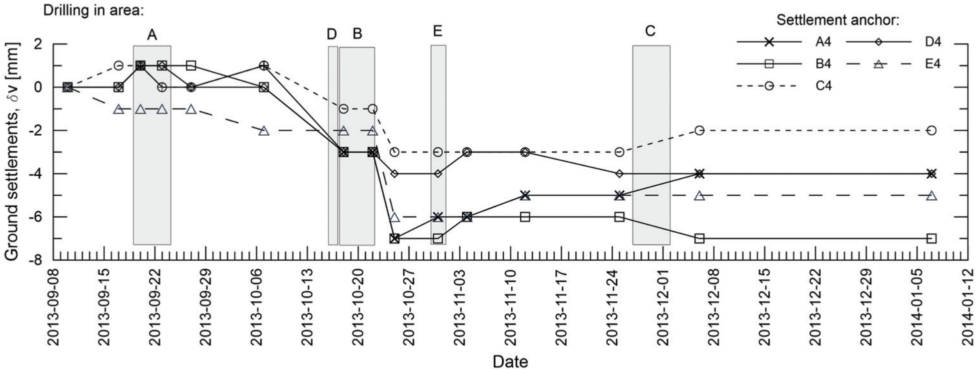

Fig. 9. Vertical ground settlements (δv) measured on Settlement Anchor 4 in Areas A–E, from September 9, 2013 to January 7, 2014.

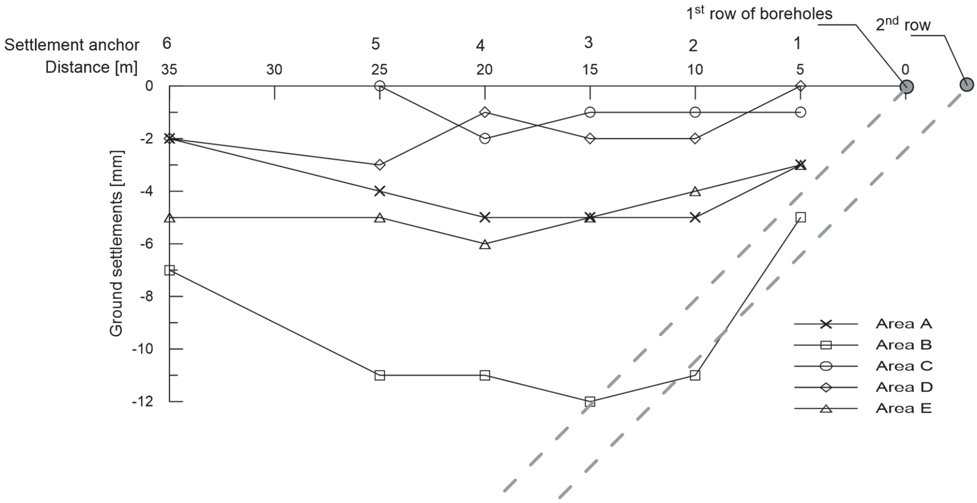

Fig. 10. Ground settlement profiles for Anchors 1–6, related to distance from first row of boreholes at ground surface. Data from final measurement on June 6, 2014.

in Area B (Method 2) and, likely, in Area D (Method 4) caused almost immediate settlements between 2 and 7 mm of all settlement anchors over the entire test site. Results of the pore pressure measurements and observations during drilling clearly indicate that these settlements were caused by air flushing used with Methods 2 and 4. The settlements in Areas B and D can be explained by local erosion and volume loss around the casings, but such volume loss can hardly explain the influence on the other test areas. None of the other drilling methods caused similar short-term settlements or effects in other areas.

After drilling in Area B was completed, subsequent measurements until June 2014 showed 2- to 6-mm settlements over a period of 3 months (between January 7 and April 4, 2014). During this period some of the remaining excess pore pressure (approximately 5–10 kPa) in PZ B10 dissipated, indicating reconsolidation of remolded clay. There were no significant further settlements in Area A, C, D, or E during this period, but there were indications of heave (1–3 mm) in some points in Areas C, D, and E. Freezing of the topsoil during the winter may have led to undesired uplift on the settlement anchors. Mitigating measures in terms of insulating the outer pipe (casing) above the ground surface and filling frost inhibiting liquid between the outer pipe and the settlement anchors were carried out. Despite these efforts, the anchors seem to have experienced some frost-induced uplift.

Fig. 10 shows the resulting ground settlement profiles for Settlement Anchors 1–6 in each test area. The settlements in Area B clearly stand out compared to the other areas with a maximum value of 12 mm (Anchor B3). The results clearly show larger settlements in the area above where the anchors hit bedrock, i.e., Settlement Anchors 2–5.

Discussion

The previously presented monitoring data clearly show that Drilling Methods 2 (Area B) and 5 (Area E) resulted in significantly larger excess pore pressures in the surrounding clay than any of the other methods. The main reason for this difference is likely due to the higher penetration rate with Methods 2 and 5 (12 m/min) compared to Method 3 (1–2 m/min). Measurements in Areas B and E showed that the amount of drill cuttings from single anchors was 50%–80% less than the volume of casings installed, thereby indicating significant soil displacement. With Methods 1 (Area A) and 3 (Area C) the drill cuttings tended to be larger than the volume of the drill rod/casing, indicating a net volume loss or so-called overcoring, which may explain the smaller excess pore pressures measured in these areas (Fig. 8).

The effects of soil displacement when installing a casing through clay may be comparable to the installation of displacement piles in clay if the displacement ratio is similar. The effects of pile installation on displacements has been reported by, among others, Randolph and Wroth (1979), Baligh (1985), Lehane and Jardine (1994), Edstam and Küllingsjö (2010), and Karlsrud (2012). However, the results presented in Fig. 8 show that the excess pore pressure and influence zone were much larger than expected based on past experience with driven closed-ended piles. This indicates that flushing with pressurized water [500–2,000 kPa (5–20 bar)] may have increased the soil displacement.

The soil displacements in Areas B and E should in theory have caused some minor ground heave, but this was not observed in the settlement measurements since they only captured the accumulated total effects after drilling of one or more anchors. It is therefore reasonable to assume that the high penetration rate and soil displacements may have reduced the ground settlements.

The immediate ground settlements occurring over the entire test site right after drilling with Methods 2 (Area B) and 4 (Area D) are likely explained by a combination of two main effects: local erosion and loss of soil volume and temporary pore pressure reduction above the bedrock:

- The uncontrolled outbursts of air, water, and remolded clay observed on the outside of Anchor B104 (Area B) plus A104 and A103 (Area A) may have contributed to some local erosion of moraine and clay, causing cavities around those specific anchors. However, this effect probably made a minor contribution to the ground settlement and cannot explain the large affected area.
- A significant amount of groundwater was flushed up through several of the casings in Area B when the bedrock was drilled into using air flushing. It was not possible to measure the volume of water, but the discharge was estimated to be between 20 and 30 L/min. It is likely that the air flushing used in Areas B and D caused a temporary drop in the in situ pore pressure within the thin permeable layer (sand/moraine) overlying the bedrock, which probably also caused some settlement. This air-flushing effect could have affected a larger area than when local volume is lost just around casings. The pore pressure measurements at PZ E10 and PZ E11 installed just above the bedrock in Area E provided evidence of such an air-flushing effect.

Both monitoring data and observations substantiate the hypothesis that drilling by air flushing may cause air-lift pumping. Flushing with air will reduce the density of the soil-air-water mixture inside the casing, thereby creating lower pressure inside the casing than in the surrounding ground. The difference in pressure induces a flow of groundwater toward the drill bit that also may cause a substantial amount of fine-grained soils, such as silt and sand, to be transported ("sucked") into the borehole. This effect was also observed and reported by Ahlund and Ögren (2016), and it could explain the ground settlements reported in case records by Konstantakos et.al (2004), Kullingsjø (2007), and Bredenberg et al. (2014).

The ground settlements in Area B continued to increase between 2 and 6 mm over a period of approximately 5 months after drilling was completed. This indicates that approximately 40%–50% of the resulting settlements is associated with the dissipation of excess pore pressures and reconsolidation (i.e., change in the void ratio) of possibly remolded/disturbed clay around the casings. The other four methods had similar, but smaller, impacts on pore pressures and settlements, and the settlements stopped shortly after drilling in these areas. Apart from some variations in the depth to bedrock, the marine clay deposit across the site is considered homogeneous. Variability in soil conditions is therefore not likely to explain the difference in settlements observed for the different drilling methods.

The total volume loss (ΔV_{1+2}) caused by reconsolidation of disturbed clay around the anchors has been calculated and is compared with the measured ground settlements (Table 4). The method is specific for tieback anchor installation, but some of the inputs are based on results from field tests with driven closed-ended piles in clay (Karlsrud and Haugen 1984) and model testing (Ni et al. 2009), which provide information on displacements and volumetric strains in the clay surrounding a closed-ended pile after complete reconsolidation. On the basis of these experiments, a potential volume reduction due to the reconsolidation of disturbed or highly strained clay around the drill string is estimated as follows: a volume reduction, $\varepsilon_{v,1}$, of 15% within a 20-mm-thick layer of assumed completely remolded clay and a 10% volume reduction ($\varepsilon_{v,2}$) within partly remolded clay assumed to extend to a radial distance of twice the radius of the casing.

Table 4 presents the different parameters used to calculate the estimated settlements from reconsolidation. Fig. 11 illustrates the total area assumed to be affected by the reconsolidation of clay around the anchors in each specific test area. The ground surface area A_{cons} is assumed to be limited horizontally by an inclination of 2:1 from the depth of the bedrock to the ground surface. The ground surface area A_{cons} is multiplied by the measured mean ground settlements from reconsolidation $\delta_{V,\mathrm{cons}}$ to find the total volume loss

Table 4. Estimated total volume loss due to reconsolidation of disturbed clay around anchors on test site

Method/area	$\sum L$ (m)	r_1 (cm)	r_2 (cm)	V_1 (m^3)	ΔV_1 (m^3)	V_2 (m^3)	ΔV_2 (m^3)	$\sum \Delta V_{1+2}$ (m^3)	A_{cons} (m^2)	$\delta_{V,\mathrm{cons}}$ (mm)	ΔV_{cons} (m^3)
1/A	176	5.5	7.0	0.995	0.149	1.037	0.104	0.253	315	0–1	0–0.315
2/B	266	9.5	15.0	2.846	0.427	11.277	1.128	1.555	680	2–3	1.36–2.04
3/C	276	9.5	15.0	2.948	0.442	11.684	1.168	1.611	710	0	0.0
5/E	223	8.0	12.0	1.963	0.295	5.610	0.561	0.855	480	0–1	0–0.48

Note: $\sum$ L = total length of all eight anchors in each area; $r_1 = r_0 + 2$ cm = radius of completely remolded clay; $r_2 = 2 \times r_0$ = radius of partly remolded clay; V_1 = volume of completely remolded clay; V_2 = volume of partly remolded clay; ΔV_1 = volume loss in completely remolded clay; ΔV_2 = volume loss in partly remolded clay; $\varepsilon_{v,1} = 15\%$ = volume reduction of completely remolded clay due to reconsolidation; $\varepsilon_{v,2} = 10\%$ = volume reduction of partly remolded clay due to reconsolidation; A_{cons} = area at ground level assumed to be influenced by reconsolidation; $\delta_{V,\mathrm{cons}}$ = mean ground settlements measured; and ΔV_{cons} = total volume loss based on measured ground settlements.

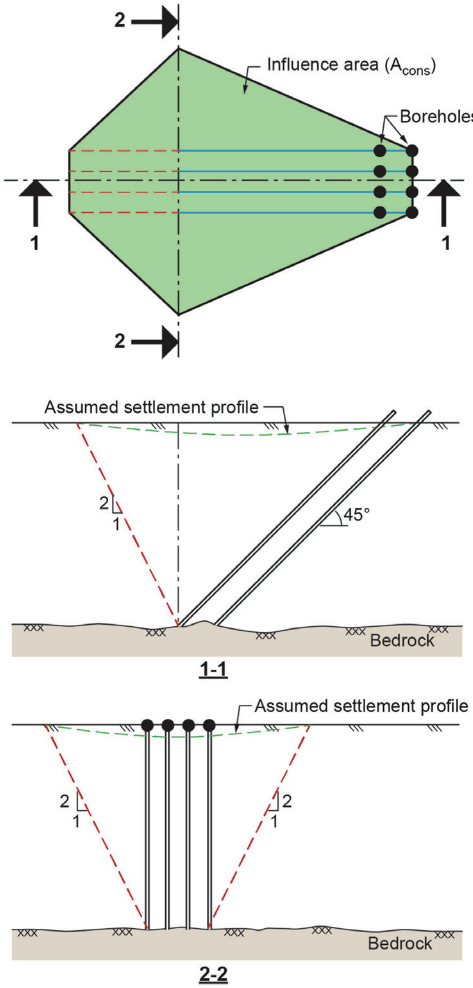

Fig. 11. Assumed influence area from reconsolidation of remolded clay around anchors.

for all anchors ΔV_{cons}. The measured volume loss can then be compared with the estimated volume loss ΔV_{1+2}. In Test Area B, a total area of approximately 680 m^2 experienced a mean ground settlement of 2–3 mm due to reconsolidation. This equals a total volume loss for all anchors, ΔV_{cons} between 1.36 and 2.04 m^3, which coincides reasonably well with the calculated volume loss, $\Delta V_{1+2} = 1.55$ m^3. The estimates for Drilling Methods 1 and 5 also show rather good agreement, but the results are only used as an indication of the potential volume loss due to reconsolidation. The measurements in Test Areas A, C, and E showed no clear settlement trends from reconsolidation.

Conclusions

This paper describes and present results from a full-scale field test program with drilling for tieback anchors through soft clay and into bedrock. Monitoring data and observations during the tests yielded new valuable information about the main installation effects due to overburden drilling, with a focus on the effect on pore water pressures and ground settlements.

Drilling through soft clay is often performed at a high penetration rate, which causes significant soil displacement and excess pore pressure. Results from Test Areas B and E shows that the excess pore pressure in the surrounding clay can grow to become much greater than expected for driven, closed-ended piles with the same diameter. However, most of the excess pore pressure seems to dissipate during the first days after drilling, with the remaining excess pressure dissipating in connection with the reconsolidation of remolded clay, which may take several months. The results from Area C (Method 3) gives reason to assume that a drilling penetration rate of approximately 1 m/min reduces unwanted soil displacements and excess pore pressures in the surrounding clay. This is assumed to be valid for flushing with water and may be different if air flushing is used in soft soils. The soil displacements caused by high penetration could, to some extent, compensate for other effects that may cause ground settlements, i.e., overcoring and loss of soil volume.

Drilling with air flushing may cause uncontrolled outbursts of compressed air along the casing and into the surrounding ground, as observed in Area B with the DTH air hammer. Such outbursts can lead to overcoring and cavities around the casings. Air flushing can also cause temporary reductions in pore pressure due to the air-lift pump effect. Results from piezometers installed down to the moraine layer in Area E (PZ E10 and PZ E11) and just above it in Area D (PZ D10) clearly indicated that air flushing with Method 4 (Area D) caused a temporary drop in pore pressure within a thin permeable layer (sand/moraine) above the bedrock. The same effect likely occurred in Area B, but was not detected by the piezometers since they were installed approximately 5 m above the moraine layer. Reduced pore pressures are most likely the main reason for the immediate settlements that were measured in the entire test field area. Air flushing may also cause erosion and cavities around the casing due to the air-lift pump effect. Drilling with only water flushing will not cause such large so-called pumping effects, and so this method of drilling will reduce the risk of large ground settlements.

The relatively small ground settlements generated in the field tests stand in strong contrast to the large settlements (up to 40 cm) reported by Langford et al. (2015) around excavations in soft clays supported by tieback anchored sheet pile walls. This difference can likely be explained by (1) the relatively few anchors/casings with small diameter installed at the test field, meaning a limited affected soil volume; (2) that drilling at the test field was carried out from the ground level, which implies a reduced unbalanced earth pressure at the top of the casing during drilling compared to drilling from a lower level within the excavation; and, more importantly, (3) the fact that drilling from the ground level excludes effects of drainage up along the casing, which is commonly observed when drilling from below the water table within an excavation. Drilling from below the water table commonly reduces the pore pressure to the level of the top of the casing and starts a consolidation process from the bedrock and up through the clay deposit, which can cause large settlements (e.g., Langford et al. 2015; Langford and Baardvik 2016).

Overburden drilling of casings involves a combination of rotation, penetration, and flushing with air or water, thus making it a very complex process. The natural variations in ground conditions, quality of workmanship, drilling systems, and procedures used make it difficult to foresee the effects of installation on the surrounding ground. Given the limited research on this topic, the authors recommend that more field data be gathered and analyzed in connection with drilling in different ground conditions. It is particularly important to investigate further the effects of drilling with air versus water flushing and to test existing and new drilling methods from the bottom of excavations in the type of controlled manner that was applied in this study from the ground surface. In time, this may lead to the development of methods that limit the undesirably large ground movements observed as a consequence of drilling for tie-back anchors or piles into bedrock from within deep excavations involving soft clays.

Data Availability Statement

Some or all data, models, or code generated or used during this study are available from the corresponding author by request.

Acknowledgments

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Notation

The following symbols are used in this paper:

c_u = undrained shear strength;
G_{50} = undrained secant shear modulus at 50% of shear strength mobilization;
I_p = plasticity index;
OCR = overconsolidation ratio = p'_c/σ'_{v0};
p'_c = apparent preconsolidation pressure (defined from oedometer test);
r_1 = radius of completely remolded clay around casing;
r_2 = radius of partly remolded clay;
S_t = clay sensitivity;
V_1 = volume of completely remolded clay;
V_2 = volume of partly remolded clay;
w = water content;
δv = vertical ground settlements;
ΔU = excess pore pressure;
ε_v = volume reduction due to reconsolidation of clay;
γ = soil density;
σ'_{v0} = in situ vertical effective stress;
ΔV_1 = volume loss in completely remolded clay;
ΔV_2 = volume loss in partly remolded clay;
$\varepsilon_{vol,1}$ = volume reduction of completely remolded clay due to reconsolidation; and
$\varepsilon_{vol,2}$ = volume reduction of partly remolded clay due to reconsolidation.

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Paper III

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Paper III

Small-scale modelling of pile drilling in sand – investigation of the influence on surrounding ground

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ABSTRACT: Overburden drilling of casings for tieback anchors and micro piles can cause considerable excess short-term ground displacements in the surroundings. The mechanisms of drilling effects on soil displacements are not fully understood. In this context, a physical modelling approach was chosen to study the installation effects of overburden drilling. This paper presents a small-scale model test of overburden drilling in a medium dense saturated sand. A detailed description of the experimental setup and test procedure is described. Specifically, the modelling of the drilling procedure including penetration, rotation and flushing is discussed. Tests with water flushing show that there is a clear relation between the flow rate and penetration rate, and the ability to achieve fluidization of the soil in front of the drill bit. Increasing flow rates caused larger excess pore pressures and increased the influence area in the surrounding soil. Under this specific model conditions increased flow rate also generated more excess drill cuttings compared to the installed pile volume.

1 INTRODUCTION

Sheet pile walls and tieback anchors that are drilled into bedrock are often used to support deep excavations in soft soils with limited depth to bedrock. Moreover, micro piles and large diameter steel tube piles are frequently drilled from the final excavation level, and subsequently used as pile foundations for future structures. This type of drilling, which is characterised by permanent casings that are drilled through soil and into bedrock, is often called overburden drilling.

The influence on surrounding ground and neighboring structures from deep excavations has been the subject of investigations for many decades. Results from Peck (1969), Mana & Clough (1981) and Karlsrud & Andresen (2008) show that ground settlements in areas surrounding a deep excavation in soft clay can range from about 0.5% to 2% of the final excavation depth. Langford et al. (2015) analyzed several deep excavations in Norway. The results showed that ground settlements caused by initial and secondary effects from installation of drilled tieback anchors and bored piles from inside deep excavations can be significantly larger than 2% of the excavation depth. The main causes for ground settlements were explained by three main factors:

- Horizontal displacements of support wall.
- Erosion and loss of soil volume around casings for anchors.
- Consolidation of clay due to reduced pore pressures above bedrock.

Recent case studies where overburden drilling has led to excessive ground settlements have been reported by Konstantakos et al. (2004), Küllingsjö (2007), Bredenberg et al. (2014) & Sandene et al. (2020).

Sandene et al. (2021) presents results from a 9 m deep excavation in soft clay in Oslo, Norway. Excessive ground settlements occurred behind the sheet pile wall during and right after the tieback anchors were drilled into bedrock. Based on observations during drilling it is likely that the use of air driven down-the-hole (DTH) hammer have caused significant erosion and cavities around the anchor casings in a layer of silty sand above bedrock.

Field data however, is inherently affected by various uncertainties. For this reason, a conclusive interpretation of the obtained results is often difficult, and the mechanisms of drilling effects on soil displacements and pore pressures are not fully understood. While extensive research on driven piles and auger cast-in-place piles exists, there has been limited research on installation effects of overburden drilling, e.g. Lande et al. (2020), Asplind (2017) and Ahlund

& Ögren (2016). In this context, a physical modelling approach was chosen to study the installation effects of overburden drilling.

This paper presents an experimental 1g model test that aims to replicate pile drilling with a continuous casing i.e. overburden drilling, in saturated sand. A novel test set-up made it possible to drill a small-scale pile with simultaneously penetration, rotation and flushing with water or air through the pile to transport the drill cuttings. This contribution investigates the influence of varying water-flushing parameters on the surrounding ground.

2 EXPERIMENTAL SET UP

2.1 *Model tank and pile*

Figure 1 presents a 3D drawing of the experimental set up. Model tests were carried out in a cube shaped steel tank (box) with a length and a width of 900 mm and a height of 840 mm. A linear actuator with a stroke length of 300 mm and a push capacity of 8000 N was connected to a reaction frame on the top of the model tank. Penetration force was measured by a load cell with a capacity of 5000 N connected between the actuator and a motor used to rotate the pile. The rotation motor had a swivel unit that made it possible to flush with water (or air) through the pile drill string and drill bit at the same time as it rotated and penetrated. Both the penetration rate and rotation speed of the pile (rpm) was controlled by adjusting the voltage on the respective power supplies. Pile penetration was measured with an extensometer connected to the frame and rotation motor.

Figure 2 shows details of the model pile and drill bit. The pile had a length of 890 mm and consisted of a casing, i.e. steel tube with an outer diameter of 35 mm and 2 mm thickness. The pile diameter was adapted to the model tank to limit potential boundary effects. The mechanical design was based on a prototype concentric drill system, resulting in a scale ratio of about 1:3.2 between the diameter of the model pile (35 mm) and the prototype (114 mm). The dimensions (i.e. cross-sectional area) of the flushing tube and flushing inlet channels in the drill bit as well as the annulus for the backflow were all based on the prototype to obtain representative flushing conditions. The drill bit was connected to the bottom of the casing/pile. At the face/front of the drill bit, four flushing holes with a diameter of 4 mm were machined. The flushing medium (water or air) was applied from the swivel device on the top of the pile and continued through the flushing tube and accessed the pile through the flushing holes. Between the casing and the flushing tube was a middle steel tube that created an annulus against the outer casing. The flushing backflow went up through this annulus and out of the pile through holes in the casing at the pile top. A small catchpot was used to collect the backflow (water and soil) during drilling.

The upper part of the drill bit has a packer system (i.e. rubber membrane) that enabled to "close" the annulus between the casing and middle tube, so that soil particles could be collected at the end of each test. The packer was activated by pumping air into it.

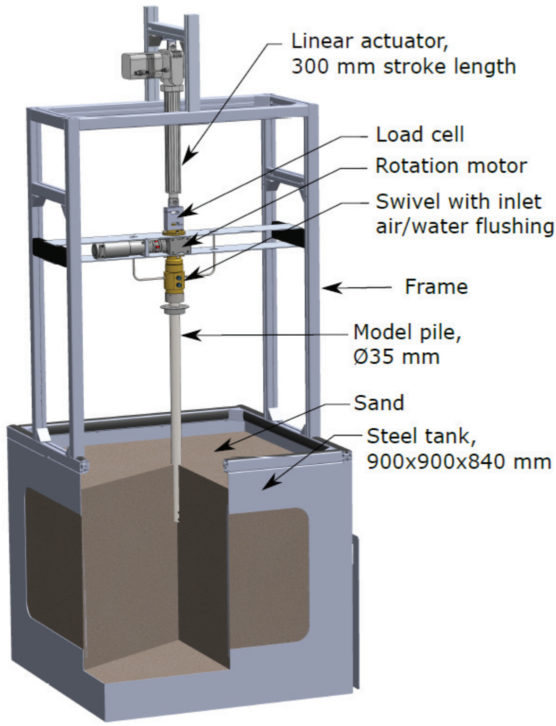


Figure 1. Experimental set up.

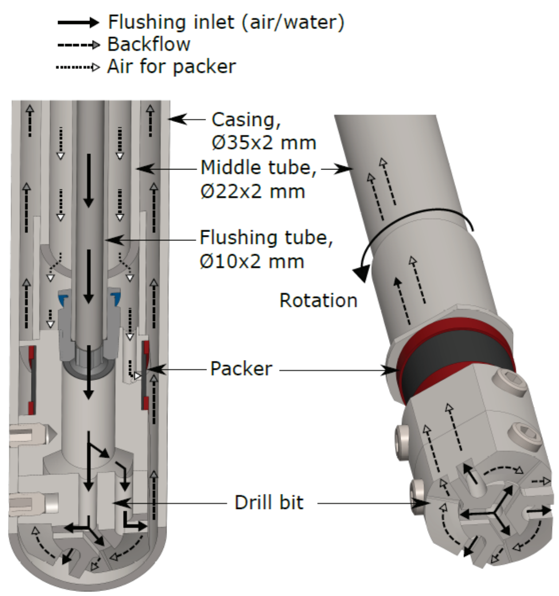


Figure 2. Model pile. Cross-section (left) and drill bit excluding casing (right).

2.2 *Instrumentation*

Figure 3 provides an overview of the model test set-up including used instrumentation. Pore pressure was measured with six sensors (PPs) connected to separate standpipes (i.e. Ø4 mm plastic tubes) installed at two different soil depths (Z_s = 170 and 370 mm) and with three radial distances from the pile center (r = 70, 140 and 210 mm) as can be seen in Figure 3b. The standpipes were supported and kept at the correct position by fastening them to three vertical steel rods (Ø10 mm) connected to a steel plate on the bottom of the model tank.

Vertical displacements of the soil surface (i.e. settlements) were measured with four linear variable differential transformers (LVDTs) positioned at different distances from the pile (Fig. 3c). A gantry (template for pile in Fig. 3c) was used to keep the pile and the LVDTs in position during the tests.

2.3 *Model sand and preparation*

Baskarp sand No. 15 from Sibelco AB was used for all tests. This is a graded fine sand with well-documented properties based on extensive laboratory investigations by among others Ibsen & Bødker (1994) and Ibsen et al. (2009). Table 1 show the main properties of the sand.

Table 1. Properties of Baskarp Sand No. 15 (after Ibsen & Bødker, 1994).

Property	Unit	Value
D_{50} grain size	mm	0.14
D_{60}/D_{10}	-	1.78
Specific gravity, G_s	kN/m^3	2.64
Maximum void ratio, e_{max}	-	0.858
Minimum void ratio, e_{min}	-	0.549

Perforated plastic tubes were placed at the bottom of the model tank in a permeable filter layer of about 70 mm Leca marbles (Figs. 3b and 3c). The filter layer and sand were separated with a geotextile. After filling the tank with dry sand water was pumped through the perforated tubes to saturate the sand. The water level was kept constant about 30 mm above the soil surface throughout the tests by using a weir at the top of the tank.

An effective soil model preparation technique based on a procedure described by Foglia & Ibsen (2014) was adopted. This procedure did not require to remove and refill the sand between each test. The soil model preparation involved the following steps:

I. Loosening of sand (15-20 min) by applying upward water flow from the bottom of the tank through the perforated tube.
II. Pre-installation of pile to a soil depth, Z_s = 200 mm.
III. Vibro-compaction of sand.
IV. Uniformity testing of sand model with miniature cone.

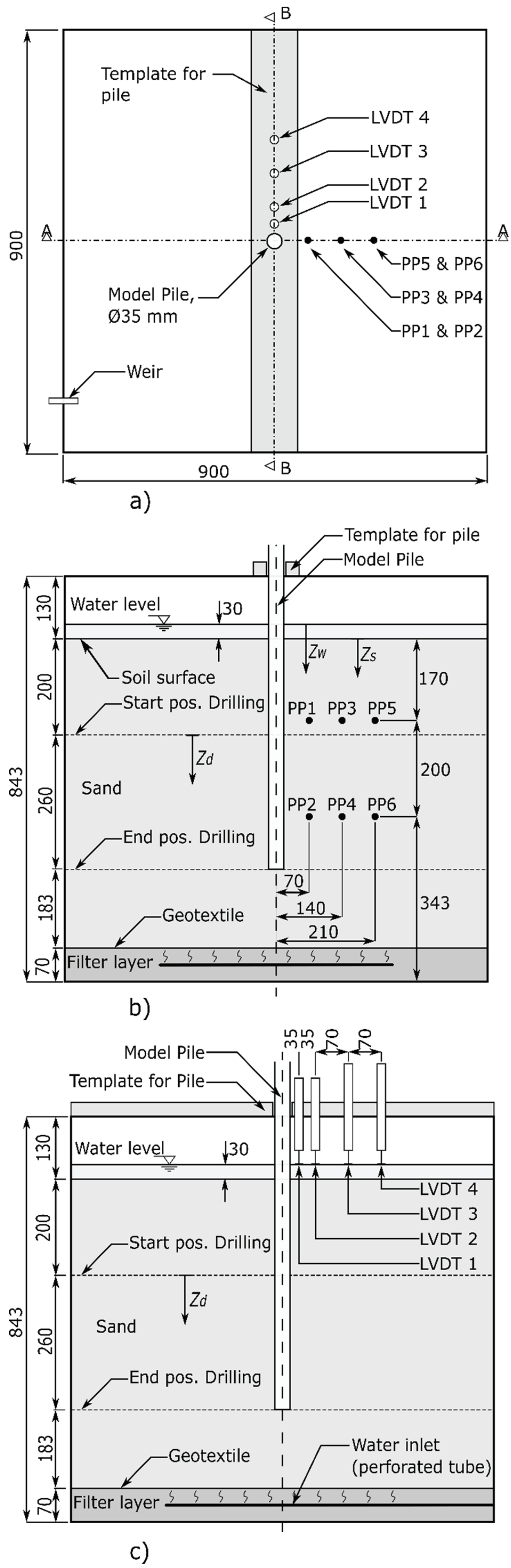


Figure 3. Model test setup: a) plan view, b) cross section A-A and c) cross-section B-B. Dimensions in mm.

Pre-installation of the pile (step II) was carried out with the packer closed and with a small water flow through the inner flushing tube. The packer was kept closed during soil compaction to avoid sand particles filling the annulus between the casing and the middle tube.

In step III the sand model was compacted by pushing a concrete vibrator vertically down to the geotextile layer before pulling it slowly up again. Figure 4 show the compaction grid/pattern with a center distance of 150 mm between each point. Compaction was carried out in two stages, starting with the "A" points followed by the "B" points.

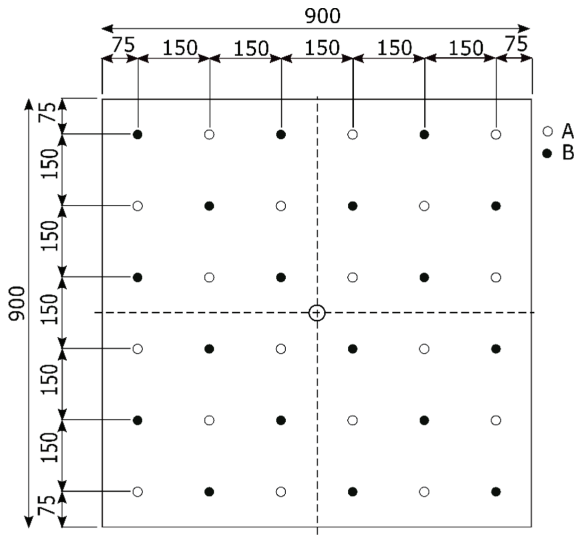


Figure 4. Grid for compaction of sand with concrete vibrator. Dimensions in mm.

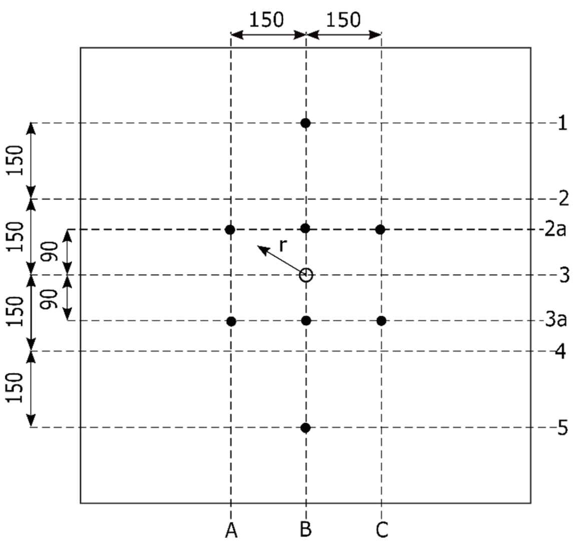


Figure 5. Layout for cone resistance testing of sand model. Black dots represent positions for testing and *r* the radial distance from the pile. Dimensions in mm.

Figure 5 show a layout of the cone resistance testing positions. The cone had a diameter of 10 mm and apex angle of 60°. Penetration rate was about 5 mm/sec. Points B1, A2a, B2a and C2a where generally tested before each pile-drilling test to verify a consistent preparation procedure and that the sand model was homogenous. After pile drilling, cone resistance tests were often also carried out in the points B3a, B4 and B5 to investigate the influence from drilling. All tests presented in this paper were carried out with a mean relative soil density (D_r) from 0.6 to 0.65. The density was calculated for each test based on the total dry mass of sand and the volume of sand in the tank.

2.4 *Test program*

A series of pile drilling tests were performed at the Norwegian Geotechnical Institute (NGI) in Oslo, Norway. An overview of the test program for this study is given in Table 2. Test A was carried out as a reference test by pushing the pile into the sand without any rotation and flushing (i.e. displacement pile). The flushing flow rate (Q) varied for test B to E, while both penetration rate and pile rotation was kept constant for all tests.

The program included testing with air flushing as well. Despite several attempts with different starting depths and flushing pressure it was not possible to get transport of drill cuttings up through the pile. Due to low soil stresses and relative high air pressures, air evacuated up along the outside of the pile, a behavior which is also frequently observed when drilling close to the soil surface in the field (Lande et al. 2020; Sandene et al. 2020). A starting depth of 40 cm and an air pressure of about 100 kPa was used in test F.

All tests with water flushing followed the same procedure. First, the flushing started, and 5 seconds later the rotation and penetration was turned on at the same time. After the drilling was stopped, the packer was immediately closed to prevent sand particles flowing out from the pile casing. Subsequently, the cone resistance tests after the drilling were conducted, after which the pile was lifted out of the sand model. Then, the packer was opened to collect the sand particles in the pile casing.

Table 2. Test program.

Test	Flow rate l/min	Pressure kPa	Penetration rate mm/s	Rotation rpm
A	-	-	2.5	0
B	1.5		2.5	20
C	2.0		2.5	20
D	3.0	15	2.5	20
E	5.0	60	2.5	20
F	-	100	1.5	20

3 RESULTS AND DISCUSSION

The following section presents and discusses the obtained test results. Figure 6 show the load cell measurements which provides a qualitative measure of the soil resistance against depth. The drilling was started at a soil depth, Z_s of 200 mm (Figs. 3b and 3c). Test A showed an immediate load increase to about 1400 N followed by an almost linear increase in penetration force with depth, resulting in a maximum value of about 4100 N at a drilling depth, Z_d of about 260 mm. This equals a tip resistance in the same range as the cone resistance tests presented in the following. Small decrease in load was observed at about 160 mm and 220 mm drilling depths respectively. These differences from the linear trend could be explained by local inhomogeneity in the sand model after vibrocompaction.

Results from tests B to E indicate that when the water flushing flow rate (Q) is above a certain threshold the sand in front of the drill bit does not provide any resistance. This is likely explained by local fluidization of the sand in front of the pile tip like model test results of water jetting for piles reported by Tsinker (1988) & Shepley & Bolton (2014). For this reason, the load cell data show no load or negative load values for the tests C, D and E. This can be explained by the weight of the pile and rotation motor pulling on the load cell.

For test B the flow rate (Q = 1.5 l/min) was likely too low to cause local fluidization, thus the soil remained some of its resistance and a maximum load of about 1300 N was observed. As the penetration load increased the friction between the pile and the soil reduced the rotation and the rotation almost stopped for the last 60 mm of the drilling.

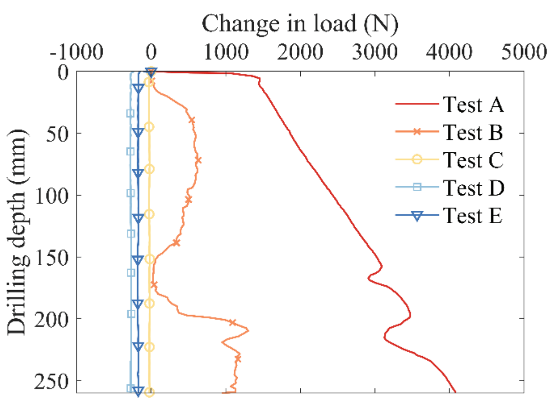


Figure 6. Load cell measurements.

Figure 7 present measured change in pore pressure (Δu) against pile drilling depth (Z_d) for all pore pressure sensors (PP1 to PP6). Test A show a clear decrease in pore pressure as the pile was pushed into the sand. A maximum pressure reduction of about – 2.3 kPa in PP1 (Fig. 7a) occurred rapidly after penetration started, before slowly increasing again during penetration, being about 0.5 kPa below the starting value at end of drilling. Similar trends were also observed in PP2, PP3 and PP5, however with less influence at greater distance from the pile. Only minor pressure reductions of about 0.2 kPa were measured in PP4 and PP6. The pressure reductions with Test A are likely explained by dilation effects in the sand surrounding the pile tip and shaft as it was displaced by the penetrating pile (White & Bolton, 2004).

All water flushing tests (Test B to E) resulted in excess pore pressures in the surrounding sand, however moderate compared to the applied input drilling fluid pressure up to 60 kPa in Test E. As expected the influence generally increased with higher flow rates. Test B (Q = 1.5 l/min) caused maximum pressure changes of about 0.5 kPa in PP1 and PP2 at 70 mm distance from the pile center. Test E (Q = 5 l/min) however caused pressure changes of about 1.7 and 2.8 kPa in PP1 and PP2 respectively. The influence gradually decreases at greater radial distance from the pile. PP5 and PP6 at 210 mm distance from the pile center generally showed only minor changes during the tests.

Measurements of vertical soil surface displacements (δ_v) against drilling depth are presented in Figure 8. Test A resulted in soil heave on all four LVDTs, as expected since the pile was pushed in. Test data show a maximum settlement of about 3 mm in LVDT1 (Fig. 8a) and 1.8 mm in LVDT4 (Fig. 8d).

Results from Test B indicate some minor heave (0.1-0.2 mm) in all LVDTs except LVDT1 closest to the pile which settled about 0.3 mm. This behavior is most likely explained by soil displacements since the flushing was not able to fluidize and remove the sand in front of the drill bit. Test C and D had no impact on the LVDTs, while Test E gave about 0.5 mm settlement in LVDT1.

The cone resistance results provide indication of the soil area influenced by the pile drilling. Figure 9 shows cone resistance (q_t) against penetration depth from the water level (Z_w). Tests carried out in position B2a and B3a at about 90 mm distance from the pile (Fig. 9a) show that an increase in flow rate reduced the cone resistance. Test E clearly stands out compared to the other water flushing tests, having an influence at position with about 150-160 mm distance from the pile (Fig. 9b).

An interesting observation is that pushing the pile in without rotation and flushing (Test A) caused the lowest cone resistance in the sand with a significant influence at a distance of about 150-160 mm from the pile. This is explained by the pile penetration causing significant dilation (i.e. volumetric expansion) in the soil. This dilative behavior agrees with results from triaxial tests on Baskarp sand No. 15 showing large dilation angles up to 18 degrees for low stress conditions (Ibsen et al. 2009).

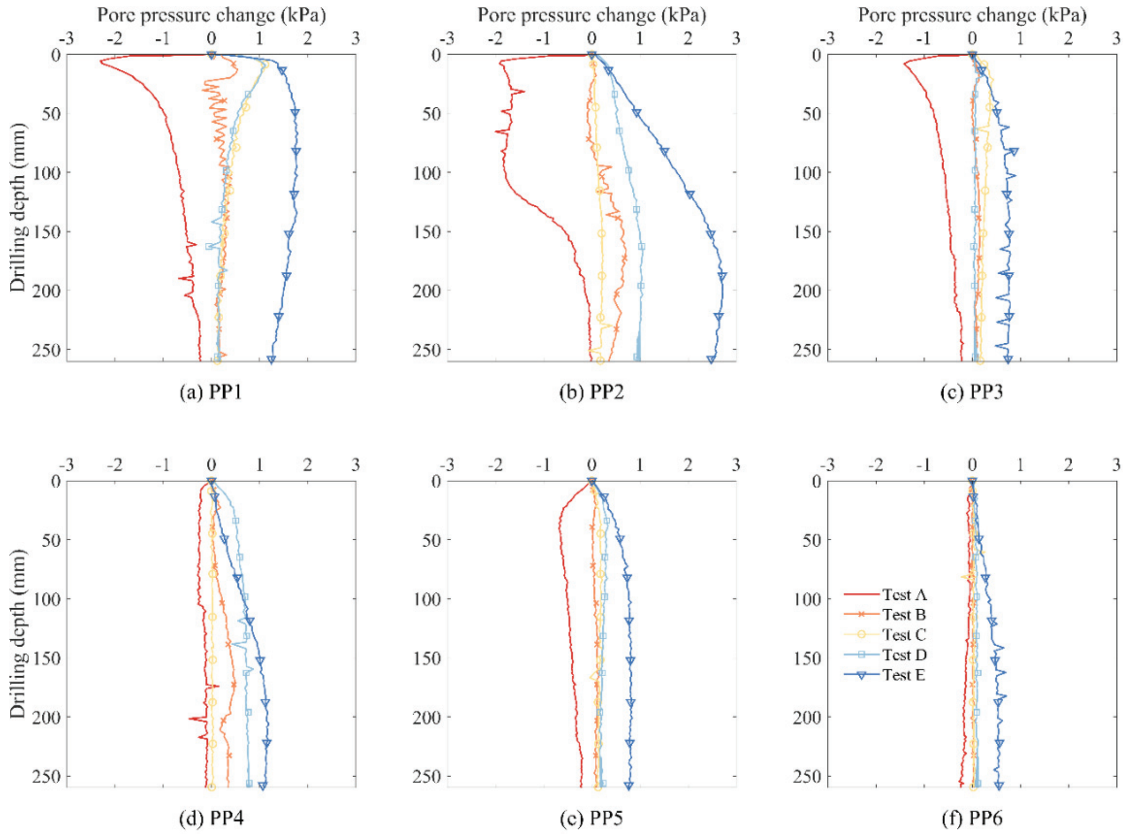


Figure 7. Change in pore pressure against drilling depth in a) PP1, b) PP2, c) PP3, d) PP4, e) PP5 and f) PP6.

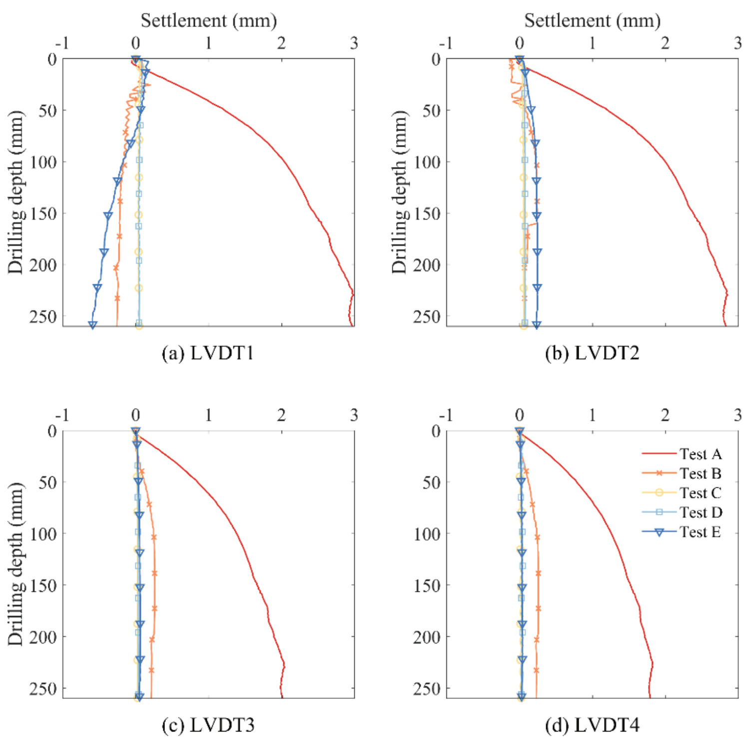
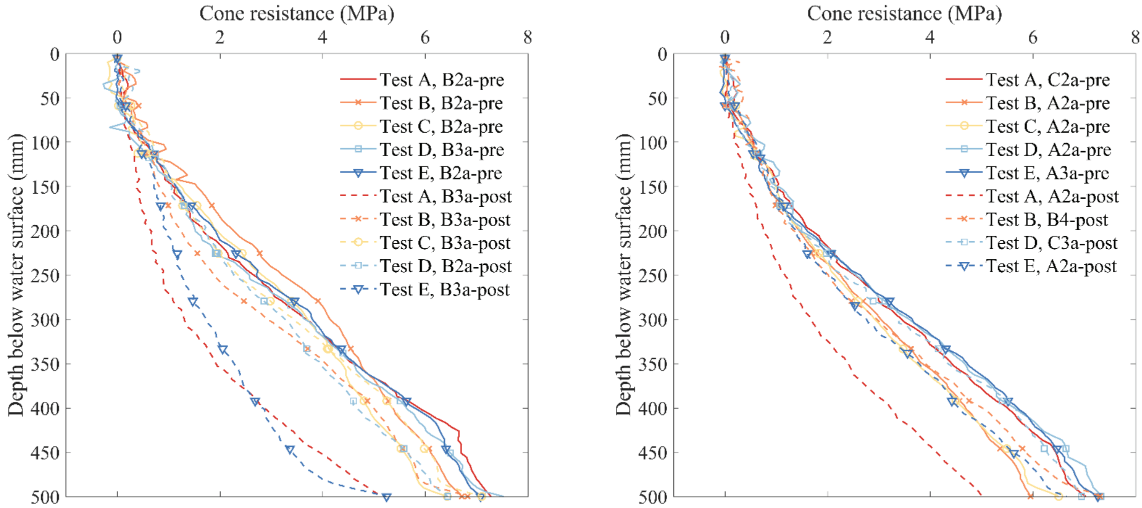


Figure 8. Soil surface settlements against drilling depth for a) LVDT1, b) LVDT2, c) LVDT3 and d) LVDT4.



(a) 90 mm from pile centre (b) 150 - 160 mm from pile centre

Figure 9. Cone resistance against depth for a) CPTs 90 mm from pile center, and b) 150-160 mm from pile center.

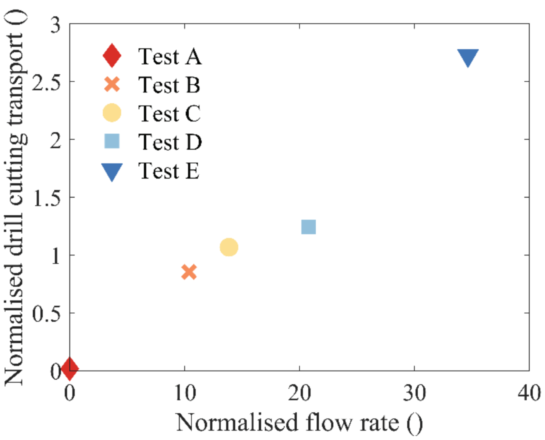


Figure 10. Normalized flow rate against normalized drill cuttings transport.

Figure 10 shows normalized flow rate (Q_{norm}) against normalized mass of drill cuttings ($M_{c,norm}$). Normalized flow rate is defined as:

$$Q_{norm} = \frac{Q}{A_{pile} \times V_{pen}} \quad (1)$$

where Q is the flushing flow rate in dm^3/min, A_{pile} is the cross-sectional area of the pile in dm^2, and V_{pen} is the penetration rate in dm/min. The normalized mass of drill cuttings is defined as the ratio between the mass of drill cuttings (M_c) collected throughout the drilling and the theoretical mass of soil given by the installed pile volume (M_{pile}). This calculation ignores potential drilling induced soil displacements and soil volume changes which is a simplification. A value lower than 1 indicates that the mass of drill cuttings is less than the theoretical one, meaning that the soil is likely replaced by the pile drilling. A value above 1 indicates that the mass of drill cuttings is higher than the theoretical mass, hence causing a potential soil volume loss. A value of 1 is defined as an "ideal" scenario.

With Test A the pile was pushed into the sand without no flushing, thus no drill cuttings were generated. Test B with a flow rate of 1.5 l/min resulted in $M_{c,norm} = 0.85$ indicating that the pile caused some soil displacements like a driven closed-ended pile. This coincides well with the observations of increased penetration resistance in the last 60 mm of drilling (Fig. 6). Test C (Q = 2.0 l/min) seem to be close to an "ideal" flow rate for the given penetration rate and the model conditions with only about 7% excess drill cuttings compared to the installed pile volume ($M_{c,norm}$ = 1.07). The results show a significant increase in drill cuttings transport with higher flow rates. Test E (Q = 5.0 l/min) caused a maximum of about 170% excess drill cuttings ($M_{c,norm}$ = 2.70) indicating a major loss of soil volume (i.e. soil mass) around the pile. This likely explain the settlements observed in LVDT1 (Fig. 8a).

4 CONCLUSIONS

This paper presents results from novel physical modelling of pile drilling in saturated sand. The test setup made it possible to drill a miniature pile with simultaneous penetration, rotation and flushing with water through the pile. Installation effects from drilling on the surrounding ground are identified and discussed based on monitoring data.

The results provide valuable knowledge and information about the physical influence from flushing, and how it may affect the soil resistance, pore water pressure, soil erosion and transport of drill cuttings. Tests with water flushing show that there is a clear relation between the flow rate and penetration rate, and the ability to achieve fluidization of the soil in front of the drill bit. If the flow rate is high enough fluidization reduces the penetration resistance similar to observations during pile jetting (Tsinker, 1988, Shepley & Bolton, 2014). Increasing flow rates caused larger excess pore pressures and increased the influence area in the surrounding soil. Under this specific model conditions increased flow rate also generated more drill cuttings, with a maximum of 2.7 times the theoretical installed pile volume for the highest flow rate of Q = 5 l/min (Test E). Measurements of soil surface settlements did however only show small influences from the drilling, most likely due to the limited soil stress. By plotting the normalized flow rate against normalized drill cuttings transport it seems that a normalized flow rate from 10 to 15 is close to an "ideal" drilling i.e. $M_{c,norm}$ equal or close to 1.0. The effect of more representative soil stresses is, however, an area that requires further research.

Based on the results from this experimental modelling, it is recommended to perform refined model tests under more representative stress conditions and pile drilling parameters. With the introduced framework for the normalized drill cutting transport and flow rate the results could lead to practical recommendations regarding the choice of drilling parameters. The introduced framework should also be investigated further through full-scale tests.

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Paper IV

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Paper IV

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ARTICLE

Physical modelling of pile drilling in sand

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Abstract: Drilling for foundation piles and tieback anchors through soils using a continuous casing to support the borehole is often referred to as "overburden drilling". Monitoring data from several case studies show that overburden drilling may cause considerable short-term ground settlements indicating a loss of soil volume around the casings. However, further insight is required to understand the mechanisms that govern overburden drilling. Novel physical model tests were carried out to investigate the effects of varying parameters such as flushing media (water or air), flow and penetration rate on the penetration force, pore pressure changes, soil displacements, and drill cutting transport. Tests with water flushing indicate a clear relation between the flow and penetration rate and the resulting influence on the surrounding ground. Increasing flow rates caused larger excess pore pressures at greater radial distances and generated more excess drill cuttings compared to the theoretical casing volume. The obtained results were translated into a non-dimensional framework to estimate optimal flushing parameters in similar conditions. The air flushing tests were considerably limited by the modelling constraints. Notable reduction of pore pressures adjacent to the casing indicate an air-lift pump effect that can lead to extensive ground movements as observed in the field.

Key words: model tests, drilling, piles, anchors, settlement, pore pressure.

Résumé : Le forage de pieux de fondation et d'ancrages d'arrimage dans le sol à l'aide d'un tubage continu pour soutenir le trou de forage est souvent appelé « forage dans les morts-terrains ». Les données de surveillance provenant de plusieurs études de cas montrent que le forage de morts-terrains peut provoquer des tassements considérables du sol à court terme, indiquant une perte de volume de sol autour des tubages. Toutefois, il est nécessaire de mieux comprendre les mécanismes qui régissent le forage des morts-terrains. De nouveaux essais sur modèle physique ont été réalisés pour étudier les effets de divers paramètres tels que le milieu de rinçage (eau ou air), le débit et le taux de pénétration sur la force de pénétration, les changements de pression interstitielle, les déplacements de sol et le transport des déblais de forage. Les essais avec rinçage à l'eau indiquent une relation claire entre le débit et le taux de pénétration et l'influence conséquente sur le sol environnant. L'augmentation des débits a entraîné des pressions de pores plus importantes à des distances radiales plus grandes et a généré plus de déblais de forage en excès par rapport au volume théorique du tubage. Les résultats obtenus ont été traduits dans un cadre non dimensionnel pour estimer les paramètres de rinçage optimaux dans des conditions similaires. Les essais de chasse d'air ont été considérablement limités par les contraintes de modélisation. Une réduction notable de la pression des pores adjacents au tubage indique un effet de pompe à air qui peut entraîner des mouvements importants du sol, comme on l'a observé sur le terrain. [Traduit par la Rédaction]

Mots-clés : essais sur maquettes, forage, pieux, ancrages, tassement, pression interstitielle.

1. Introduction

In areas where soft soil deposits of limited depth are overlying competent bedrock so-called overburden drilling is often carried out to install tieback anchors for sheet pile walls (SPWs) as well as end bearing foundation piles into bedrock. Overburden drilling is characterized by permanent casings that are penetrated through the soil (i.e., overburden) using rotary percussive drilling until it reaches bedrock (Sabatini et al. 2005; Finnish Road Authorities 2003). Recent case histories reported by Langford et al. (2015) indicate that overburden drilling for tieback anchors and piles from inside deep excavations in soft clay may cause ground settlements that exceed those reported in previous studies, e.g., Peck (1969), Mana and Clough (1981), Karlsrud and Andresen (2008). While previous research extensively studied the effects from displacements of the retaining walls as well as consolidation effects, few studies have investigated the mechanisms of overburden drilling. Understanding the installation effects and influence from overburden drilling on the surrounding ground is vital to avoid damages on adjacent buildings.

Reported field tests (Lande et al. 2020; Ahlund and Ögren 2016) and case records (Konstantakos et al. 2004; Kullingsjø 2007; Bredenberg et al. 2014; Sandene et al. 2021) indicate that overburden drilling with air driven down-the-hole (DTH) hammers may cause significant excess ground settlements immediately after drilling. These findings are likely explained by a loss of soil

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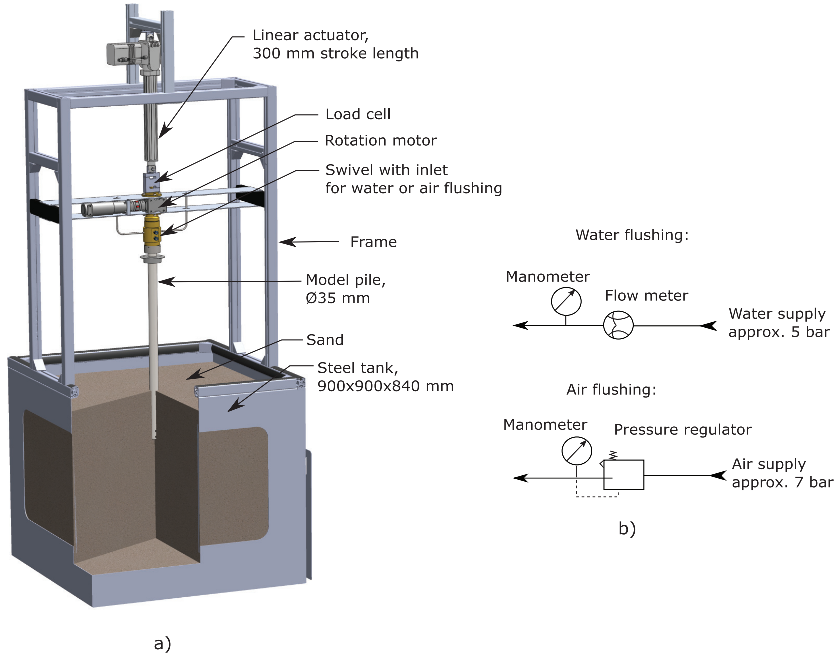
Fig. 1. Test set-up: (*a*) model tank and (*b*) flushing pressure line. (1 bar = 100 kPa.) [Colour online.]



volume around the casings that was often observed when drilling through silty and sandy soils, or granular material (i.e., a moraine layer) above bedrock. The soil loss might be related to the so-called air-lift pump effect (Behringer 1930; Kato et al. 1975) as silt and sand particles are eroded and transported to the ground surface. By contrast, the studies reported by Lande et al. (2020), Asplind (2017), and Ahlund and Ögren (2016) indicate that drilling with a water driven DTH hammer caused less settlements and excess pore pressures compared to air flushing. None of the previous studies included systematic and accurate measurements of drill cutting volume or mass to assess the potential soil volume loss and to verify the hypothesis of the air-lift pump effect. Neither have systematic studies of the effects of drilling parameters on the soil response to overburden drilling including pore pressure changes and soil displacements been carried out.

Another mechanism that was observed with overburden drilling is uncontrolled piping or hydraulic fracturing (i.e., pneumatic blowouts) along the outside of the casing, which was predominantly identified when flushing with compressed air (Lande et al. 2020; Sandene et al. 2021). This behaviour typically occurs during drilling of the first metres below ground surface due to low soil stresses, but it has also been observed when drilling at large depths (e.g., depths >20 m). Such piping effects are comparable to fluidization that has been investigated extensively (Tsinker 1988; van Zyl et al. 2013; Alsaydalani and Clayton 2014; de Brum Passini and Schnaid 2015).

There has been limited research on installation effects of overburden drilling, hence the mechanisms affecting the surrounding ground are not fully understood. In this context, a physical modelling approach was chosen. A series of pile drilling tests in saturated sand was carried out at the Norwegian Geotechnical Institute (NGI) in Oslo, Norway. The objective was to deepen the understanding of the mechanisms due to flushing with water or air and to investigate how the drilling parameters penetration and flushing rate affect the influence on the surrounding soil. The present research aims at providing knowledge that can be used in planning and execution of overburden drilling to reduce the risk of unwanted influence on the surrounding ground. The following section gives a detailed description of the experimental set-up including the model tank and the model pile, instrumentation, drilling simulation, and test procedure. After the results are presented and discussed, conclusions are drawn.

2. Experimental set-up

A novel test set-up made it possible to replicate overburden drilling using a small-scale pile (i.e., casing with an internal drill string and drill bit) with simultaneous penetration, rotation, and water or air flushing to transport the drill cuttings.

2.1. Model tank and instrumentation

Figure 1 illustrates the model test set-up. The soil model was placed in a cube shaped steel tank (Fig. 1*a*). An aluminium reaction frame was fixed to the top of the model tank, acting as support for a linear actuator used to vertically move the pile. The actuator had a maximum stroke length of 300 mm and a push capacity of 8000 N. A load cell with a capacity of 5000 N was connected between the actuator and a rotation motor unit to measure the penetration force on the pile during the tests. The rotation motor had a swivel unit that made it possible to flush with water or air through the pile drill string and drill bit at the same time as the entire pile rotated and penetrated. Both the penetration rate and rotation speed (rpm) of the pile were controlled by adjusting the voltage on the respective power supplies. The pile penetration was measured with an extensometer connected to the frame and rotation motor.

Figure 1*b* shows a schematic illustration of the pressure lines for both water and air flushing. The water supply came directly from the main supply tap with an approximate pressure of

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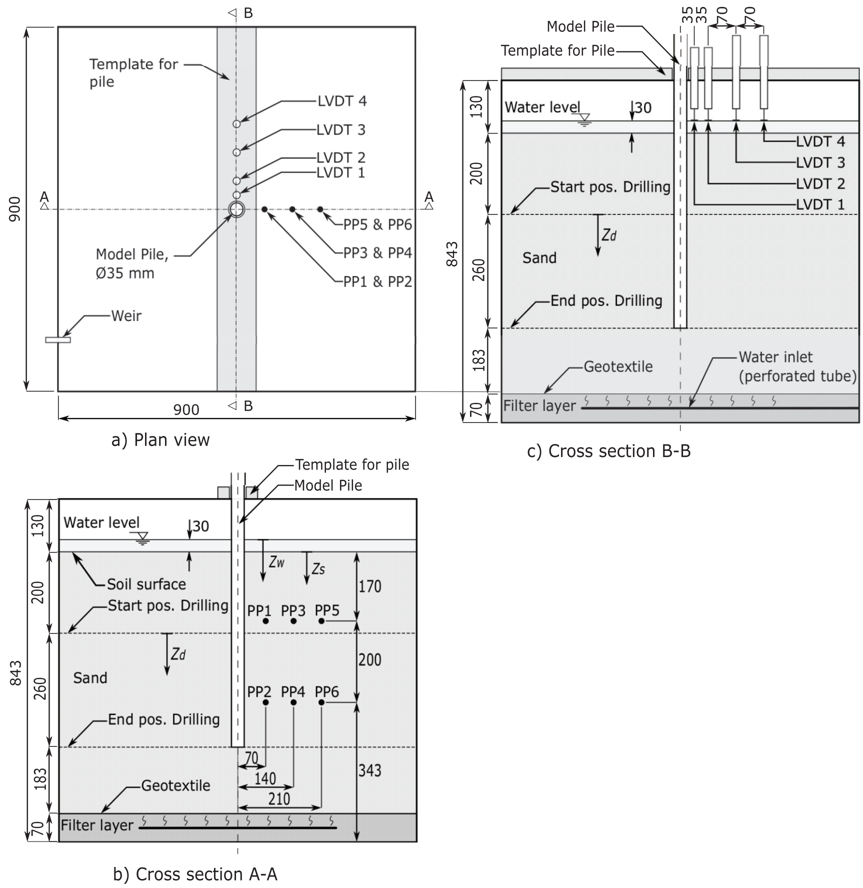
Fig. 2. Experimental setup: (*a*) plan view, (*b*) cross section A-A through pore-water pressure sensors (PPs), and (*c*) cross section B-B through linear variable differential transformers (LVDTs). (All dimensions in millimetres.)



500 kPa. A flow meter was used to control the flow rate while a manometer was used to monitor the water pressure delivered to the pile during a test. For the air flushing tests, a pressure regulator with a manometer was used to control the pressure from the supply that had a pressure of approximately 700 kPa.

The entire tests were carried out with the model pile placed in the centre position of the soil model. Figure 2 presents a layout (Fig. 2*a*) and cross sections (Figs. 2*b* and 2*c*) of the model test setup including the instrumentation used to monitor the soil response. Measurements of pore-water pressures in the sand model were obtained using standpipes, i.e., plastic tubes with a diameter of 4 mm, that were connected to pressure sensors located at the outside of the model tank. Six standpipes were installed at two different soil depths (Z_s = 170 and 370 mm) and with three radial distances from the pile centre (r = 70, 140, and 210 mm) as can be seen in Fig. 2*b*. The standpipes were supported and kept at the correct positions in the sand model throughout the entire test program by fastening them to three vertical steel rods (Ø10 mm) that were connected to a steel plate placed on the bottom of the model tank (see illustration in Section 2.3). A filter was placed at the top of each standpipe to prevent sand grains from entering and affecting the measurements. All sensors were calibrated before the conducted test series, and the values checked before each test to verify that the standpipes were unaffected.

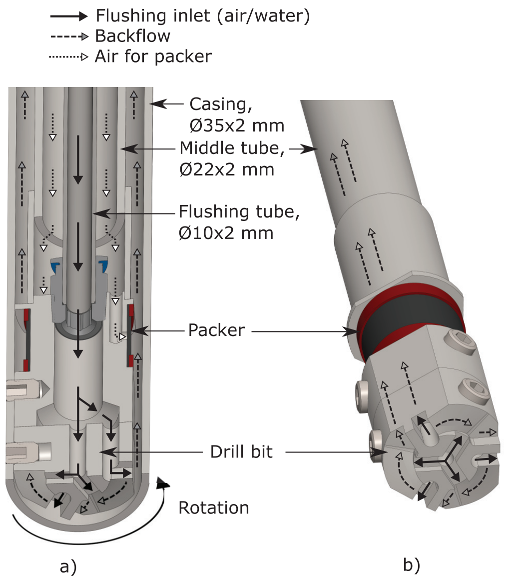
Vertical displacements of the soil surface (i.e., settlements) were measured with four linear variable differential transformers (LVDTs) positioned at different distance from the pile (Figs. 2*a* and 2*c*). A gantry (template for pile in Fig. 2*c*) was used to keep the pile and the LVDTs in position during the tests.

2.2. Model pile and drilling simulation

Figure 3 shows a drawing of the model pile including details of the drill bit. The model pile is 890 mm long and consist of a casing, i.e., a steel tube with an outer diameter of 35 mm and thickness of 2 mm as can be seen in Fig. 3*a*. The flushing medium (water or air) is applied from the swivel device on the top of the pile through an inner steel tube with internal diameter of 6 mm. Between the casing and the inner tube is a middle steel tube that creates an annulus against the outer casing. The flushing backflow is transported through this annulus to the top of the pile, where a small catch-pot is used to collect the backflow (water and soil).

A drill bit is connected to the bottom of the casing–pile with six bolts. Four openings with a diameter of 4 mm were drilled

Fig. 3. Model pile: (*a*) cross section and (*b*) inner parts excluding casing. [Colour online.]



through the drill bit and are used as the flushing inlet during drilling (Fig. 3*b*). The face of the drill bit was designed with cutting grooves at each flushing inlet to direct the flushing media and drill cuttings towards the backflow paths like a typical prototype design. The upper part of the drill bit has a packer system that enables one to close the annulus between the casing and middle tube to collect the soil particles in this annulus at the end of each test. The packer consists of a rubber membrane fixed with tie wrap and can be activated by pumping air into it.

The pile diameter was adapted to the model tank to limit potential boundary effects. The mechanical design was based on a prototype concentric drill system named "Symmetrix" (Epiroc 2020) with a 114 mm diameter casing, resulting in a scale ratio of about 1:3.2 between the diameter of the model pile and the prototype. The dimensions (i.e., cross-sectional area) of the flushing tube and flushing inlet channels in the drill bit as well as the annulus for the backflow were all based on the prototype to obtain representative flushing conditions.

2.3. Sand model preparation

Figure 4 depicts different stages of the initial soil model preparation that was only carried out once for the entire test series. Two perforated plastic tubes were placed at the bottom of the model tank (Fig. 4*a*). One tube was used to pump water into the tank, i.e., applying an upward gradient, and the second tube for draining water from the bottom, i.e., applying a downward gradient. A permeable layer of approximately 70 mm lightweight expanded clay aggregates (LECA) was then placed over the perforated tubes (Fig. 4*b*). A geotextile layer was placed on top of the LECA and taped to the sides of the steel tank (Fig. 4*c*). The model tank was then filled with dry sand up to a thickness of about 640 mm (Fig. 4*d*).

All tests were carried out using Baskarp sand No. 15 (from Sibelco AB), which is a graded fine sand with well-documented properties based on extensive laboratory investigations (e.g., Ibsen and Bødker 1994; Ibsen et al. 2009). Typical index properties of this sand are shown in Table 1.

The soil was saturated using a similar procedure as reported by de Brum Passini and Schnaid (2015). An upward water flow from the perforated tube at the base of the model tank (Figs. 2*b* and 4*a*) with a hydraulic gradient lower than critical was applied. Since the saturation was carried out with water directly from the tap (oxygen rich) and special measures like adding backpressures or using CO_2 gas were not taken, a fully saturated soil model was not achieved. After saturation, the water level was kept constant at approximately 30 mm above the soil surface throughout the tests by using a weir at the top of the tank (Fig. 2*b*).

For each test, a model preparation technique following Foglia and Ibsen (2014) was adopted. This systematic approach enabled reuse of the initial soil model without emptying the model tank. The following procedure was used for each water flushing test:

1. *Sand loosening (approximately 15 min)* — Apply an upward water flow from the bottom of the tank through the perforated pipe using a hydraulic gradient close to critical.
2. *Pile positioning* — Position the pile vertically and horizontally above the soil surface using the pile template (Fig. 2). Fill the annulus between the casing and middle tube with water up to the backflow holes at the top of the pile while the packer remains closed.
3. *Pile pre-installation* — Penetrate the pile until it reaches its start position approximately 200 mm below the soil surface (Fig. 2) with limited water flow and no rotation. Open the packer to ensure that the water level inside the pile equalized to water level in the model tank. Close the packer.
4. *Sand compaction* — Densify the sand using a concrete vibrator by following a specified pattern (Fig. 5). The concrete vibrator was gently pushed down vertically until a defined penetration depth of approximately 600 mm was reached before it was slowly pulled up. Open the packer.
5. *Uniformity testing* — Test the uniformity of the sand model using a miniature cone. Figure 6 depicts the positions of these cone resistance tests.
6. *Pile drilling* — First, the data acquisition was switched on. Then, the flushing was turned on by opening the flow meter to a predefined value. Five seconds later, the pile rotation and penetration were turned on simultaneously. When the pile reached the end position of approximately 460 to 470 mm soil depth, penetration, rotation, and flushing was stopped at the same time and the packer was closed immediately to prevent sand particles flowing out of the pile casing.
7. *Cone resistance testing* — Cone resistance tests were carried out to investigate the influence from pile drilling.
8. *Pile lifting and collection of drill cuttings* — The pile was lifted and drill cuttings in the catch-pot and inside the pile were collected, dried, and weighed.

After the initial soil model preparation (Fig. 4), the sand was very loose with a mean relative density, D_r, of about 0.2. The relative density increased gradually after several rounds of loosening and compaction, reaching values between 0.6–0.65 for the presented tests. The mean relative density was calculated before each test based on the known total dry weight (m_s) and volume (v_s) of the sand in the model tank according to the following equations:

(1) $$\rho_d = \frac{m_s}{v_s}$$

(2) $$e = \left(\frac{\rho_s}{\rho_d}\right) - 1$$

(3) $$D_r = \frac{(e_{max} - e)}{(e_{max} - e_{min})}$$

Fig. 4. Stages in initial soil model preparation: (*a*) support for pore pressure standpipes and perforated tubes for saturation and drainage of soil model; (*b*) filter layer (LECA and geotextile); (*c*) geotextile as separation layer; (*d*) filling of dry sand. [Colour online.]

Table 1. Index properties of Baskarp sand No. 15 (after Ibsen and Bødker 1994).

Property	Value
D_{50} grain size (mm)	0.14
D_{60}/D_{10}	1.78
Grain density, ρ_s (g/cm^3)	2.64
Maximum void ratio, e_{max}	0.858
Minimum void ratio, e_{min}	0.549

Fig. 5. Grid for compaction of sand with concrete vibrator. Points "A" followed by points "B". (All dimensions in millimetres.)

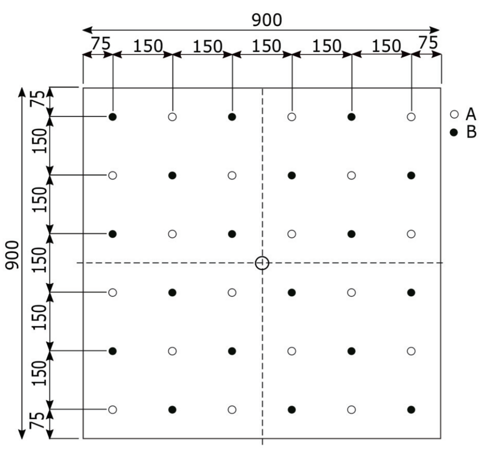
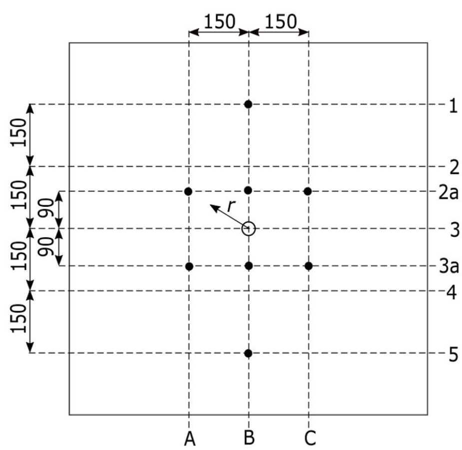


Fig. 6. Layout for cone resistance testing of sand model. Black dots represent positions for testing and *r* radial distance from pile centre. (All dimensions in millimetres.)



Cone resistance tests were carried out before the pile drilling to verify a consistent sand model preparation. A miniature cone with a diameter of 10 mm and an apex angle of 60° was connected to a steel rod and pushed 500 mm into the sand model using an actuator. The penetration rate used was 5 mm/s. Cone resistance tests were generally carried out after each soil model preparation and pile drilling test at distances of 90, 175, and 300 mm from the

Fig. 7. Cone resistance prior to pile drilling against depth measured at different radial distances from pile centre, *r*. [Colour online.]

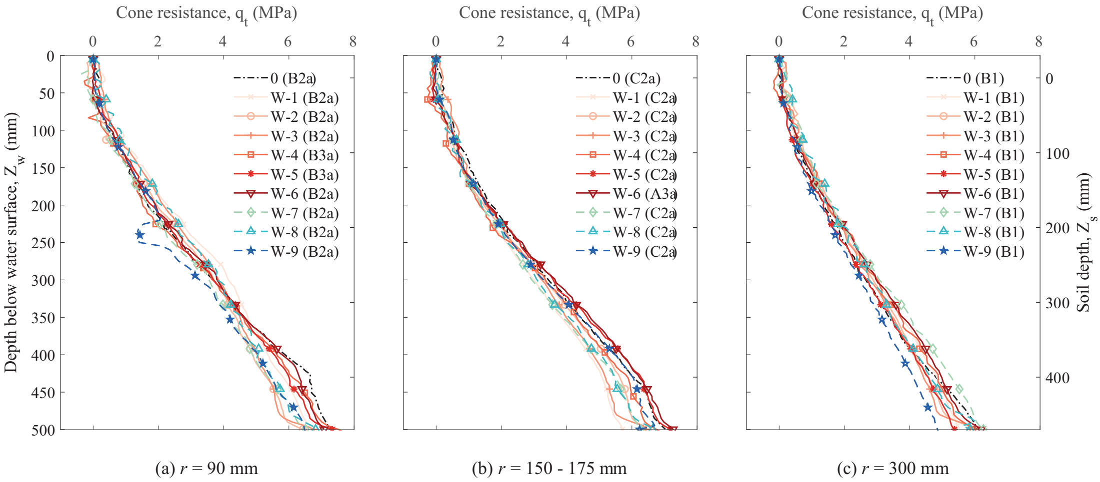
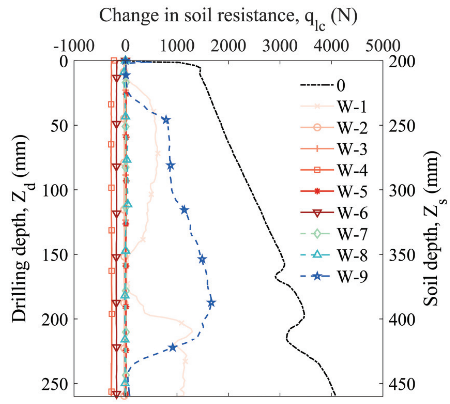


Table 2. Test program.

Test No.	Flow rate (L/min)	Pressure (kPa)[a]	Penetration rate (mm/s)
0	—	—	2.5[b]
W-1	1.5	—	2.5[b]
W-2	2.0	—	2.5
W-3	2.5	—	2.5
W-4	3.0	15	2.5
W-5	4.0	35	2.5
W-6	5.0	60	2.5
W-7	2.0	—	2.0
W-8	2.0	—	3.0
W-9	2.0	—	4.0[b]
A-1	—	50	1.5
A-2	—	75	1.5
A-3	—	100	1.5

[a]Water pressure not measured for tests with flow rate below 3 L/min.
[b]A notable reduction of initial penetration rate of 2.5 mm/s was observed throughout the test.

Fig. 8. Load cell measurements against drilling depth. [Colour online.]



pile centre to assess the influence from drilling (Fig. 6). The order for testing in given positions were swapped for some of the pile tests to investigate potential local differences after the vibro-compaction.

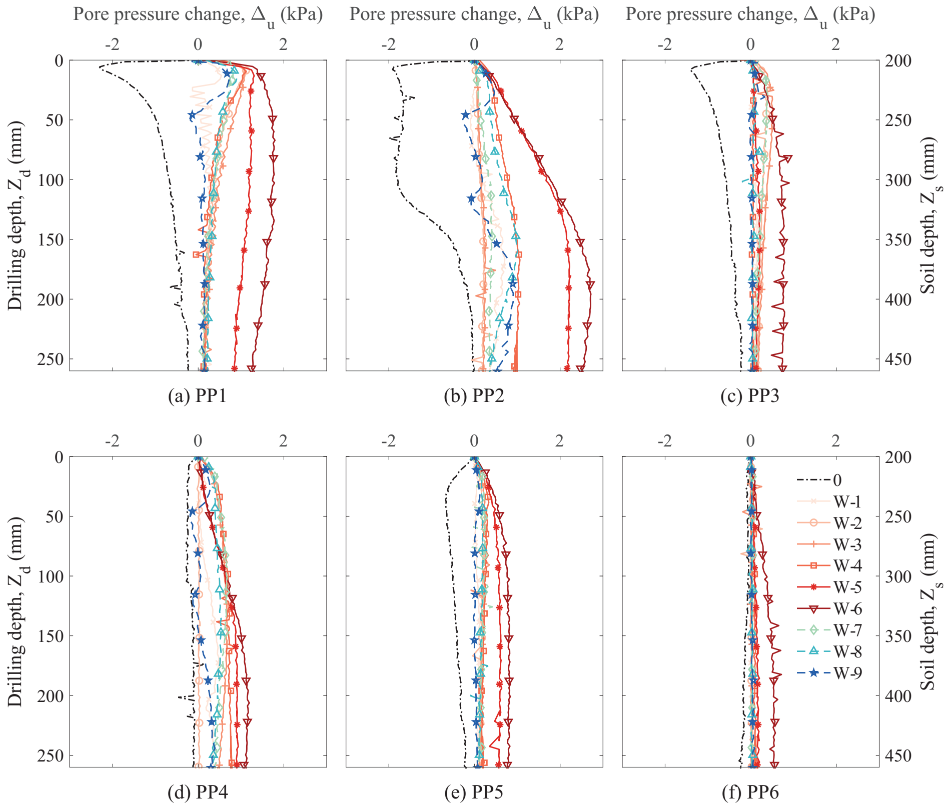
Figure 7 shows measured cone resistance after vibro-compaction at different distances from the pile centre (r) against penetration depth. The results show a relatively high tip resistance at more than 200 mm soil depths, confirming that the compaction had a satisfying effect. The results also confirm a relatively uniform soil model. The data, however, reveal that the soil resistance at distance r = 300 mm (Fig. 7*c*) is slightly lower than at r = 90 mm (Fig. 7*a*) and r = 175 mm (Fig. 7*b*). This could likely be explained due to the vicinity to the model boundary. This trend was found to be consistent for the entire test series and considered to have a marginal impact on the test results. For test W-9, a reduced cone resistance was measured at a depth of approximately 220 mm below the soil surface (Fig. 7*a*). This irregularity was caused by hitting the catch-pot on the model pile during penetration and does not represent the real soil response.

2.4. Test procedure

An overview of the test program for this study is given in Table 2. The prefixes "W" or "A" indicate water or air flushing, respectively. Test 0 was carried out as a reference test by pushing the pile into the sand without any rotation and flushing. The flushing flow rate, Q, was varied between 1.5 to 5.0 L/min for the tests W-1 to W-6, while the penetration rate, V_{pen}, was kept constant at 2.5 mm/s. For the tests W-7 to W-9, the penetration rate varied between 2.0 and 4.0 mm/s while the flow rate was kept constant at 2.0 L/min. The starting value for the pile rotation was kept constant at 20 rpm for all tests except test 0. The flushing water pressure changed according to the given flow rate.

The tests A-1 to A-3 were carried out with the pile tip pre-installed to a starting depth of 400 mm and with flushing pressures of 50, 75, and 100 kPa, respectively. An increased starting depth compared to the water flushing tests was required, because initial tests at a starting depth of 200 mm (i.e., identical to the water flushing tests) caused immediate piping effects on the outside of the pile and drill cutting transport was not observed.

Fig. 9. Measured pore-water pressure changes against pile drilling depth in PP1 to PP6. [Colour online.]



3. Water flushing tests

3.1. Influence on penetration resistance

Figure 8 shows the load cell measurements that provide a qualitative measure of the soil resistance against drilling depth. The start of drilling is at 200 mm soil depth (Figs. 2*b* and 2*c*). Test 0 (reference test) showed an immediate load increase to approximately 1400 N. This resistance aligns well with the expected bearing capacity of the pile tip at 200 mm soil depth. Further measurements show an almost linear increase in penetration force with depth, resulting in a maximum value of approximately 4100 N at a drilling depth of about 260 mm (soil depth Z_s = 460 mm). This equals a tip resistance of approximately 4.3 MPa, which is in the same range as the cone resistance tests (Fig. 7). Small decreases in load were observed at about 160 and 220 mm drilling depths. These differences−deviations from the linear trend could be explained by local inhomogeneities in the sand model.

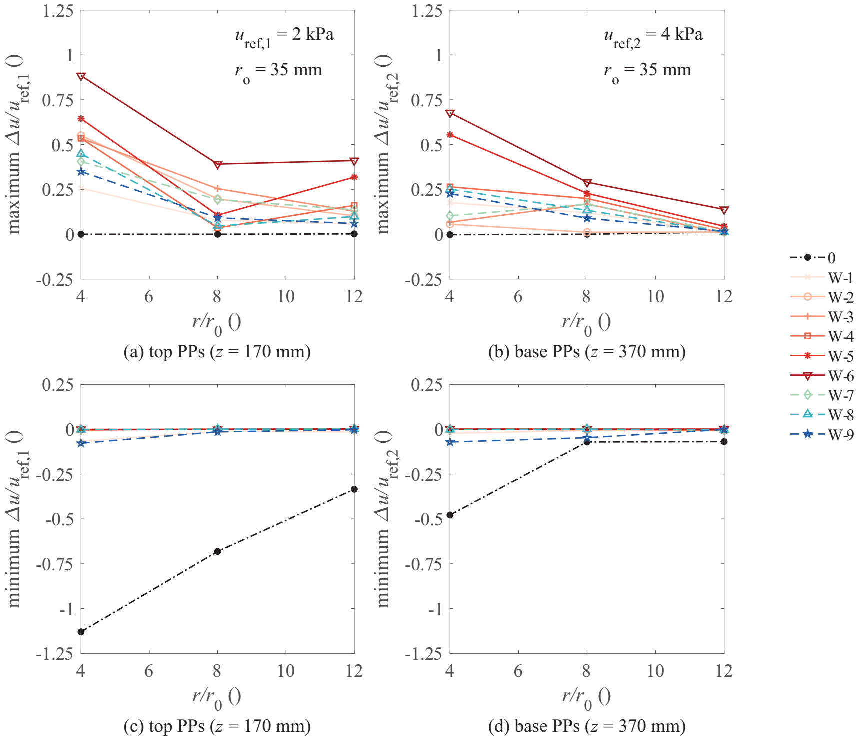
Results from the water flushing tests W-2 to W-8 show no load or negative load values indicating tension caused by the self-weight of the model pile and rotation motor. The value of about −150 N matches the self-weight. This behaviour indicates that the soil did not provide any resistance, which is likely explained by local fluidization of the sand in front of the pile tip due to water flushing. Similar observations for pile jetting tests were reported by Tsinker (1988) and Shepley and Bolton (2014).

The load cell data for the tests W-1 and W-9 indicate that the flow rate (Q = 1.5 and 2.0 L/min, respectively) was too low to cause consistent local fluidization combined with the given initial penetration rate (V = 2.5 and 4.0 mm/s, respectively). For this reason, some soil resistance remained during drilling. This resulted in an increased penetration load compared to the other tests with maximum values of 1275 and 1665 N in tests W-1 and W-9, respectively. A reduction of the initial rotation speed was observed for these tests, which can be explained by substantial friction in the soil–pile interface. The results imply that both flow rate and penetration rate impact the soil behaviour surrounding the pile, and that these two parameters should be considered in combination when studying overburden drilling. A thorough discussion of these two parameters on the performance and effects on overburden will be presented below (Section 3.5).

3.2. Influence on pore-water pressure

Figure 9 presents the measured pore-water pressure changes (Δu) against pile drilling depth for the entire pore pressure sensors (PP1 to PP6) for test 0 (reference) and all the water flushing tests. Test 0 clearly stands out compared to the other tests. The data show a significant decrease in pore pressure as the pile was pushed into the sand; a maximum change of about −2.3 kPa in PP1 (Fig. 9*a*) occurred rapidly after the pile penetration started. The pore pressure slowly increased during penetration, being about 0.5 kPa lower than the initial starting value at the end of

Fig. 10. Normalized change in pore pressure against normalized distance from pile. Maximum pore pressure changes for (*a*) top PPs and (*b*) base PPs. Minimum changes in pore pressure for (*c*) top PPs and (*d*) base PPs. [Colour online.]



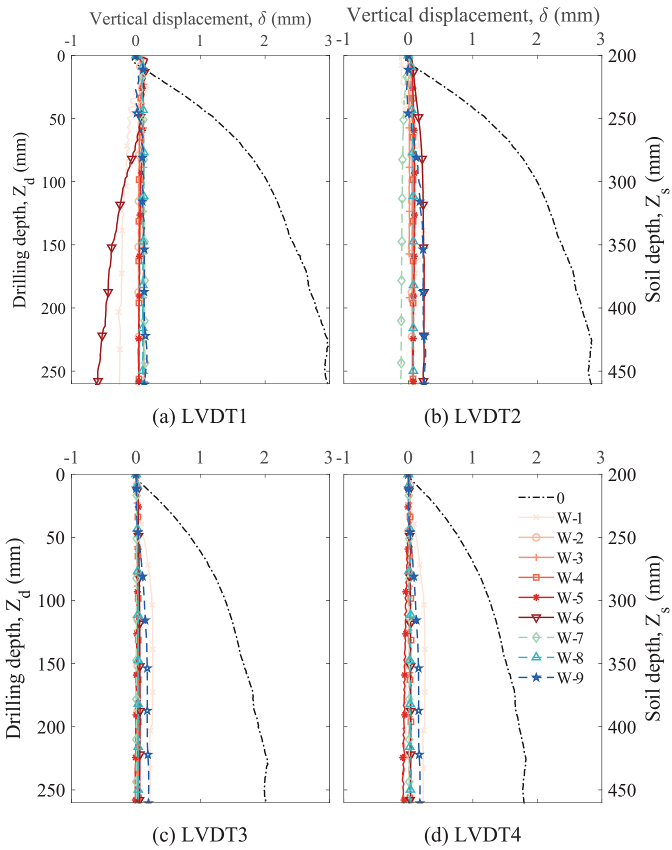
installation. Similar trends were also observed in PP3 (Fig. 9*c*) and PP5 (Fig. 9*e*); however, the influence decreased at greater distance from the pile. PP2 (Fig. 9*b*) showed an immediate pressure drop of about –1.9 kPa, but unlike the "top PPs" (PP1, PP3, PP5) the pressure did not increase again before the pile tip reached a soil depth of about 320 mm. This response may be explained by the pile moving closer to the "base PPs" while the distance to the top PPs increased. Only minor pressure reductions of about 0.2 kPa were measured in PP4 (Fig. 9*d*) and PP6 (Fig. 9*f*). The pressure reductions observed in test 0 are likely explained by dilation effects in the sand surrounding the pile tip and shaft like the behaviour of a driven closed-ended pile (e.g., White and Bolton 2004). When the relatively dense sand is displaced by the penetrating pile, large shear strains develop and cause a volume expansion (i.e., dilation). The soil becomes looser and pore water flows into the voids causing a pore pressure reduction.

All the water flushing tests (W-1 to W-9) caused excess pore pressures in the surrounding sand. The pore pressure changes are very moderate if compared to the applied input drilling fluid pressure up to about 60 kPa for test W-6. This indicates that most of the pressure is likely lost in the flushing tube, the drill bit, and the soil immediately surrounding the pile tip. As expected, the pore pressure values increase with the flow rate. Test W-1 (Q = 1.5 L/min) caused maximum pressure changes of about 0.5 kPa in PP1 and PP2 while test W-6 (Q = 5 L/min) showed corresponding values of about 1.7 and 2.8 kPa, respectively. Tests W-1 and W-9 were the only tests that caused some minor pressure reductions, i.e., negative pressure changes. This behaviour is likely explained by the same dilation effects as observed with test 0, which agrees with the results from the load cell measurements (Fig. 8).

PP1 at only 70 mm distance from the pile centre typically showed an immediate excess pressure when the flushing was turned on, before slowly dissipating again as the penetration depth increased. The base PPs installed at 370 mm soil depth (PP2, PP4, and PP6) displayed a more delayed response in excess pressures compared to the top PPs at 170 mm soil depth. This was expected as the base PPs where furthest from the pile tip at the beginning of the tests; hence, the maximum influence was recorded when the pile tip reached approximately the same depth as the base PPs (i.e., a drilling depth of approximately 200 mm). PP5 and PP6 at a horizontal distance of approximately 210 mm from the pile centre generally showed minor pressure changes during the tests. Only for test W-6, approximately 0.5 to 0.7 kPa excess pressure was measured by PP5 and PP6. For the tests with varying penetration rates (i.e., W-7 to W-9), clear trends in pore-water pressure changes were not observed.

Figure 10 shows the ratio between the pore pressure change (Δu) and the reference pressure (u_{ref}) against the normalized radial distance from the pile (r/r_0), where r_0 is the pile radius and r the radial distance from the pile. Figures 10*a* and 10*b* present the maximum pore pressure change (Δu_{max}) for the top PPs and base PPs, respectively, while Figs. 10*c* and 10*d* present the minimum pore pressure change (Δu_{min}). The reference pressure is defined as the hydrostatic head at the theoretical position of the standpipe ends (i.e., filter positions) that results in a pore-water pressure of 2 kPa for the top PPs and 4 kPa for the base PPs. From Fig. 10 the

Fig. 11. Soil surface settlements against drilling depth for (*a*) LVDT1, (*b*) LVDT2, (*c*) LVDT3, and (*d*) LVDT4. Negative values indicate settlements. [Colour online.]



pore pressure change generally decreases with distance from the pile. This trend is more prominent for the tests with considerable flow rates (i.e., W-5 and W-6) and for the base PPs at 370 mm soil depth (PP2, PP4, and PP6). The data further indicate that the normalized change in pore pressure is less in the base PPs compared to the top PPs. This can be explained by the increase of the reference pressure with depth.

3.3. Influence on soil displacements

Figure 11 presents the measured vertical soil displacement (δ) against drilling depth for the four LVDTs. For test 0, a significant soil heave was monitored. This effect was expected because the pile was pushed in like a closed-ended displacement pile. However, LVDT1 and LVDT2 both show a small (approx. 0.1 mm) settlement before the heave begins after 5 to 10 mm penetration. This behaviour is likely explained by a combination of the penetrating pile and the loose state of the soil close to the surface. The initial pile penetration caused a compaction of the top soil adjacent to the pile before dilation effects became dominant and heave occurred. LVDT1 positioned 35 mm from the pile centre showed a maximum heave of approximately 3 mm (Fig. 11*a*), which reduced to 1.8 mm in LVDT4 at about 210 mm distance from the pile centre (Fig. 11*d*).

Results from the water flushing tests generally showed small soil surface displacements. Test W-1 indicates some minor heave (0.1–0.2 mm) in all LVDTs except LVDT1 closest to the pile that settled at about 0.3 mm. Test W-9 caused heave in all the LVDTs. The heave in tests W-1 and W-9 could be explained by the pile drilling causing some soil displacements as the flushing was only able to partially fluidize and remove the sand in front of the drill bit (see above).

Visual observations after drilling showed that all water flushing tests as well as test 0 caused a small cavity (recess) in the soil surface with about 10 mm influence from the pile casing. Figure 12 shows a photo of such a cavity after the completion of test W-4. This effect could not be captured by LVDT1 due to its too large distance of about 17.5 mm from the pile casing. The size of this cavity remained almost constant for the conducted tests, thus being independent of the flushing flow rate and penetration rate. The cavities most likely occurred because at very shallow depths the failure mechanism does not present any dilative behaviour and so, for the first centimetres of penetration, the soil adjacent to the pile tends to densify. The cone penetration tests confirm this hypothesis showing very low resistance in the first 100 mm of penetration and a similar cavity. A supplementary test that is not reported in this paper was carried out with the same flow and

Fig. 12. Local cavity at soil surface around pile casing after test W-4. [Colour online.]

penetration rate as test W-2, but without pile rotation. This test resulted in a noticeable smaller cavity, which may indicate that the pile rotation even further increased this densification adjacent to the casing.

Given that the flow rate in test W-1 was not able to fluidize the sand completely, some of the soil resistance remained (Fig. 8). The penetrating pile caused less compaction effects as observed in test 0. Some drill cutting transport through the pile occurred, which likely reduced dilation effects and probably contributed to the settlements measured in LVDT1 of test W-1. Test W-6 resulted in about 0.6 mm settlement in LVDT1, but no significant displacement in the other LVDTs. These settlements are likely due to the high flow rate causing considerable erosion and loss of soil volume around the pile, which is further discussed below. The other water flushing tests showed negligible soil surface displacements.

3.4. Influence on soil resistance

Cone resistance tests after each pile drilling test were used to assess the impact of different drilling parameters on the soil. Figure 13 presents the measured cone resistance at different distances from the pile centre (r) against penetration depth for test 0 and the water flushing tests. The results for a distance r of 90 mm indicate a general trend of reduced cone resistance with increasing flow rate (Fig. 13*a*). This difference is particularly noticeable between the tests W-4 to W-6. For the tests W-1 to W-4, this trend is less obvious, which most likely is explained by the relatively small variations in flow rates. Test W-6 clearly stands out compared to the other water flushing tests with a significant lower soil resistance from about 200 mm soil depth until the final depth of about 480 mm. The results show an unexpected lower soil resistance after test W-9 compared to test W-8 even though the penetration rate was higher. Based on the load cell measurements (Fig. 8) it is likely that the high penetration rate (5 mm/s) with test W-9 caused soil displacements and dilation effects that reduced the soil resistance. The same effect could also explain why test W-1 shows less resistance than observed after the tests W-2, W-3, and W-8.

Figure 13*b* shows cone resistance from the positions with a distance between 150 to 175 mm from the pile centre excluding data for the tests W-2, W-3, and W-9. Due to the greater distance to the pile, the trends observed above diminish. The results do not show an influence from any of the tests at a distance of r = 300 mm (Fig. 13*c*).

An interesting observation is that test 0 caused the lowest cone resistance in the surrounding sand for all tests. The considerable installation effect is clearly visible at both 90 (Fig. 13*a*) and 175 mm (Fig. 13*b*) distance from the pile while the impact diminished at a radial distance of 300 mm (Fig. 13*c*). A possible explanation could be that the soil displacements due to the pile penetration without flushing caused large shear strains and volumetric expansion that reduced the soil resistance. This behaviour agrees with results from pile tests in sand (White and Bolton 2004) and triaxial tests on Baskarp sand No. 15 showing large dilation angles up to 18° for low stress conditions (Ibsen et al. 2009). This finding is in accord with results from LVDTs and is to some degree also applicable for the tests W-9 and W-1.

Figure 14 shows the cone resistance at different radial distance from the pile centre before ("pre") and after ("post") the tests W-6 (Fig. 14*a*) and 0 (Fig. 14*b*). As discussed above, test W-6 shows that at r = 90 mm the soil resistance reduced considerably after the pile drilling (B2a versus B3a, Fig. 14*a*) with a maximum difference of approximately 3.2 MPa at about 400 mm soil depth. The results show a notable influence also at 175 mm from the pile (A3a versus C2a, Fig. 14*a*) with a maximum reduction in the cone resistance of about 1.1 MPa. For r = 300 mm the difference between pre- and post-test resistance appears to be negligible (B5 versus B1, Fig. 14*a*). Test 0 shows a similar behaviour. However, a greater reduction in the soil resistance due to the pile test can also be seen at r = 175 mm (A2a versus C2a, Fig. 14*b*). This implies that the radial influence was greater for test 0.

3.5. Effect of flushing parameters on drill cuttings transport

An important aspect to understand the mechanism of overburden drilling is to assess the balance between the generated drill cuttings and the theoretically replaced soil mass represented by the pile volume generated during drilling (i.e., installed pile volume). For this reason, the mass of drill cuttings, M_c, which is the sum of the soil collected in the catch-pot and in the annulus between the casing and the middle tube of the model pile, was measured for each test. The obtained data indicates that the variations of the flushing parameters considerably affected the mass of drill cuttings. To highlight this finding, non-dimensional parameters of normalized flow, Q_{norm}, and normalized mass of drill cuttings, $M_{c,norm}$, were introduced. The normalized flow is defined as:

$$Q_{norm} = \frac{Q}{A_{pile} V_{pen}} \quad (4)$$

where Q is the flushing flow rate in dm^3/min, A_{pile} is the cross-sectional area of the pile in dm^2, and V_{pen} is the penetration rate in dm/min. This dimensionless parameter combines both the flow and penetration rate with the pile area, and therefore provides a simple means to evaluate the effect of flushing parameters on the drill cuttings transport.

The normalized mass of drill cuttings, $M_{c,norm}$, is defined as the ratio between the mass of drill cuttings, M_c, collected from the pile throughout a test and the theoretical mass of soil, M_{pile}, given by the installed pile volume and the calculated relative density of the respective soil model. This calculation disregards potential drilling induced soil displacements and soil volume changes, which is a simplification. A value lower than 1 indicates that the mass of drill cuttings is less than the theoretical one,

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Fig. 13. Cone resistance against depth for test 0 and tests W-1 to W-9 at radial distance from pile centre of: (*a*) 90 mm; (*b*) 175 mm; (*c*) 300 mm. [Colour online.]

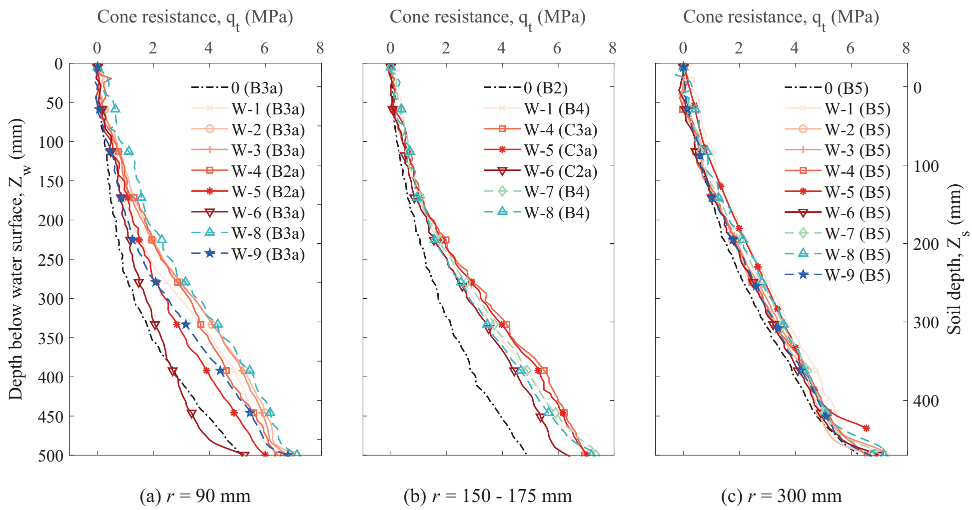
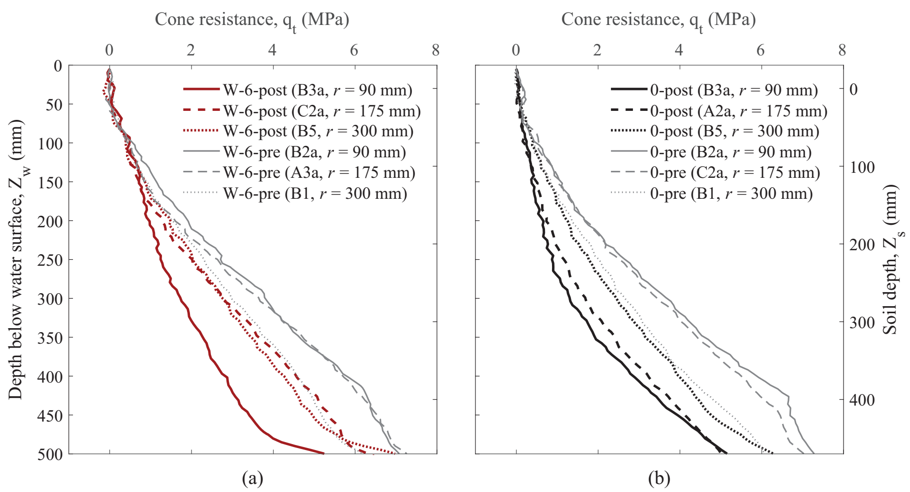


Fig. 14. Comparison of cone resistance against depth at different radial distances (*r*) from pile centre prior to ("pre") and after ("post"): (*a*) test W-6 and (*b*) test 0. [Colour online.]

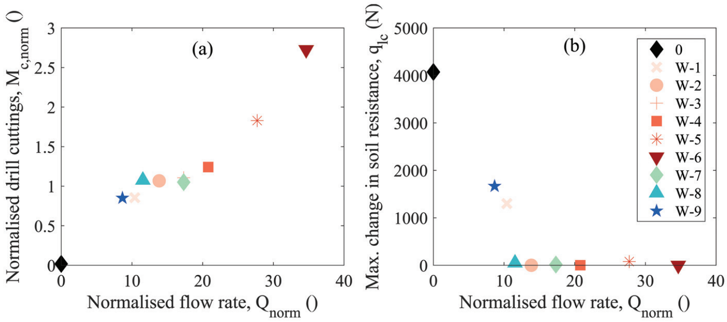


meaning that the soil is likely replaced by the pile drilling. A value above 1 indicates that the mass of drill cuttings is higher than the theoretical mass, hence causing a potential soil volume loss. A value of 1 is defined as an "ideal" scenario.

Figure 15 presents normalized flow rate, Q_{norm}, against normalized mass of drill cuttings, $M_{c,norm}$ (Fig. 15*a*) and the maximum change in soil resistance, q_{lc}, measured by the load cell (Fig. 15*b*). The results show an overall linear trend of increase in normalized mass of drill cuttings with normalized flow rate. The data reveal that an increase in the flow rate caused an increase in the normalized mass of drill cuttings (compare tests W-1 to W-6). By contrast, an increase of the penetration rate reduced the mass of drill cuttings (compare test W-2 and tests W-7 to W-9). This indicates an inverse correlation between the parameters flow rate and penetration rate.

Given that test 0 was carried out without flushing, the normalized flow and the mass of drill cuttings were zero. The tests W-1 and W-9 both resulted in a value for $M_{c,norm}$ of about 0.85 with a corresponding value of Q_{norm} just below 10. This indicates that the installation caused some soil displacement surrounding the pile, which is supported by the observed increase in penetration resistance in parts of these tests as can be seen in Fig. 15*b* (and Fig. 8). For test W-1 the penetration rate of 2.5 mm/s means that a soil volume of approximately 0.14 L/min should be displaced by the pile

Fig. 15. Normalized flow rate (Q_{norm}) against (*a*) normalized mass of drill cuttings ($M_{c,norm}$) and (*b*) maximum change in soil resistance, q_{lc}, measured by load cell. [Colour online.]



tip, i.e., removed by drilling. The flow rate was about 10 times higher (Q = 1.5 L/min), which is similar in value to that of test W-9. This indicates that the flow rate needs to be large enough to be able to attain a specific penetration rate, or alternatively the penetration rate needs to be adapted to the flow rate.

Test W-2 (Q = 2.0 L/min) represents an almost ideal scenario for the modelled conditions with $M_{c,norm}$ of about 1.07, only about 7% excess drill cuttings compared to the installed pile volume was measured. This is in line with the small load cell and pore-water pressure readings observed for this test (Fig. 15*b* and Fig. 9). A maximum $M_{c,norm}$ and Q_{norm} value of about 2.7 and 35, respectively, was obtained for the test W-6 (Q = 5.0 L/min). This significant loss of soil volume likely explains the settlements observed with LVDT1 (Fig. 11*a*). However, the other LVDTs at greater distance from the pile showed minor settlements. This observation might be related to soil loosening (i.e., reduction in relative density) caused by the high flow rate, which likely compensates the soil volume loss adjacent to the pile. The significant reduction in cone resistance measured after the test (Fig. 14*a*) supports this interpretation. At prototype stress conditions it is likely that the extensive loss of soil volume observed for test W-6 would lead to considerable ground settlements.

The experimental data reveal that a normalized flow rate between 10 to 20 results in an "ideal drilling" in terms of drill cuttings balance, i.e., $M_{c,norm}$ equal or close to 1.0. Compared to prototype drilling in medium dense sand with a casing diameter of 76 mm, typical values for normalized flow rate are estimated to vary from about 20 to 55 with a given flow rate, Q, from 80 to 150 L/min and an assumed penetration rate, V_{pen}, from 500 to 1000 mm/min (8.33 to 16.66 mm/s). These values are higher than the obtained "ideal" normalized flow rate and according to Fig. 15*a* would result in a too high drill cutting transport. The difference is likely a result of the low stress conditions modelled, and refined investigations are required to translate this framework into practice.

3.6. Air flushing tests

Tests carried out with air flushing were generally not able to create a successful transport of drill cuttings. An increased starting soil depth of 400 mm improved the flushing backflow. At the beginning of the tests A-1 to A-3 small outbursts of water, sand, and air were observed at the pile top. However, after about 15 to 20 s of drilling the air caused soil fractures and piping (i.e., flow paths) along the outside of the pile wall, which continued until the tests were stopped after 100 mm of drilling. This effect has been observed in the field when drilling with air flushing is carried out at shallow depths or when the flushing pressure is too high (e.g., Lande et al. 2020; Sandene et al. 2021). Due to these challenges the air flushing tests could not be compared with the water flushing tests.

Figure 16 presents pore-water pressure changes (Δu) against pile drilling depth for the tests A1 to A3. The results generally display a reduced pore pressure in the surrounding ground, which could indicate that the air flushing caused an air-lift pump effect as suggested in case studies (Lande et al. 2020; Ahlund and Ögren 2016; Bredenberg et al. 2014). However, the limited dataset makes the analysis challenging and no clear conclusions can be drawn. As seen in the Figs. 16*a*, 16*c*, and 16*e* only minor changes were observed in the top PPs (PP1, PP3, and PP5). This is likely explained by the starting depth of 400 mm and the distance from the pile tip to these top PPs. PP2 (Fig. 16*b*) and PP4 (Fig. 16*d*) at 370 mm soil depth experienced the largest pore pressure reductions with maximum values of about –0.6 kPa for test A-1 and A-2, respectively. Despite the highest flushing pressure (i.e., 100 kPa), test A-3 caused less influence compared to the other tests.

Results from the load cell measurements presented in Fig. 17 suggest that the air flushing was not able to fluidize and loosen the sand as the water flushing did (Fig. 8). In general, an increase in penetration force with depth was observed until the drilling stopped. Only test A-2 showed a reduced resistance from about 45 to 85 mm drilling depth until it increased again. The tests A-1 and A-3 reached a maximum value of approximately 3600 N and test A-2 a value of 2300 N. Given that the load cell data display similar trends and penetration force as with test 0, the pore pressure reductions might be related to dilation effects.

Data from the load cell and the observed lack of drill cuttings transport give reason to assume that the air flushing pressure and flow rate was too low to remove the drill cuttings in front of the drill bit during drilling. Due to the low effective soil stresses sudden piping was observed, and the air pressure could not be further increased. However, after the tests some sand sticking to the inside of the pile casing was detected.

4. Applicability of results

Overburden drilling is characterized by very complex simultaneous processes that are carried out in varying ground conditions. For that reason, some simplifications were necessary in the described experiments. The test set-up did not replicate the details of a percussive hammer that is typically used in overburden drilling to maintain an acceptable penetration rate when drilling in dense granular soils and rock (Sabatini et al. 2005; Finnish Road Authorities 2003). The effect of this parameter on the surrounding ground is likely insignificant for the modelled ground conditions. All tests were carried out under 1g conditions at rather low soil stresses, thus having some unavoidable limitations compared to prototype drilling in the field. Consequently,

Fig. 16. Measured pore-water pressure changes against pile drilling depth in PP1 to PP6 for air flushing tests. [Colour online.]

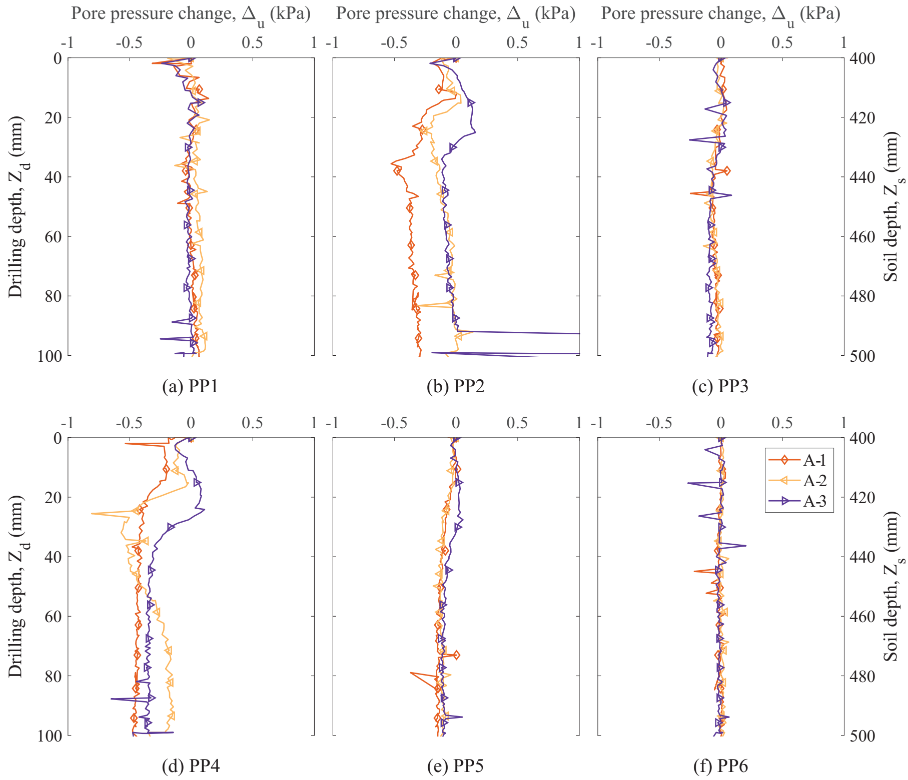
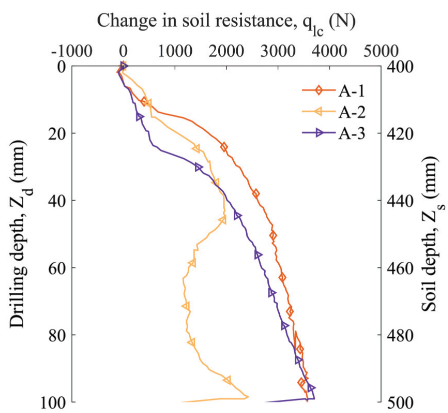


Fig. 17. Load cell measurements against drilling depth for air flushing tests. [Colour online.]



the flushing pressures and flow rates used in the tests were lower than in a real case scenario making it difficult to compare the normalized flow rate values from the tests directly with prototype drilling parameters. The limited soil depth and soil stress most likely affected the range of normalized flow rate at which an "ideal" drilling scenario was identified (Fig. 15*a*).

To further investigate the physical effects from drilling on the surrounding ground and to be able to reliably translate the obtained results into practice, refined model tests including more representative soil stresses, hydraulic conditions (e.g., confined aquifer), and flushing parameters are recommended. Future tests should include higher and more representative soil stresses (e.g., by adding a surcharge) and pore-water pressures representative for drilling at larger soil depth. Higher soil stress would likely require increased flushing parameters (i.e., flow rate and pressure). Other aspects that should be investigated are (*i*) the effect of drilling in a confined aquifer (e.g., under an impermeable soil layer) as frequently observed in the field (Lande et al. 2020; Ahlund and Ögren 2016; Sandene et al. 2021), (*ii*) the influence of varying soil density, (*iii*) different degrees of saturation, and (*iv*) the impact of different pile rotation rates.

It is expected that refined tests will provide further insight into overburden drilling. Such results in combination with the introduced framework of normalized drill cutting transport and flow rate could lead to practical recommendations regarding a more informed choice of overburden drilling systems and parameters.

The introduced framework should be investigated further through full-scale testing to validate its applicability.

5. Conclusions

This paper presents physical model tests to study the effects from overburden drilling of piles on the surrounding ground. Novel experimental data are provided that reveal the impact of different flushing media (i.e., water and air) and flushing parameters such as flow and penetration rate on the penetration force, pore pressure changes, soil displacements, and drill cuttings transport. Based on the results of the water flushing tests, the following conclusions can be drawn:

1. Increasing flow rates caused larger excess pore-water pressures with a greater influence area in the surrounding soil. The measured pore pressure changes were generally small and decreased with the distance from the pile. An increased flow rate generated more drill cuttings and reduced the soil resistance significantly in the soil adjacent to the pile.
2. An increased penetration rate compensated the effects observed when increasing the flow rate. This observation indicates an inverse correlation between these parameters.
3. The drill cutting transport depends on both the flow and penetration. The experimental results indicate an almost linear relationship between the normalized flow rate, Q_{norm}, which relates the flow to the penetration rate and the cross-sectional area of the model pile, and the normalized mass of drill cuttings, $M_{\mathrm{c,norm}}$.
4. For high normalized flow rates, Q_{norm}, the soil in front of the drill bit fluidized, which reduced or practically eliminated the penetration resistance. This response is comparable to observations during pile jetting. The fluidization, however, may lead to considerable ground settlements. For the tests with too low flow rate (e.g., W-1) or too high penetration rate (e.g., W-9), opposite behaviour was observed, and the soil resistance partly remained.
5. The introduced framework of normalized flow rate, Q_{norm}, and normalized mass of drill cuttings, $M_{\mathrm{c,norm}}$, could provide a first effective means to derive ideal drilling parameters. The experimental data reveal that a normalized flow rate between 10 to 20 results in an "ideal" drilling in terms of drill cuttings balance, i.e. $M_{\mathrm{c,norm}}$ equal or close to 1.0. The effect of more representative soil stresses is, however, an area that requires further research.

The air flushing tests were limited by modelling constraints; thus, no clear conclusions can be drawn from these tests. However, a notable reduction of pore pressures adjacent to the casing was measured. This finding may indicate that air flushing causes a behaviour equivalent to an air-lift pump effect that could lead to considerable erosion, soil loss, and resulting ground movements. Similar observations were reported in case studies (Lande et al. 2020; Ahlund and Ögren 2016; Bredenberg et al. 2014).

The presented experimental data provide a new insight into the mechanisms of overburden drilling on the surrounding ground. Refined model tests that should focus on more representative stress conditions and flushing parameters are recommended. In addition, full-scale tests should be explored to further assess the introduced framework of normalized drill cutting transport and flow rate to evaluate the obtained data.

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List of symbols

A_{pile} cross-sectional area of pile (mm^2)
D_{10} 10% fractile in grain-size distribution (mm)
D_{50} 50% fractile in grain-size distribution (mm)
D_{60} 60% fractile in grain-size distribution (mm)
D_r relative soil density
e void ratio
e_{max} maximum void ratio
e_{min} minimum void ratio
M_c measured mass of drill cuttings (g)
$M_{c,norm}$ normalized mass of drill cuttings
M_{pile} theoretical mass of soil given by installed pile volume (g)
m_s mass of dry sand in model tank (g)
Q flushing flow rate (L/min)
Q_{norm} normalized flow rate
q_{lc} change in soil resistance (N)
q_t cone resistance (MPa)
r radial distance from pile centre (mm)
r_0 pile radius (mm)
u_{ref} reference pore-water pressure (kPa)
Δu change in pore-water pressure from reference value (kPa)
Δu_{max} maximum pore pressure change (kPa)
Δu_{min} minimum pore pressure change (kPa)
V_{pen} drilling penetration rate (mm/s)
v_s volume of dry sand in model tank (cm^3)
Z_d drilling depth (mm)
Z_s soil depth (mm)
Z_w depth from water surface (mm)
δ vertical displacement of soil surface (mm)
ρ_d dry density (g/cm^3)
ρ_s grain density (g/cm^3)
$\varnothing$ pile diameter

Paper V

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Understanding effects from overburden drilling of piles – a rational approach to reduce the impacts on the surroundings

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Abstract

This paper presents two case studies dealing with undesirable impacts of overburden drilling of casings for end bearing piles to bedrock. Monitored pore-water pressures and ground settlements are used to document and assess the influence from rotary percussive drilling with "down-the-hole" (DTH) hammers. The studies show that drilling with high-pressure air driven DTH hammers may cause considerable erosion and soil volume loss adjacent to the drill bit and along the casing resulting in settlements of the surrounding ground. The risk of soil volume loss increases when the drilling is carried out in erodible soils like silt and fine sands. The volume loss is found to be caused by a combined air-lift pump effect and a Venturi suction effect. Monitoring pore pressures in the vicinity of the drilling may be used to reduce soil volume loss and prevent damaging settlements. Results from drilling with water-driven DTH hammer showed significantly less ground settlements and influence on pore pressures compared to using an air-driven hammer. The study suggests that the drilling parameters flow rate and penetration rate, and the cross-sectional area of the pile casing can be combined in a non-dimensional methodology to assess the mass balance of drill cuttings when drilling with water flushing. A design framework is suggested to guide overburden drilling in urban settings to reduce potential impact on the surroundings.

Keywords

Overburden drilling; piles; settlements; pore pressure, installation effects, soil erosion

List of notations

A_{pile}	is the cross-sectional area of the pile
$V_{BF,c}$	is the total volume of backflow from drilling in clay
$V_{BF,t}$	is the total volume of backflow from drilling in till
c_{uA}	is the undrained active shear strength
d	is the installation soil depth
δv	is the vertical displacements
γ	is the soil unit weight
$L_{p,c}$	is the length of casing in clay
$L_{p,t}$	is the length of casing in till
$M_{c,c}$	is the soil volume balance in clay given by $V_{c,c}/V_{p,c}$
$M_{c,t}$	is the soil volume balance in till given by $V_{c,t}/V_{p,t}$
$M_{c,norm}$	is the normalized mass of drill cuttings
PZ	is the piezometer
Q	is the flow rate of water flushing
Q_{norm}	is the normalized flow rate

ρ	is the soil density
S	is the settlement anchor
SPW	is the sheet pile wall
U	is the measured pore-water pressure
u_{ref}	is the reference pore-water pressure
ΔU	is the pore pressure change
ΔU_{max}	is the maximum pore pressure change related to u_{ref}
V_{pen}	is the penetration rate in m/min
$V_{p,c}$	is the volume of pile casing in clay
$V_{p,t}$	is the volume of pile casing in till
$V_{c,c}$	is the volume of in-situ clay from drilling backflow
$V_{c,t}$	is the volume of in-situ till from drilling backflow
w	is the soil water content

1. Introduction

To effectively utilize the piles axial capacity, end bearing piles are often preferred in areas with limited depths of soft soils overlying solid bedrock. Required embedment in bedrock may be obtained by drilling a continuous permanent casing to support the borehole through varying soils (i.e. overburden) and with an embedment into bedrock (Finnish Road Authorities 2003), an installation method called overburden drilling. The pile is installed or cast in the open casing after drilling. Studies of deep excavations reported by Langford et al. (2015) showed that overburden drilling both for tieback anchors and piles may cause ground settlements considerably larger than what is related to lateral deformations of supporting walls around an excavation (e.g. Peck 1969; Mana and Clough 1981; Long 2001; Karlsrud and Andresen 2008).

Overburden drilling typically uses rotary percussive duplex drilling (FHWA 2005). The percussion on the drill bit is provided by either a hammer acting on top of the drill rod or a down-the-hole (DTH) hammer located just above the drill bit (Finnish Road Authorities 2003). These drilling techniques are widely adopted since they enable continuous and straight penetration through soft soils, fill material, boulders and into hard bedrock (FHWA 2005). This paper focuses on DTH hammer drilling driven by either air (Halco Rocktools 2021) or water flushing (Wassara 2021) to install pile casings with outer diameters (OD) from 140-1016 mm. DTH air hammers are often preferred for cost and practical reasons since water hammers require more expensive drilling equipment and large amounts of clean water that often involves implementing efforts to handle the back flow at the construction site (Veslegard and Simonsen 2014). In addition, the water hammer casing dimensions are presently limited upwards to 500 mm.

Installation effects from overburden drilling have been studied through field tests (Lande et al. 2020; Ahlund and Ögren 2016) and case records (e.g. Sandene et al. 2023; Sandene et al. 2021; Asplind, 2017; Bredenberg 2014; Küllingsjö 2007; Konstantakos et al. 2004). The field tests showed that drilling with DTH air hammers caused more ground settlements and changes in pore-water pressures in the surroundings compared to drilling with water-driven hammers. Data from both field tests and case records indicated immediate ground settlements when drilling with air hammers through silty and sandy soils. The sudden displacements appear to be related to the so-called air-lift pump effect (Behringer 1930; Kato et al. 1975) and the Venturi effect (Venturi 1799; Bredenberg 2014) causing suction around the drill bit, leading to significant erosion and soil volume loss around the drill bit and the casing. Neither the field tests nor case records had detailed monitoring to directly link the observed settlements to soil loss and the air-lift pump effect.

The principle of an air-lift pump is schematically illustrated in Figure 1(a). Injection of compressed air at the bottom of a riser tube reduces the density of the air-water mixture in the riser tube, thus creating a lower pressure compared to the water pressure outside the tube. This causes a flow upwards in the riser tube by the surrounding water with higher density. Figure 1(b) illustrates overburden drilling with a DTH air hammer where the air-lift pump effect may lead to substantial ground water flow towards the drill bit resulting in considerable erosion of soil particles and volume loss (i.e. cavities) around the casing. It is found that the Venturi effect enhances the air-lift effect due to the high air flow velocity around the drill bit contributing to lower static water pressure than in the surrounding ground (Bredenberg et al. 2014). Drilling in erodible silty and sandy soils combined with artesian pore pressures and a high degree of ground water recharge is expected to increase the adverse effect of the drilling.

This paper investigates the effects of overburden drilling of pile casings on the adjacent ground through two well documented case histories, which are first presented and discussed. Subsequently, the field data from the two practical cases are compared to previously published data (Lande et al. 2021). The findings are finally translated into a suggested design framework that can be adopted in practice to reduce the impacts from drilling on the surroundings.

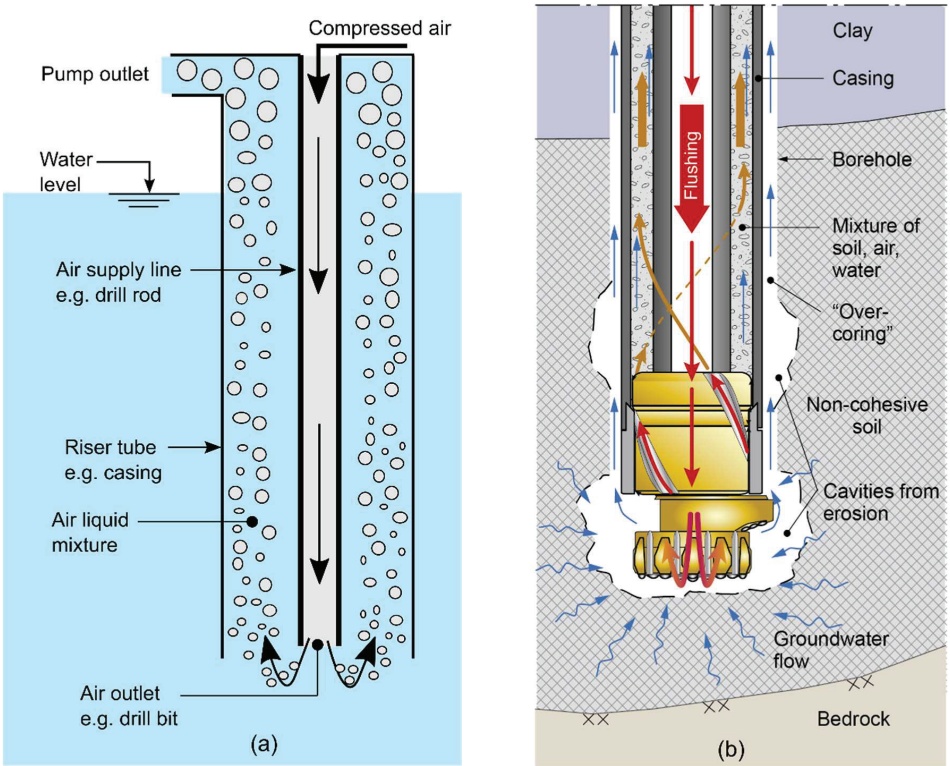


Figure 1. Illustration of (a) principals of an air-lift pump; and (b) erosion and loss of soil volume from drilling with air flushing.

2. Case 1

2.1 Project description

This project involved a 290 m long, and 28 m wide concrete bridge founded on nine end-bearing pile groups (Figure 2). All piles were installed by drilling a continuous permanent casing (OD = 711 mm, 12.5 mm thickness) through the soil and with 1.5 m embedment into bedrock. The casings were reinforced and cast with concrete after the drilling was completed and the borehole was cleaned. The sections below focus on the overburden drilling of pile groups 3, 4, and 5.

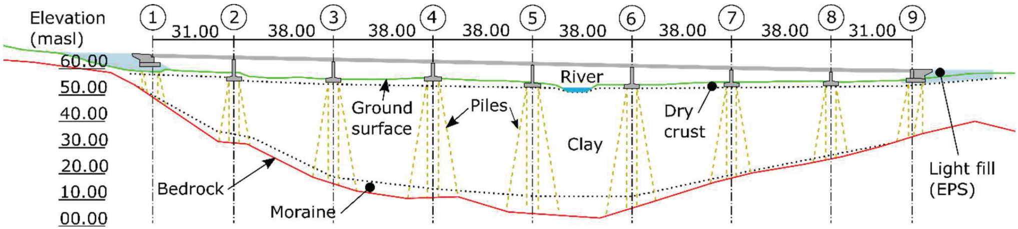


Figure 2. Longitudinal profile of the bridge founded on nine pile groups. Bedrock and soil stratigraphy based on ground investigations and pile drilling logs. Units in meter.

2.2 Ground conditions

Ground investigations (Rambøll Norway 2012) indicate a thin layer of organic topsoil over 2-4 m of dry crust and about 7-48 m of medium stiff to stiff marine clay. Above bedrock a 1-9 m thick layer of dense glacial till with silty and sandy material was detected. The depth to bedrock varies from 10-55 m (Figure 2).

Laboratory investigations on undisturbed clay samples showed water contents (w) between 17-25% and unit weights (γ) between 17.1-18.8 kN/m^3. Interpreted cone penetration tests (CPTu) showed that the undrained active shear strength (c_{uA}) in the clay increased from 40 kPa at 4 m depth to approximately 100 kPa at 40 m depth. Rotary-pressure soundings indicated that the clay is highly sensitive in the deeper parts just above the till or bedrock, but also in "pockets" within the clay deposit.

Pore-water pressure data showed that the ground water level typically varied from 1 m below ground surface by the river to about 4 m below ground at the bridge abutments where the elevation was 4-6 m higher than by the river. Piezometers installed in the till showed an artesian pressure approximately 15% higher than the hydrostatic pressure.

2.3 Drilling method and procedures

A DTH air hammer and a concentric drilling system was used, as shown in Figure 3. The five holes in the center of the pilot bit indicate where the compressed "exhaust" air exits after passing through the DTH hammer. The exhaust air guides the drill cuttings to channels between the casing and the pilot bit (red arrows in Figure 3) and transports them to the surface through the annulus between the casing and drill rod.

Typical air flushing pressures between 5-10 bars were used when drilling through clay, and 12-20 bars when drilling through the till and into bedrock. Water flushing with a typical pressure of 10 bar was also combined with air flushing, to loosen the soil. This was followed by air flushing to remove the drill cuttings and water from the borehole. The water flow rate (Q) varied between 250 L/min during drilling through clay and up to about 350 L/min when drilling through the till. The penetration rate (V_{pen}) was generally kept between 70-100 cm/min through clay. Due to the high drilling resistance in the till, the penetration rate was mostly between 10-15 cm/min but in some cases as low as 3-4 cm/min. A rotational speed of 3-4 revolutions per minute (rpm) was used through the different soils.

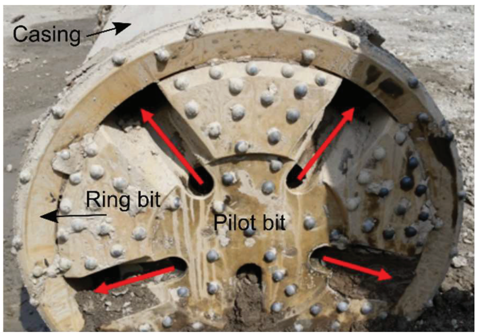


Figure 3. Concentric pilot and ring bit for a 711 mm diameter casing. (Photo by Einar John Lande)

2.4 Instrumentation

Figure 4 shows that five Borros type settlement anchors (Geokon 2020) and four electrical piezometers (PZ) were installed adjacent to pile group 4. PZ1, PZ4, and anchor S5 were installed at the top of the till layer, while the other instruments were in the clay. Table 1 shows the depths of each instrument.

Vertical displacements were measured on top of each anchor rod using a total station theodolite. The accuracy of the measured values was approximately ±5 mm. Settlements were monitored regularly over a total period of about 9 months, starting about six weeks prior to the drilling of pile group 4 and lasting about six months after drilling in pile group 4 was completed. The measurements were generally carried out between 2-4 times each week during pile drilling in pile group 5, 3, and 4 to be able to document immediate effects. After drilling was completed in pile group 4, the subsequent measurements were carried out with longer time intervals (between 1-4 times each month) and only on anchor S4 and S5 since the others were destroyed by an excavator.

The piezometers were installed at the same time as the pile drilling started in pile group 5 which was approximately 8 weeks before the drilling started in pile group 4. The sensors had a general logging frequency of one-hour over a total period of 4 months, except PZ4 which logged data every 30 minutes during the period of drilling in pile group 4.

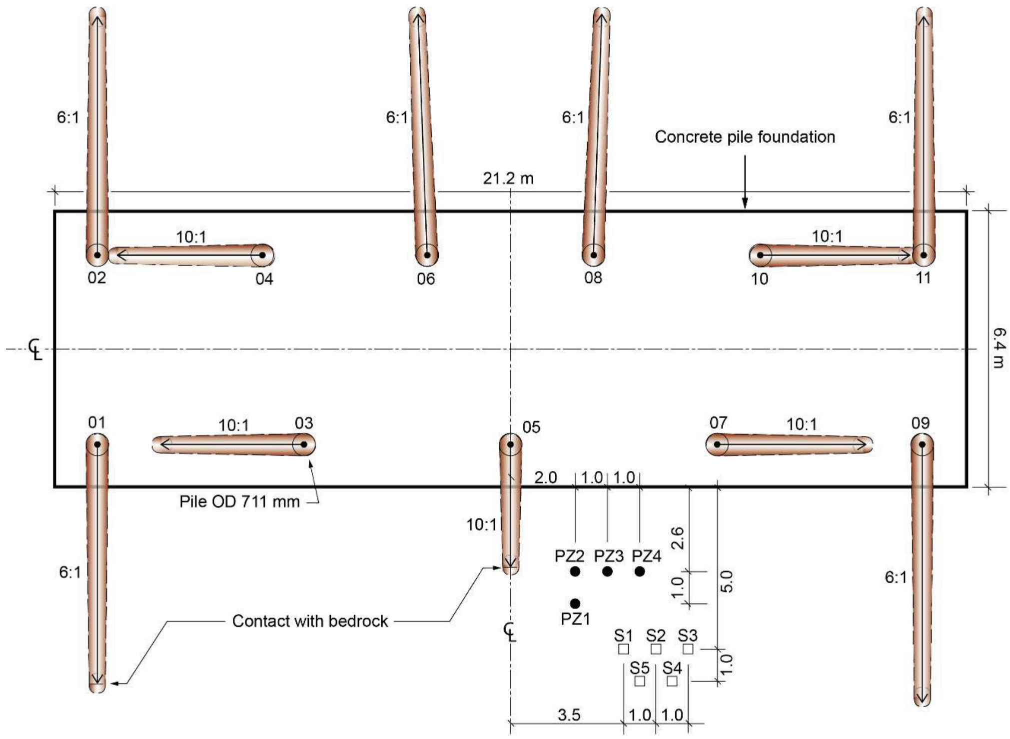


Figure 4. Foundation 4 with 11 piles (inclined 6:1 and 10:1) including settlement anchors (S1-S5) and piezometers (PZ1-PZ4).

Table 1. Instruments installed adjacent to pile group 4. (S = settlement anchor and PZ = piezometer).

Instrument	S1	S2	S3	S4	S5	PZ1	PZ2	PZ3	PZ4
Depth [m]	16	26	31	36	41	43	36	36	42

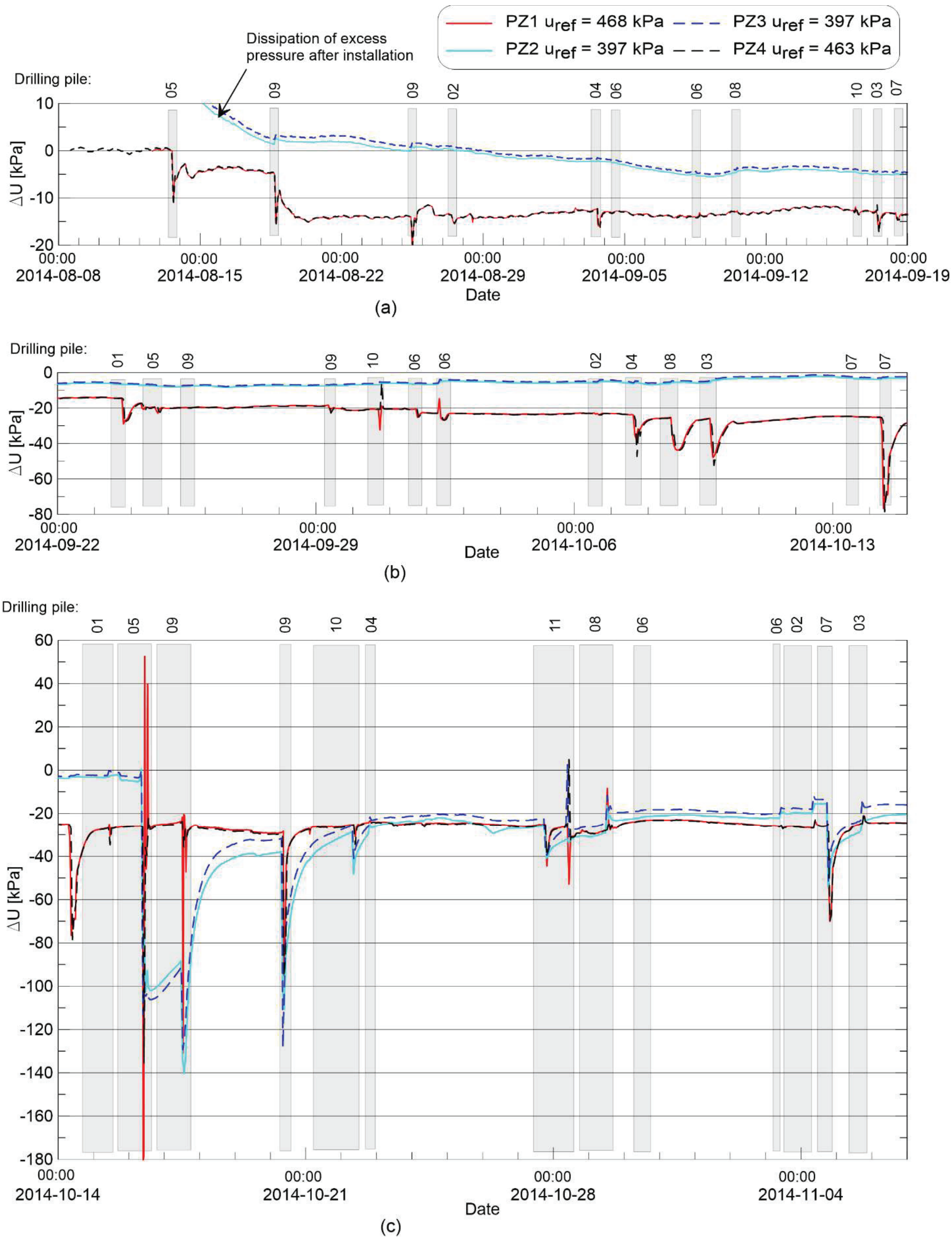
2.5 Influence on pore-water pressure

Figure 5 presents pore pressure changes (ΔU) against time during drilling of pile groups 5, 3, and 4. The pressure changes were related to the reference pressures (u_{ref}) at the given soil depth, based on measurements prior to drilling. The reference pressures for PZ2 and PZ3 installed in clay were chosen after the excess pressure from installation had dissipated and the pressure stabilized (approx. 1 week after installation).

The largest pore pressure changes were measured when drilling closest to the piezometers. A general trend of sudden pore pressure reductions was observed in the piezometers PZ1 and PZ4 when drilling through the till. Within a short time (1-6 hours) after drilling stopped, the pressure increased again. However, the pore pressure remained lower than before drilling of pile groups 5 (Fig. 5(a)) and 3 (Fig. 5(b)). Overall, a pore pressure reduction of about 25 kPa was observed in the till.

The piezometers in clay (PZ2, PZ3) did not show any sudden pore pressure changes when drilling pile groups 5 and 3. After the excess pore pressures from the installation had dissipated (about 20th August), the pressure decreased gradually to about 5 kPa below the reference value during drilling in pile group 5. The pore pressure remained unchanged during drilling of pile group 3.

Figure 5(c) shows that drilling of pile group 4 caused the greatest impact due to the close distance to the piezometers. Sudden pressure reductions were registered in both the till and the clay. The pre-drilling pressure in the till typically recovered within a few hours after drilling, indicating a high hydraulic conductivity and recharge of ground water in the till. The clay showed a delayed response. After drilling of piles 05 and 09, the pore pressures in PZ2 and PZ3 did not recover completely and were approximately 20 kPa lower than the reference pressure when the drilling was completed in pile group 4. Five weeks later the pressure remained 7-9 kPa below the reference pressure.



237

238 **Figure 5.** Pore pressure changes (*ΔU*) measured in piezometers installed adjacent to pile group 4 during

239 pile drilling in: (a) pile group 5; (b) pile group 3; (c) pile group 4. The drilling time for each pile is marked

240 with grey rectangles.

241

Due to the small distance to the piezometers (min. 1.7-2.4 m from pile 05 to PZ2 and PZ1), the most severe effects were observed when drilling for piles 05 and 09. Maximum pressure reductions of about 140 kPa (PZ2) and 115 kPa (PZ3) were registered in the clay, and about 180 kPa (PZ1) and 105 kPa (PZ4) in the till. The data also show rapid shifts between pressure reductions to pressure increase when drilling in the till for pile 05, 11, and 08 with a maximum excess pressure of 52 kPa registered in PZ1 during drilling of pile 05. These shifts were likely related to alternating water and air flushing. The logging frequencies of 30 and 60 minutes probably explain why these variations were not observed for all piles and that some piles located close to the piezometers (e.g. piles 01, 03) showed less impact than others further away (e.g. piles 08, 11).

2.6 Influence on ground settlements

Figure 6 presents vertical ground displacements, δ_v, against time for the settlement anchors adjacent to pile group 4. The reference measurements were taken on September 1st, 2014, about 3 weeks after the drilling was started in pile group 5 and went on until 4th May 2015 (Fig. 6(a)). The data show a general fluctuation explained by ongoing ground works affecting the measurement accuracy. This probably explains the small heave (about 0 to 10 mm) registered during drilling of pile groups 5 and 3.

Drilling of pile group 4 caused considerable ground settlements. Anchor S5 installed in top of the dense till layer at 41 m depth generally showed larger accumulated settlement than the shallower anchors. The greatest impact was registered when piles 01, 05, and 09 were drilled, as shown in Figure 6(b). During this period (14th to 20th October) anchor S5 settled 45-50 mm while the other anchors settled between 15-30 mm. Despite some variations in the measurements, the settlements increased when drilling the remaining piles in pile group 4 and reached a maximum value between 55-60 mm in S5 and a minimum value of about 30 mm in S1. The immediate settlements in S5 imply that drilling in the till caused extensive soil volume loss. The smaller settlements in S1 to S4 indicate a delayed response in the clay.

Three months after drilling of pile group 4, additional settlements of approximately 10 and 25 mm were observed at anchors S5 and S4 (Fig. 6(a)). Later measurements indicated that the settlements subsequently ceased. The anchors S1 to S3 showed small settlements (0-5 mm) about six weeks after drilling was completed. At that time these anchors were accidentally destroyed by an excavator.

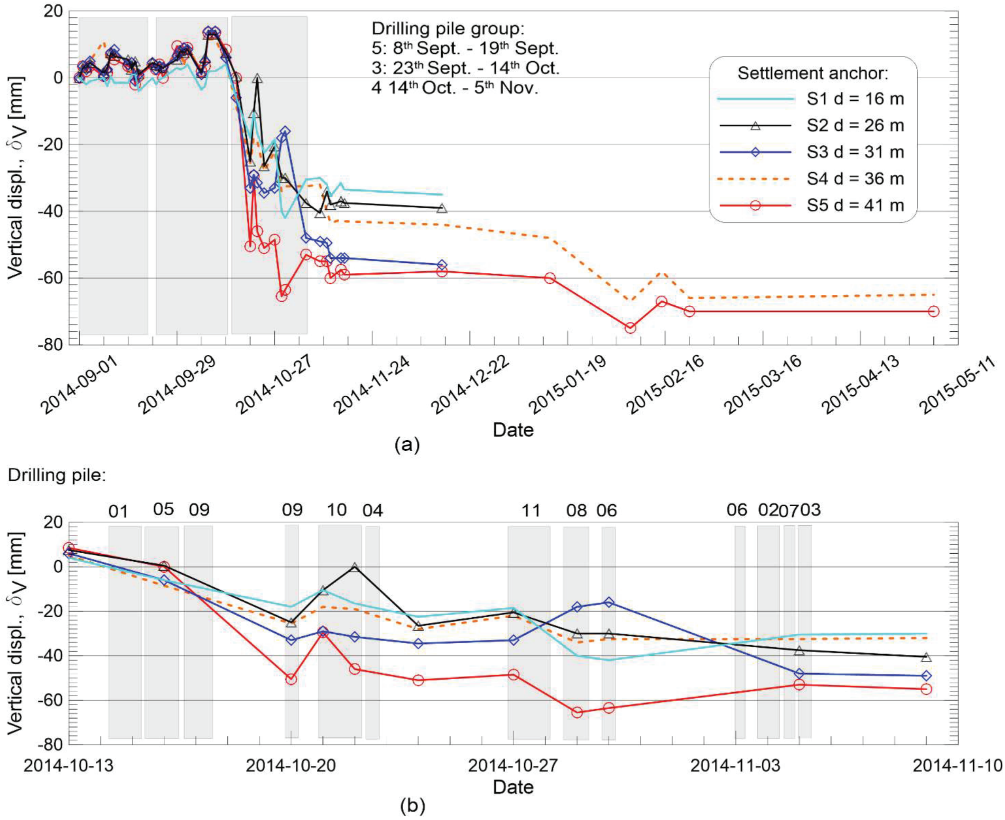


Figure 6. Vertical displacements (δ_V) of settlement anchors S1 to S5 for: (a) the entire surveying period; (b) during drilling in pile group 4. The drilling time is marked with grey rectangles. Negative values represent settlements.

3. Case 2

3.1 Project description

This case involves the construction of a large building complex close to Oslo, Norway. The construction involved an approximately 4 m deep excavation within a 15-24 m deep sheet pile wall (SPW) supported by one level of horizontal tieback anchors connected to a secondary anchoring wall (5-8 m deep), as illustrated in Figure 7. After excavation to the final level, an approximately 0.2 m thick reinforced concrete slab was cast to support the SPW and act as a working platform for drilling more than 1,000 end bearing steel core piles with diameters between 150-230 mm. The steel cores were installed in permanent casings, which were drilled a minimum length of 1.0 m into bedrock using DTH drilling systems. More than 350 of the piles were installed within the building pit for the basement, which has a total footprint of about 13,000 m^2.

Figure 7(a) presents a layout of the building pit including the pile positions in the central part. The building pit was divided into areas A-I; area E is discussed in detail below. Figure 7(b) shows a cross-section through the areas E and F which visualizes the ground surface, soil layers, bedrock surface, SPW and positions of piezometers.

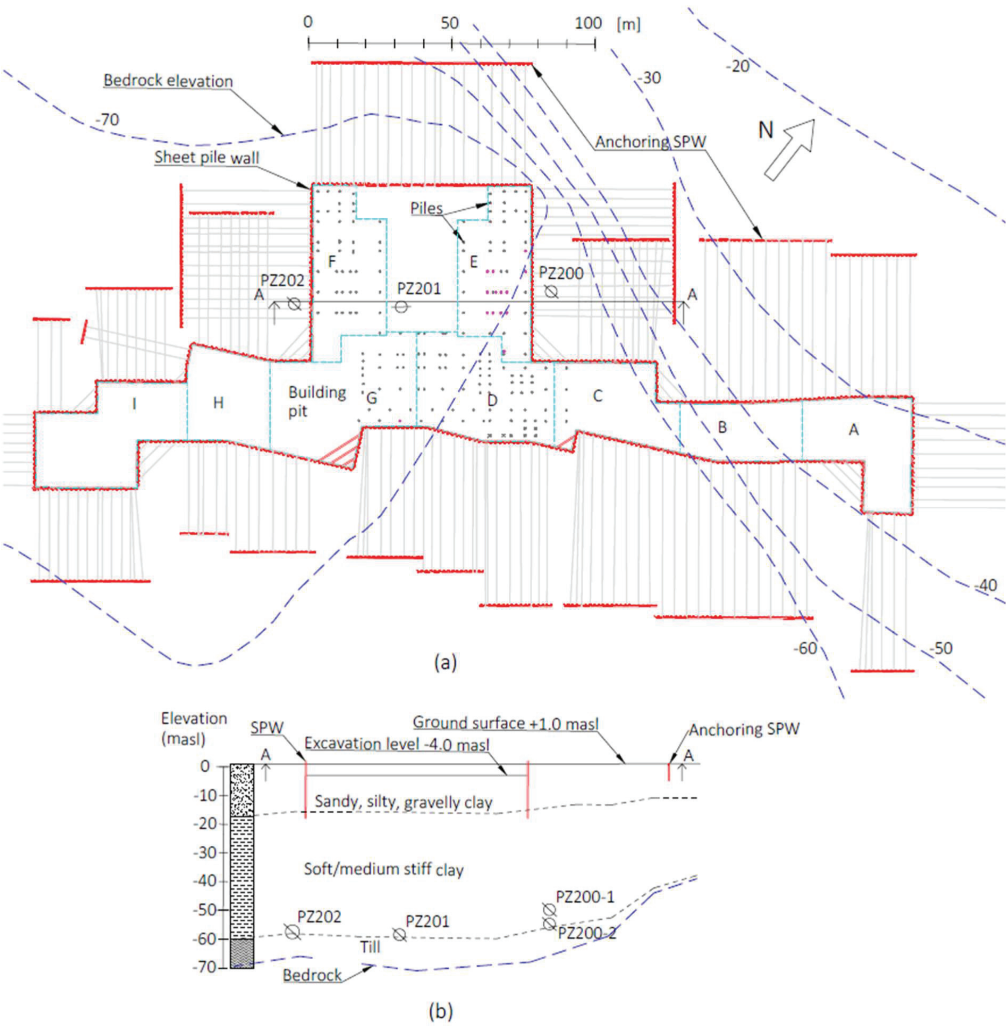


Figure 7. Overview of building pit: (a) layout with sheet pile wall (SPW) with tie-back anchoring system and (b) cross section A-A through area E and F. Area E indicates the area of interest for this study.

3.2 Sequence of events

The excavation and casting of the concrete slab was completed in May 2020. The pile drilling started on the 29^{th} of May in Area A (Fig. 7(a)) where the thickness of the glacial till and depth to bedrock was modest. A DTH air hammer was used. The drilling continued further west to the deeper central part, starting in Area E on the 13^{th} of July. In the beginning of September, considerable displacements occurred, causing local damages on the SPW and anchoring system at the corner of Area E and Area D. The pile drilling continued in the western areas F, G, H, and I under close surveillance. However, on the 30^{th} of October the pile drilling had to stop due to major ground displacements affecting the SPW, concrete slab and the pile casings. Alternative drilling methods were considered, and it was decided to carry out test drilling with a water-driven DTH hammer. The test program involved drilling of 13 casings: nine with an outer diameter of 273 mm, and two with 324 mm and 406 mm diameter.

3.3 Ground conditions

The ground elevation varied between +1.0 to +1.5 masl. Ground investigations mainly consisted of rotary pressure soundings to bedrock, cone penetration tests (CPTu) and core sampling (72 mm diameter) to a maximum soil depth of 27 m. The investigations showed layers of sandy, silty and gravelly clays to depths between 10-15 m followed by a more homogenous soft to medium stiff silty marine clay deposit. Laboratory tests on undisturbed core samples showed that the clay deposit had sensitivities mostly between 5 and 20. Rotary-pressure soundings indicated major clay deposits continuing to depths greater than 60 m and a compact layer of glacio-fluvial till (silty, sandy, gravelly soil) with thickness ranging from 5-15 m above bedrock. The bedrock depth varied from less than 30 m in the eastern area to more than 70 m in the central and western area (Fig. 7(b)).

The ground water table coincided with the sea level (i.e. 0 masl). Measurements showed an approximate linearly increasing artesian pore-water pressure with depth, and at the top of the till layer (i.e. a depth of approximately 60 m), the pore pressure exceeded the hydrostatic pressure by up to 100 kPa.

3.4 Drilling method and procedures

Initially, a similar drilling system as in Case 1 using a DTH air hammer and a concentric pilot- and ring bit was chosen. A so-called “telescope” solution was adopted, see illustration in Figure 8. This involved first drilling a 406 mm diameter casing. For situations where the 406 mm casing could not be drilled through the dense till, a smaller casing (323 or 273 mm) was then drilled deeper inside the 406 mm casing.

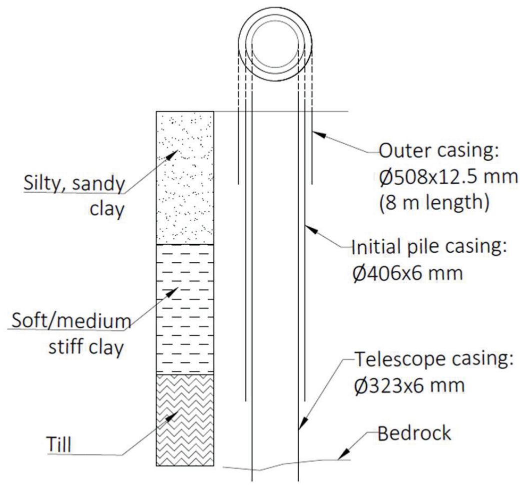


Figure 8. “Telescope” pile solution including an outer casing. Illustration not to scale.

Drilling through the clay deposit was mainly carried out with rotation and combined air and water flushing. A moderate amount of added water (Q ~ 100-200 L/min) loosened the clay which was then lifted up through the casing and to the ground surface by compressed air flushing. A moderate penetration rate of about 100 cm/min was used to limit soil displacements in the clay. When the till layer was reached, the air pressures was increased to 12-15 bars to drive the DTH hammer. The drilling penetration rates in the till were low, down to 2-3 cm/min at the lowest. It was observed that the drill cuttings contained a considerable amount of naturally rounded (i.e. uncrushed) stones from the till deposit which implies that the hammer did not perform as intended. This could be related to the high pore-water pressures measured in the till (u_{ref} = 560-610 kPa) combined with significant ground water flow into the casings. This reduced the effective air pressure driving the percussive hammer as described by Halco Rock Tools (2021).

The test drilling with the water hammer utilized a double head rotary drill rig. This enabled the casing to be rotated separately from the drill rod and the concentric pilot bit. Drilling through the clay deposit was carried out with a penetration rate between 1.2-4.4 m/min with an average of 2.2 m/min. The water flow rate varied between 380-1100 L/min for the Ø406 mm casings with an average value of approximately 700 L/min. In the till layer, substantially higher penetration rates (i.e. 0.25-1.37 m/min with an average of 0.9 m/min) were achieved compared to the air hammer.

3.5 Instrumentation and monitoring

Prior to the basement excavation, geodetic surveying points were established on top of the SPW and the secondary support wall. The measuring frequencies varied from a few days to a few weeks during the excavation work and were not followed up on a regular basis during the first seven weeks of pile drilling. When the large displacements occurred, supplementary surveying points were established on the SPW around the building pit and on bolts fixed to the concrete working platform cast directly on the ground. Prior to the test drilling, additional survey points were also established on top of 7 of the outer 508 mm casings in Area E. The monitoring frequency was increased to better assess the influence from the remaining pile drilling.

In addition to the surveying points, 10 electrical piezometers were installed. Five were installed at the top of the till layer at soil depths between 53-59 m, while five were installed in the clay at depths between 15-50 m. Figure 9 shows in more detail the locations of the piles in areas D, E, F, and G, the location of the surveying points and piezometers in the central part of the building pit.

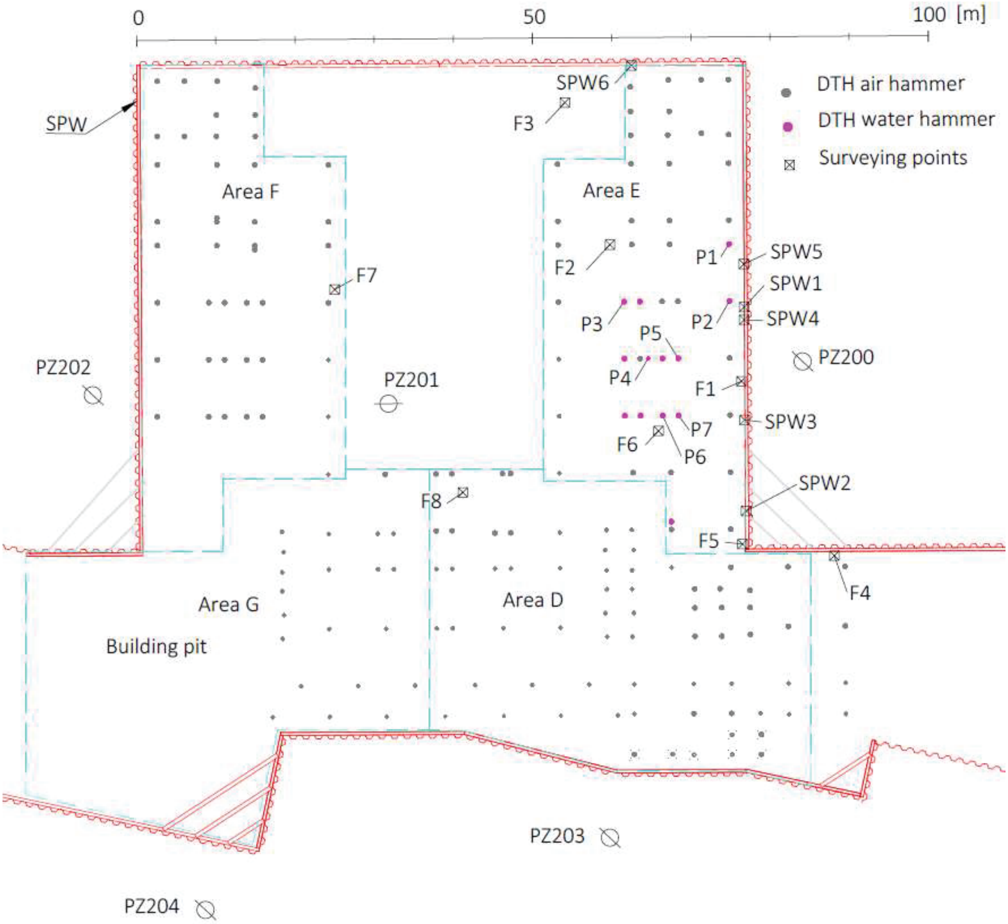


Figure 9. The central part of the building pit including the foundation piles, geodetic surveying points and piezometers.

3.6 Ground settlements

Figure 10 presents vertical displacements (δ_v) against time for the surveying points located in Area E. As can be seen from Figure 9 (a) the two initial surveying points on the SPW (SPW1 and SPW4) settled about 250 mm at the time the above-mentioned damage was observed on the SPW (2nd of Sept.) The settlements continued with varying rates and reached approximately 500 mm before all air hammer drilling stopped on the 30th of October. The varying settlement rates could be related to drilling activities in different areas. The supplementary point F1 on the concrete slab which was closest to the points SPW1 and SPW4 showed similar settlement rates, while the points F3 and F4 located further away settled less. An immediate reduction in the settlement rates was observed when the drilling stopped. Some ongoing settlements could be related to a delayed response in the clay deposit overlying the glacial till layer.

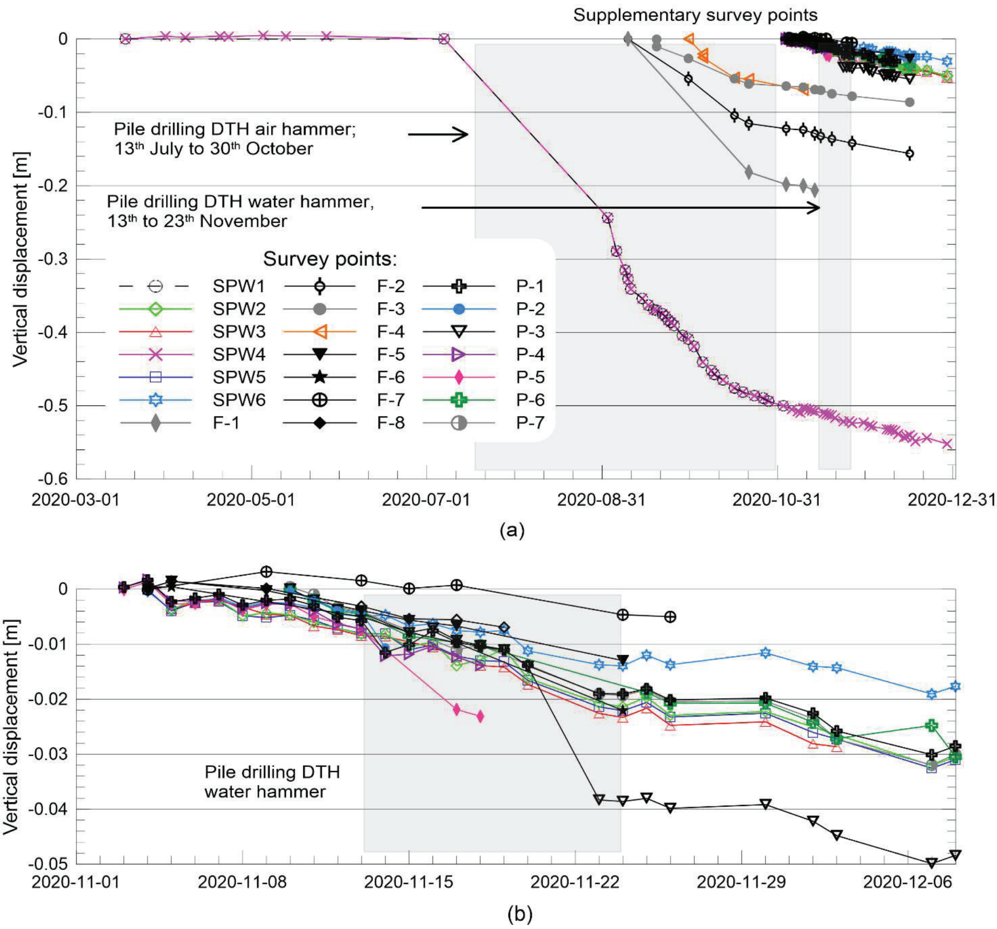


Figure 10. Vertical displacements against time on surveying points on top of the sheet pile wall (SPW), concrete slab (F) and 508 mm diameter outer casings (P).

The monitoring data and drilling protocols suggest that the substantial and immediate displacements were caused by two main effects: (1) erosion and loss of soil mass due to the air-lift pump effect when drilling in the till layer (as for Case 1), and (2) hydraulic ground failure into the bottom of the 406 mm casings. About 30% of all 406 mm casings drilled with the air hammer stopped in the till layer, thus requiring the telescope solution to reach bedrock. In many of these casings, hydraulic ground failure occurred at the bottom when the drill rod and pilot bit was pulled up, hence partly filling the casings up with fine-grained soil from the till before drilling of the smaller casings continued. Measurements showed a maximum of about 13 m of soil at the bottom of the casing (equals approximately 1.55 m^3 of soil). The local ground conditions with artesian pore-water pressures and ground water flow (i.e. recharge in the aquifer) combined with the low drilling penetration rate probably enhanced the erosion effect from the drilling, and caused problems with hydraulic uplift at the bottom of the casing.

After having observed the above-described negative effect of the drilling works, the drilling procedure was changed to reduce further impact. The updated procedure included to stop drilling with the 406 mm casings if the penetration rate in the till became lower than 4-5 cm/min. The casings were also filled to the top with water before removing the drill rod to reduce the risk of hydraulic failure. Despite these changes the displacements continued to develop.

Figure 10(b) presents the measured vertical displacements on the supplementary survey points. The points denoted P1 to P7 were located on top of the outer 508 mm diameter floating casings. The data indicates ongoing settlements between 4-8 mm over a period of ten days prior to the test drilling. Survey point F7 remained stable, likely due to the greater distance from the area most affected by the previous air hammer drilling. During the test drilling, the settlements continued at nearly the same rate as prior to the test. For point P3 and P5, immediate and significantly larger settlements were measured compared to the other points. This was caused by the drill rig accidentally running over and pushing the outer casing (Ø508 mm) into the soft clay. When the test drilling was completed, the settlements stabilized for about a week before they continued with a rate of 5-6 mm/week. The points F7 and SPW6, which were furthest from the test piles, showed the least settlements but followed the same long-term trend as the other points. Monitoring point SPW4, which covered the entire period of drilling with both air and water hammer, showed similar settlement rates as the supplementary points located in close vicinity to the SPW during and after the test drilling. Overall, the results clearly show a minor influence on the surrounding ground when drilling with the water hammer compared to when drilling with the air hammer.

3.7 Pore-water pressure response

Figure 11 presents pore-water pressures (U) at the top of the till layer against time. Air hammer drilling caused sudden and temporary pressure reductions in PZ201 and PZ202. A maximum change of about 15 kPa in PZ201 (29th of October) was registered when drilling more than 50 m south from the piezometer. PZ202 and PZ203 were not affected despite being located closer to the pile. This was explained by the piezometers being installed in clay just above the till. The pressure reductions support the hypothesis of air flushing causing an air-lift pump effect. Since the piezometers were installed just a few days before drilling with the air hammer stopped, the results do not provide a complete picture of the influence from air drilling.

Water hammer drilling increased the pore pressure by about 5 kPa in PZ200. This negligible effect may be related to the relatively large in-situ pore pressure and distance between the piezometer and the casings. PZ203 showed a gradual pore pressure reduction of about 15 kPa which was likely caused by a significant leakage of ground water observed through several of the open casings in area D of the building pit (Fig. 9).

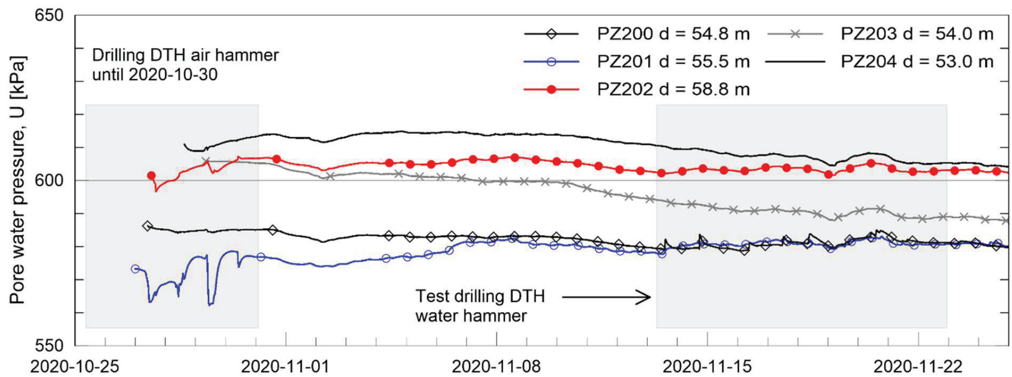


Figure 11. Measured pore-water pressures (U) against time in piezometers installed to the top of the till layer in the central part of the building pit. Depth (d) is related to the installation level.

3.8 Drill cutting volume balance

An important part of the test drilling scheme was to assess the volume balance of the generated drill cuttings (V_c) and the theoretical volume of the installed pile (V_p). This was determined by measuring the total volume of the backflow (V_{BF}), i.e. flushing water plus drill cuttings of each pile. For the backflow in clay and till, average densities (ρ) of respectively 1120 kg/m^3 and 1220 kg/m^3 were measured on representative samples (approx. 0.5 L) taken during drilling of every 12 m sections of the casings. The respective in-situ soil volumes were back-calculated from an average water content (w) of 35% and clay in-situ total unit weight of 1800 kg/m^3 based on findings from the ground investigations. An in-situ total unit weight of 2000 kg/m^3 was chosen for the till. Potential volumetric changes in the soil (compression, dilation) caused by the drilling were ignored.

Table 2 presents an overview of the volume balance calculations for eleven of the thirteen casings drilled with the water hammer. The volume balance for drilling in clay ($M_{c,c}$) and till ($M_{c,t}$) are given as the ratio between the volume of drill cuttings (V_c) and the installed pile casing volume (V_p). The results indicate that the drill cuttings generated through the clay represent in-situ soil volumes of 57 to 111% of the theoretical gross volume of the installed casings. This implies that the soil was partly displaced during drilling, which seems reasonable considering the relatively high penetration rates reported by the drilling contractor. Through the till deposit, the opposite was observed. In other words, the volume of the generated drill cuttings exceeded the pile volume (i.e. $M_{c,t}$ = 108-144%), suggesting a soil volume loss. It is possible that the soil displacement through the clay partly compensated for the volume loss through the till and to some extent contributed to the small ground settlements when using water drilling (Fig. 10(b)).

In addition to the volume calculations presented in Table 2, measurements of the total drill cuttings volume from the till layer were carried out for one of the test piles. This was done by collecting all the backflow generated during drilling in the till layer in a closed container. After a few days of sedimentation, the free water was pumped out and drained from the tank and the remaining soil volume was measured in a loose state. The results indicate that the total in-situ volume of soil from drill cuttings through the till represent approx. 100% of the gross volume of the casing. This assumes an in-situ soil density (ρ) of the till equal 2000 kg/m^3 and saturated density of the collected cuttings of 1500 kg/m^3, respectively. Despite uncertainties in the measured volume and assumed densities, the volume balance estimate indicates that water hammer drilling through the glacial till deposit did not cause notable excess drill cuttings. The differences compared to Table 2 could be related to uncertainties in estimating the density of the backflow which was limited by a small number of samples.

Table 2. Drill cuttings volume balance based on measured density and volume of drill cuttings (backflow) from test drilling with a water hammer.

Pile	OD casing [mm]	$L_{p,c}$ [m]	$L_{p,t}$ [m]	$V_{p,c}$ [L]	$V_{p,t}$ [L]	$V_{BF,c}$ [m^3]	$V_{BF,t}$ [m^3]	$V_{c,c}$ [L]	$V_{c,t}$ [L]	$M_{c,c}$ [-]	$M_{c,t}$ [-]
1	273	42.5	12.0	2488	702	13.5	8.5	2025	935	0.81	1.33
2	324	53.8	3.7	4433	305	19.3	3.7	2895	407	0.65	1.33
3	273	48.8	8.6	2857	503	12.0	6.0	1800	660	0.63	1.31
4	406	42.8	15.0	5552	1946	27.5	25.5	4125	2805	0.74	1.44
5	406	47.9	9.3	6213	1206	45.8	11.8	6870	1298	1.11	1.08
6	273	54.1	9.6	3167	562	14.7	5.7	2205	627	0.70	1.12
7	273	47.2	9.2	2763	539	11.2	6.1	1680	671	0.61	1.25
8	273	47.8	8.0	2798	468	10.7	4.9	1605	539	0.57	1.15
9	273	49.3	9.2	2886	539	12.9	6.5	1935	715	0.67	1.33
10	273	48.0	9.1	2810	533	11.5	6.1	1725	671	0.61	1.26
11	273	49.2	8.0	2880	468	12.2	5.2	1830	572	0.64	1.22

$L_{p,c}$ – Length of casing in clay

$L_{p,t}$ – Length of casing in till

$V_{p,c}$ – Volume of pile casing in clay

$V_{p,t}$ – Volume of pile casing in till

$V_{BF,c}$ – Total volume of backflow from drilling in clay

$V_{BF,t}$ – Total volume of backflow from drilling in till

$V_{c,c}$ – Volume of in-situ clay from backflow

$V_{c,t}$ – Volume of in-situ till from backflow

$M_{c,c}$ – Soil volume balance in clay given by $V_{c,c}/V_{p,c}$

$M_{c,t}$ – Soil volume balance in till given by $V_{c,t}/V_{p,t}$

4. Discussion

4.1 Effects on ground settlements

The two case studies show that drilling with the DTH air hammer caused significant ground settlements during or a short time after the drilling. The displacements registered on settlement anchor S5 in case 1 (Fig. 6) clearly imply that an excessive loss of soil mass occurred in the glacial till layer while drilling through this material. Approximately 75% of the final settlements (~60 mm) of S5 occurred directly after drilling of piles 01, 05, and 09 closest to the instrument. For the remaining piles at greater distance, S5 showed minor settlement. This indicates that settlement anchor S5 was "outside" a potential influence zone, which can be approximated by an inverted cone around each casing with an apex angle of 45 degrees, when drilling in the till.

The anchors in the clay (i.e. S1-S4) showed smaller settlements which can likely be related to both a delayed response and gradual decrease of the volume loss effect with closer distance to the soil surface. These lesser settlements are likely a result of soil arching effects (Terzaghi 1936).

Both case studies show that it was challenging to drill through the silty and sandy till as was also reported by Konstantakos et al. (2004), Küllingsjö (2007), and Sandene et al. (2021). However, the settlements in case 2 exceed case 1 and results reported in other previous studies. The large number of piles combined with a large depth to bedrock, a thick layer of glacial till, artesian pore pressures and considerable ground water recharge in the till, likely magnified the negative impacts when drilling with the air hammer in this case. Hydraulic ground failure occurred at the bottom of approximately 30% of the 406 mm diameter casings and contributed also to excessive soil loss and ground settlements.

The test drilling for case 2 showed a definite difference between air and water drilling. The possible benefits of using water flushing to reduce settlements in the surrounding ground became evident. While the air hammer struggled to drill through the till with high pore-water pressures and ground water recharge, average penetration rates between 25-137 cm/min were achieved with the water hammer. Hydraulic ground failure was not observed when water drilling since the casings were constantly filled with water. In addition, considerable drilling stops could be avoided in the till and the casing could be drilled at least 1 meter into bedrock. Asplind (2017), however, reported ground failure occurring in some casings installed by use of water hammer drilling through granular soils, but without discussing the reasons for it.

Measurements from both case studies show that the ground settlements continued after drilling was completed, but at a substantially reduced rate. The settlements of S4 and S5 in case 1 could likely be explained by a combination of the following effects: 1) Soil closing the presumed gaps/cavities created adjacent to the casings (illustrated in Fig. 1(b)). 2) Consolidation settlements in the clay deposit due to permanent pore pressure reduction (about 25 kPa) in the till (Langford and Baardvik 2016). 3) Re-consolidation of remolded clay along the casings, as described by Lande et al. (2020). However, due to limited data for S1-S3 this could not be verified. The same effects most likely explain the long-term settlements of case 2, where the large till thickness and the large number of piles magnified the influence. Since the construction area was filled up in the past, creep settlements may as well have contributed.

4.2 Effects on pore pressures

The sudden and temporary pore-water pressure reductions registered at the top of the till deposits (Figs. 5 and Fig. 11) document the impacts from air flushing and supports the hypothesis of excessive soil volume loss due to the air-lift pump and Venturi effects. The delayed re-establishment of pore pressures following sudden pressure reductions in the clay in case 1 could be explained by lower hydraulic conductivity compared to the till.

Figure 12 presents the maximum changes in pore pressure (ΔU_{max}) measured during drilling of single casings in cases 1 and 2 against normalized distance r/r_0 and radial distance r from the piezometers to the respective casings, where r_0 is the radius of the casings. Results from eight additional case records studied as part of the R&D projects "Limiting Damage I and II" (Langford et al. 2016) were included. Some of these projects measured rapid ground settlements during pile drilling as with cases 1 and 2. The data show a general trend of pore pressure reductions (i.e. negative values) for piezometers in till (open symbols) for air hammer drilling. The piezometers in clay (grey solid symbols) mainly show excess pressures due to the high drilling penetration rate causing soil displacements as observed by Lande et al. (2020). Results from water hammer drilling mainly show excess pore pressures (i.e. positive values) which implies that this method has less impact on the surrounding ground. The results shown in Figure 12 can guide practitioners when assessing negative impacts of drilling works.

Results from cases 1 and 2 indicate that the pore pressures in the till returned to almost the pre-drilling levels within a few hours (1-2 hours) after drilling (see Fig. 5 and Fig. 11). This response was caused by the ground water recharge in the till layer and the procedure of filling the casings with water after being drilled into bedrock. This behavior is in line with the studies by Lande et al. (2020) and Ahlund and Ögren (2016). The remaining, permanent pore pressure reductions in the till, which were observed for both cases (Fig. 5 and Fig. 11), are likely due to observed leakage of ground water through the top of some of the open casings before they were filled with cement mortar. This is comparable to drainage effects from bored piles within deep excavation pits (Langford and Baardvik 2016; Langford et al. 2015).

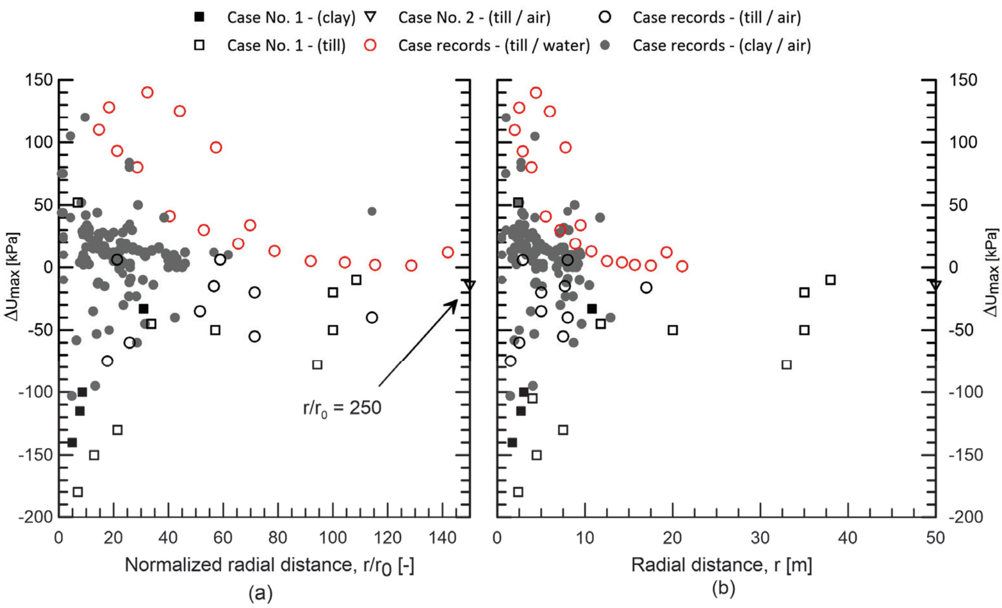


Figure 12. Maximum pore pressure changes (ΔU_{max}) against (a) normalized radial distance; and (b) metric radial distance between piezometer to piles.

4.3 Design framework to reduce impacts on the surroundings from overburden drilling in urban settings

As discussed in Section 3.8 the volume balance calculations required several assumptions. An alternative approach is to use a non-dimensional framework, as proposed by Lande et al. (2021), which compares the mass of drill cuttings with drilling parameters. In this framework the parameter normalized flow rate (Q_{norm}) is defined as:

$$Q_{norm} = \frac{Q}{A_{pile} \times V_{pen}} \quad (1)$$

where Q is the flushing flow rate in m^3/min, A_{pile} is the cross-sectional area of the pile in m^2, and V_{pen} is the penetration rate in m/min. The normalized mass of drill cuttings, $M_{c,norm}$, is identical to M_c (see Table 2, where $M_{c,c}$ and $M_{c,t}$ are for clay and till respectively), and represents the ratio of drill cuttings volume to the original volume. Values of $M_{c,norm}$ >1 indicates volume loss and $M_{c,norm}$ <1 soil displacement.

Figure 13 presents results from the soil volume balance calculations for the water drilling of case 2 using black solid symbols for drilling in till and open symbols for drilling in clay. The black line is a linear polynomial fitted to the till data. Results from small-scale model tests replicating overburden drilling in sand (Lande et al. 2021) are included for comparison.

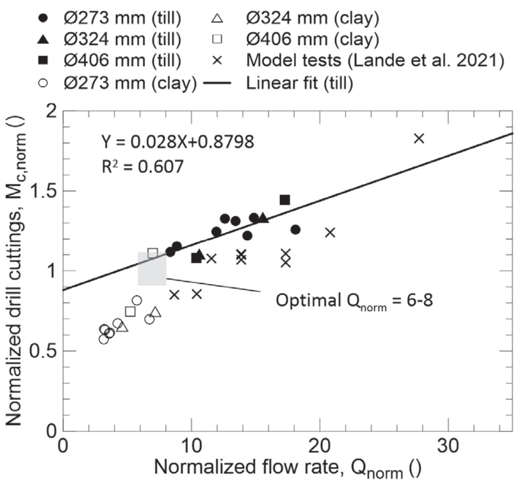


Figure 13. Design chart: normalized flow rate (Q_{norm}) against normalized mass of drill cuttings ($M_{c,norm}$) for test piles drilled with DTH water hammer in case 2.

The data indicate that the drill cuttings generally increase with the normalized flow rate. However, the normalized drill cuttings in till are consistently above the results from the model tests for the same normalized flow rate. This could be related to larger dimensions of the pilot drill bit (including channels for the backflow) for the full-scale drilling system compared to the model test. Differences in the ground conditions (soil density, grain sizes and pore pressure regime), soil stresses and water flushing pressures may also explain these deviations.

The results (Fig 13.) indicate that a normalized flushing flow rate Q_{norm} between 6-8 represents an optimal design value for drilling in till since this setup limits the amount of drill cuttings to just slightly larger than the gross volume of the casing. Consequently, careful logging of the drilling parameters Q and V_{pen}, preferably with sensors on the drill rig is beneficial.

Figure 14 presents a simple design framework which can be used to guide practitioners when selecting an overburden drilling method. This design framework focuses on reducing the risk of excessive ground settlements due to drilling which could be crucial when drilling in urban settings. The framework is meant for situations where vulnerable structures are located within an assumed influence zone of drilling which could be described by a 45-degree inverted cone from the point where the anchors or piles enter bedrock.

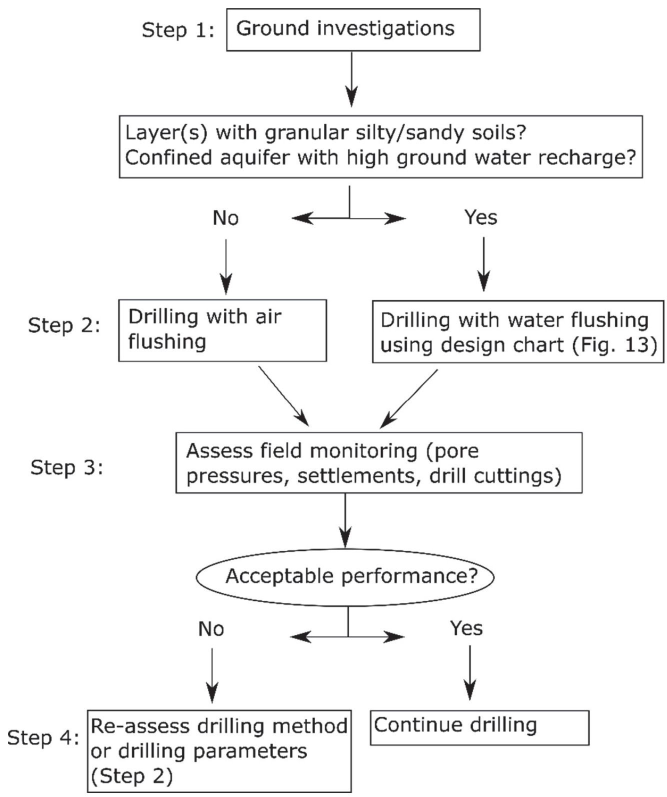


Figure 14. Flow chart illustrating design framework to choose drilling method in urban settings.

Step 1 of this design framework involves ground investigations to map potential layers of granular silty and sandy soils (e.g., glacial tills) susceptible for erosion during drilling. As a minimum information, grain size distributions and hydrogeological conditions (i.e., pore-water pressure, aquifer recharge properties) should be obtained. If such soils are encountered, drilling with water flushing is generally recommended (step 2) and optimal drilling parameters can be derived using the proposed design chart (Fig. 13). For such challenging ground conditions, it could be necessary to re-design and reduce the pile dimensions so that water hammers could be used (i.e., OD < 500 mm). If no layers with granular silty and sandy soils combined with confined aquifers with high ground water recharge are identified, then the less expensive air hammers may be used. Monitoring with piezometers and settlement anchors (extensometers) at different soil depths is suggested to assess the field performance of the drilling method (step 3). In addition, the amount of drill cuttings should be measured for test piles to be able to more accurately assess if excessive soil loss occurs. If the performance of the selected drilling method is not acceptable, the drilling method should be reassessed, or the drilling parameters (flow and penetration) adjusted (step 4).

5. Conclusions

This paper presents two cases of overburden drilling through marine clay, glacial till and into bedrock. Monitoring of pore-water pressures and ground settlements provides valuable documentation and deeper insights into the effects of drilling with DTH hammers on the surrounding ground. Based on the monitoring data and field observations, the following general trends can be drawn:

(1) Drilling with air-driven DTH hammers causes an air-lift pump effect at the front of the drill bit that may lead to significant local erosion and loss of soil mass. This effect seems to be particularly problematic when drilling in erodible soils like silt and fine sands. Drilling through confined aquifers with artesian pore-water pressures and a high recharge of ground water (as observed in case 2) further increase the risk of excessive erosion, volume loss and ground settlements.

(2) Rapid pore pressure reductions measured in till during drilling with DTH air hammers confirm the air-lift pump effect and can be used as an indicator for potential soil volume loss.

(3) Drilling with water-driven DTH hammers (case 2) considerably reduced soil volume loss compared to drilling with air hammers.

(4) The proposed design chart for water drilling (Fig. 13) can provide guidance to derive drilling parameters (i.e., flow and penetration rates) which limit impacts on surrounding ground. Field results indicate that a normalized flow rate (Q_{norm}) between 6-8 may result in ideal drilling in compact till.

The collected field data from case 2 validated results from model tests on how to select parameters for overburden drilling with water hammers. For situations where drilling shall be carried out adjacent to sensitive infrastructure, a rational framework is proposed to aid practitioners in the choice of drilling method and monitoring of the performance to reduce detrimental impact on the surroundings.

Data availability

Some or all data, models, or code generated or used during this study are available from the corresponding author by request.

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Competing interests

The authors declare that there are no competing interests.

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