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.**

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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).

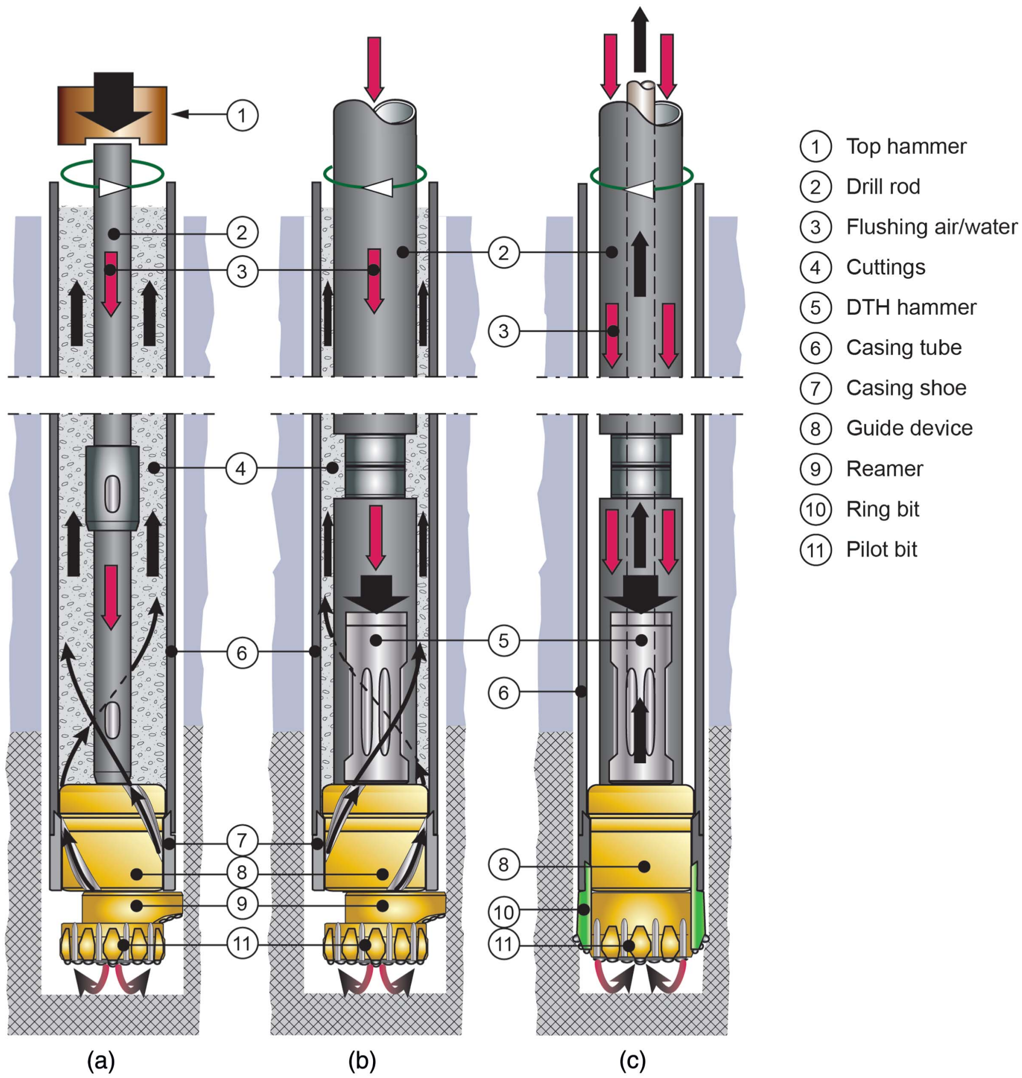

Fig. 1. Rotary percussive duplex drilling methods and drill bits used for overburden drilling: (a) top drive (top-hammer) eccentric; (b) DTH hammer eccentric; and (c) reverse circulation (RC) DTH hammer concentric.

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

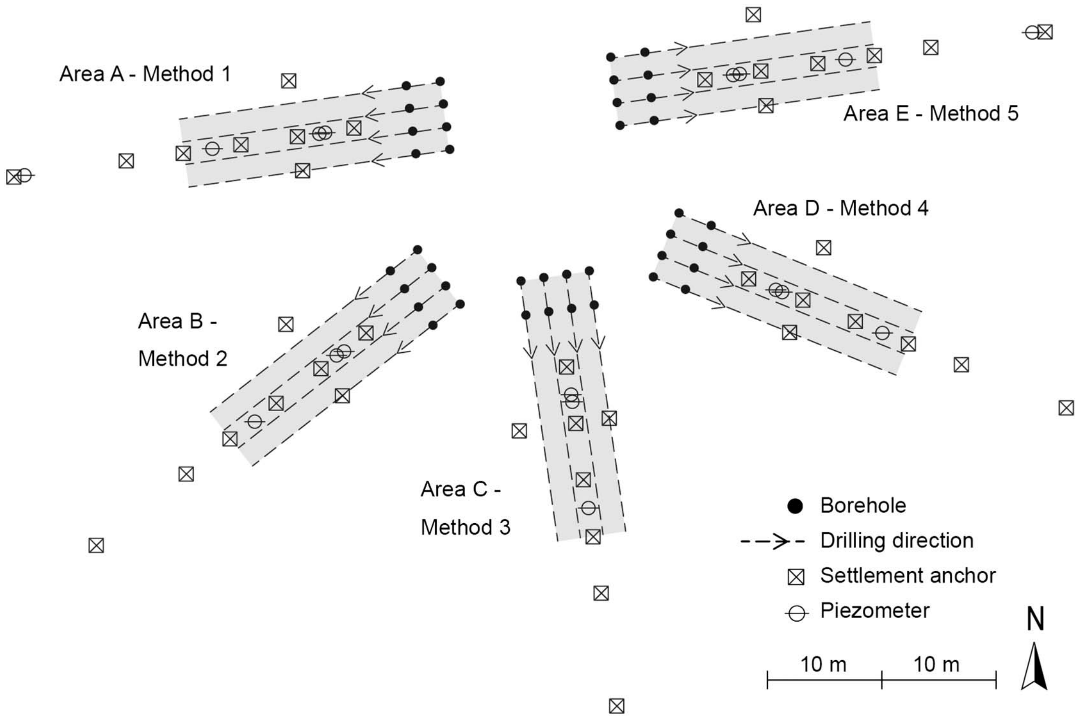

Fig. 2. Layout of test site with Areas A–E and related drilling methods.

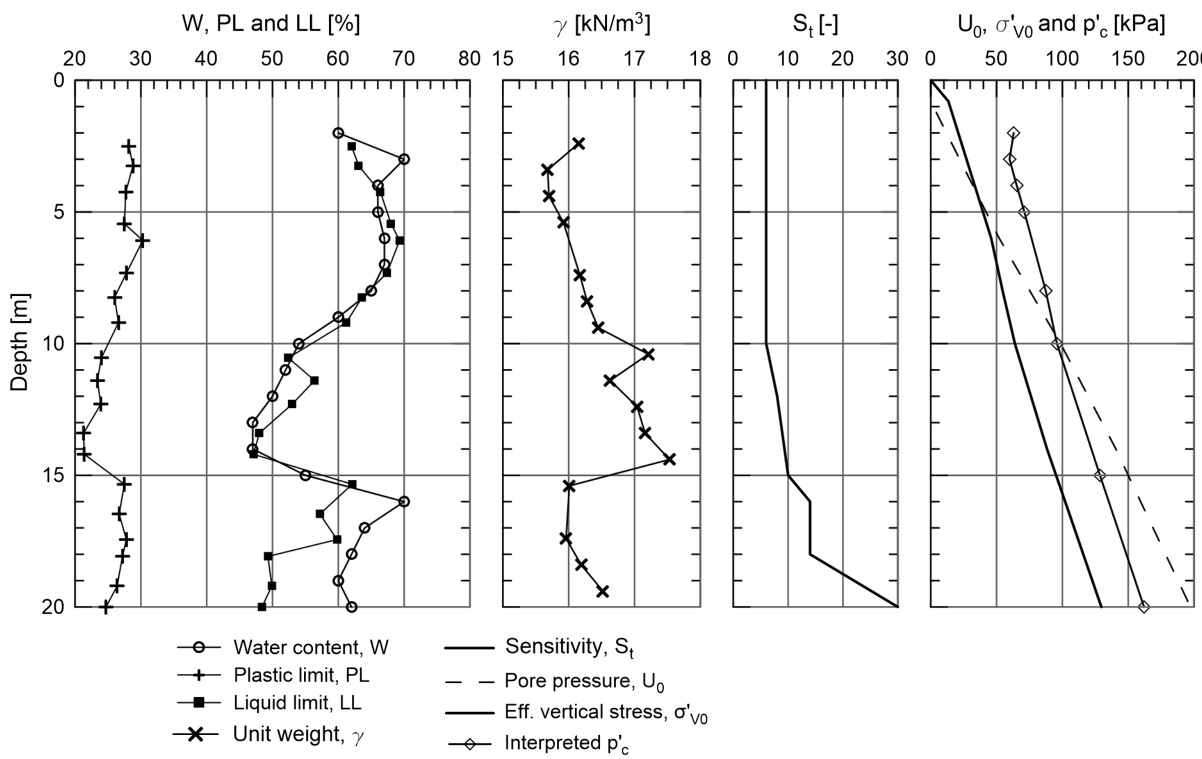

Fig. 3. Index data and interpreted in situ stress conditions for Onsøy clay from pile load test site. (Adapted from Karlsrud et al. 2014.)

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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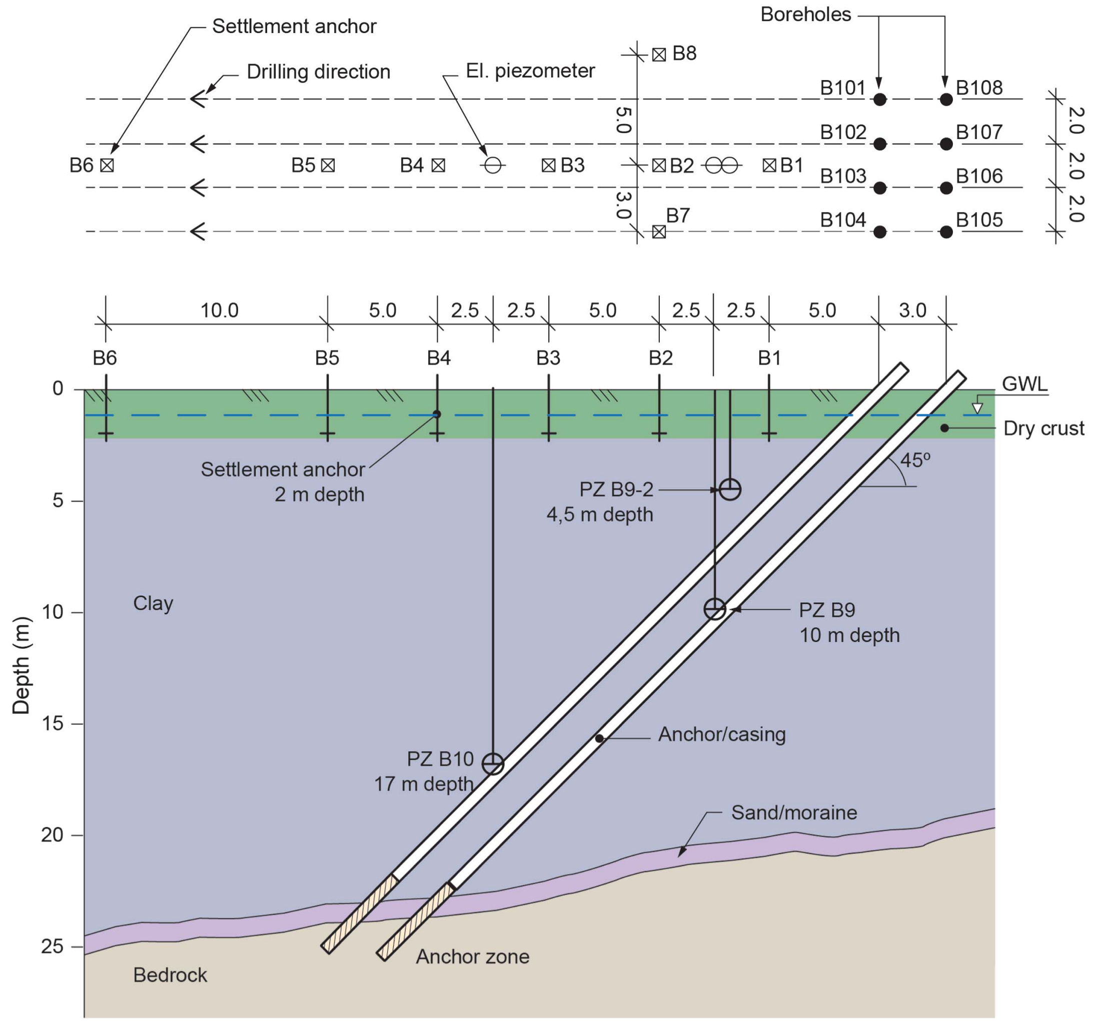

Fig. 4. Layout and cross section of instrumentation for each drilling method (example from Area B).

Table 1. Overview of drilling methods used in field test

Area	Drilling method	OD (mm) Casing (mm)	OD (mm) Reamer (mm)	Period of drilling
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.

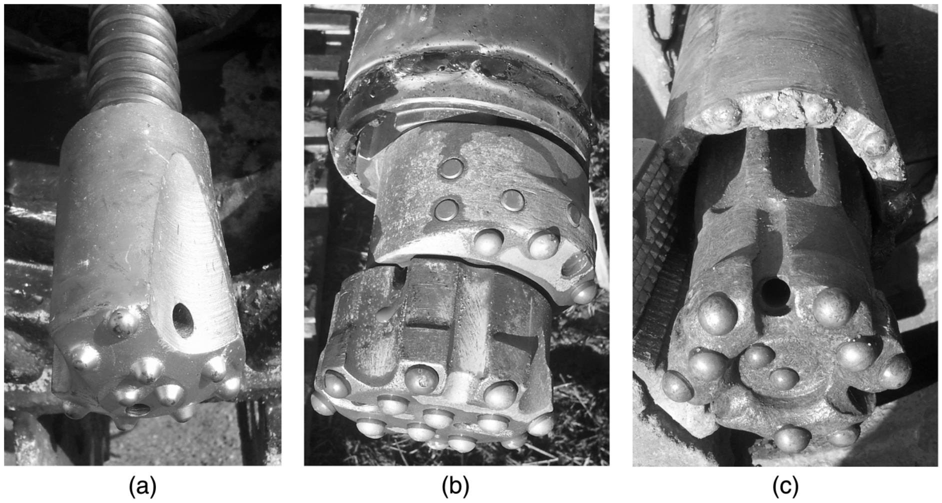

Fig. 5. Pictures of different drilling systems and drill bits used in field test. (Images by Einar John Lande.)

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

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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

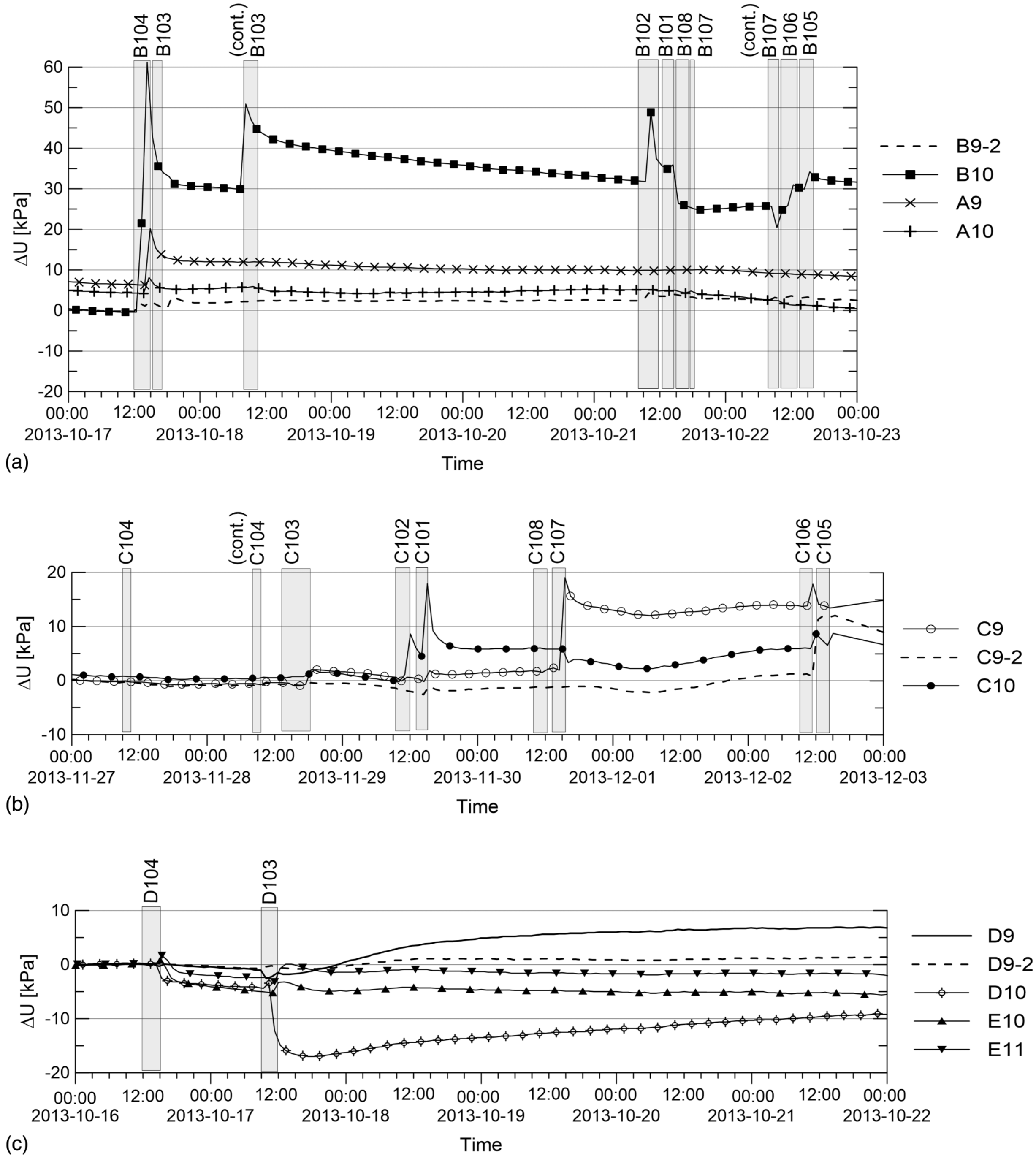

Fig. 6. Change in pore pressure (ΔU) with respect to time during drilling in (a) Area B; (b) Area C; and (c) Area D.

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 (ΔU_{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

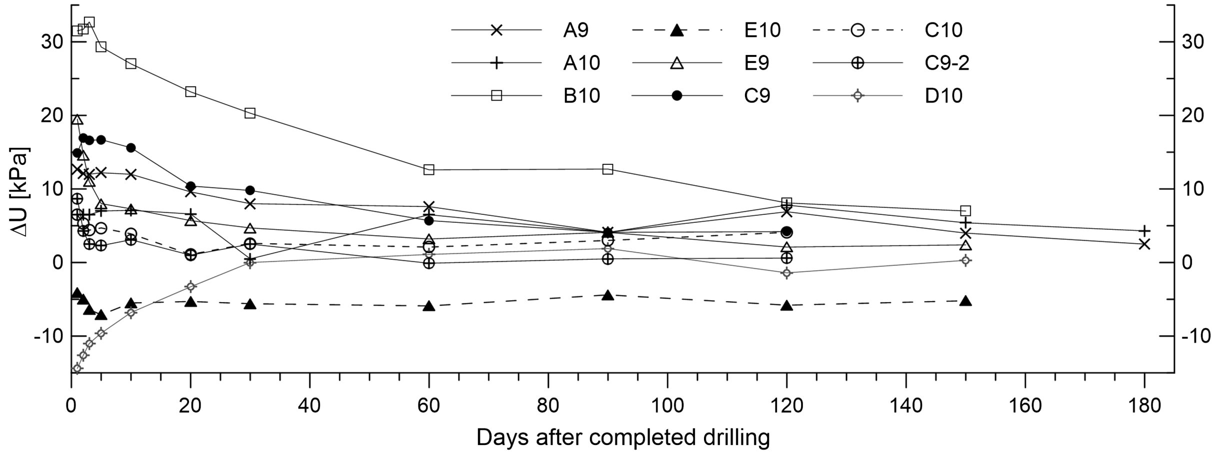

Fig. 7. Dissipation of excess pore pressure (ΔU) over time.

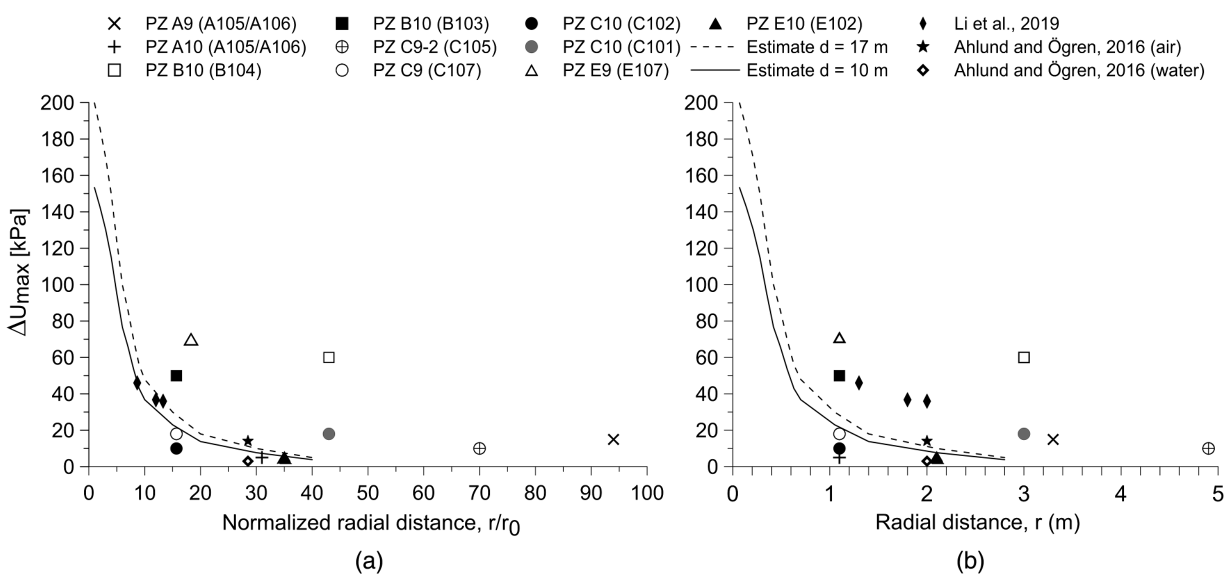

Fig. 8. Maximum change in pore pressure (ΔU_{max}) against (a) normalized radial distance; and (b) metric radial distance from anchors (casings).

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

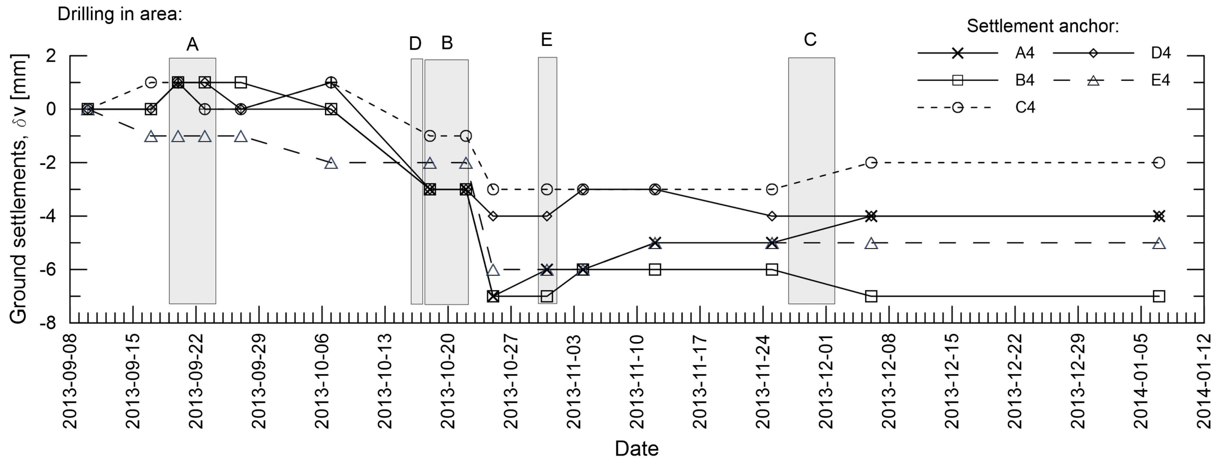

Fig. 9. Vertical ground settlements (δv) measured on Settlement Anchor 4 in Areas A–E, from September 9, 2013 to January 7, 2014.

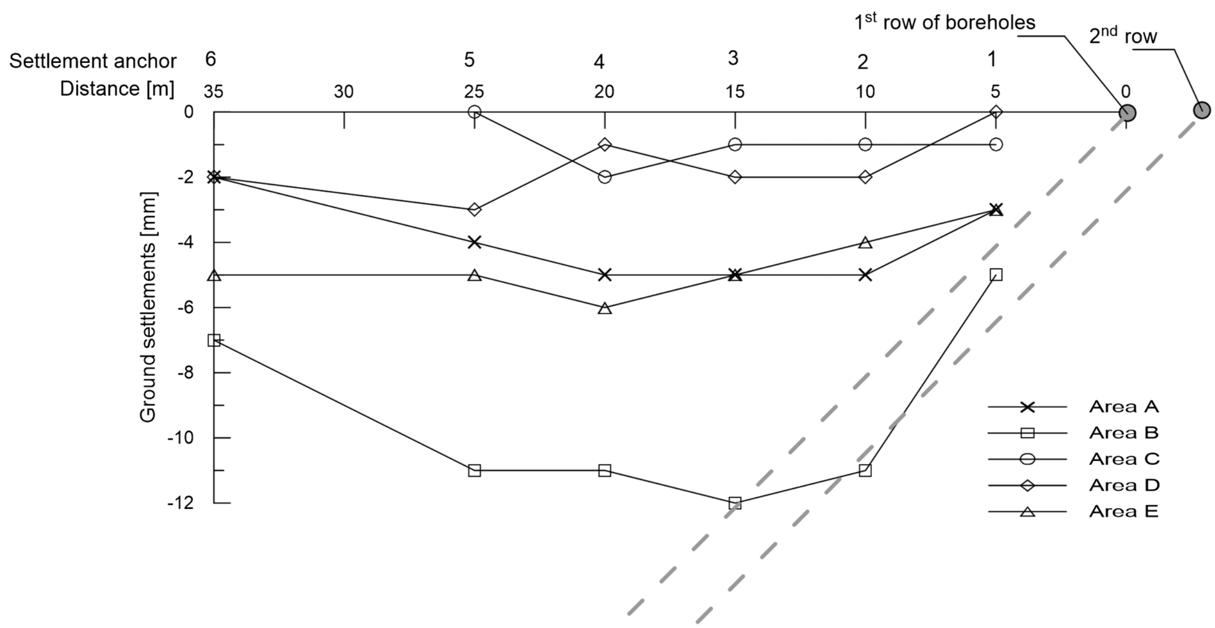

Fig. 10. Ground settlement profiles for Anchors 1–6, related to distance from first row of boreholes at ground surface. Data from final measurement on June 6, 2014.

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

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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.

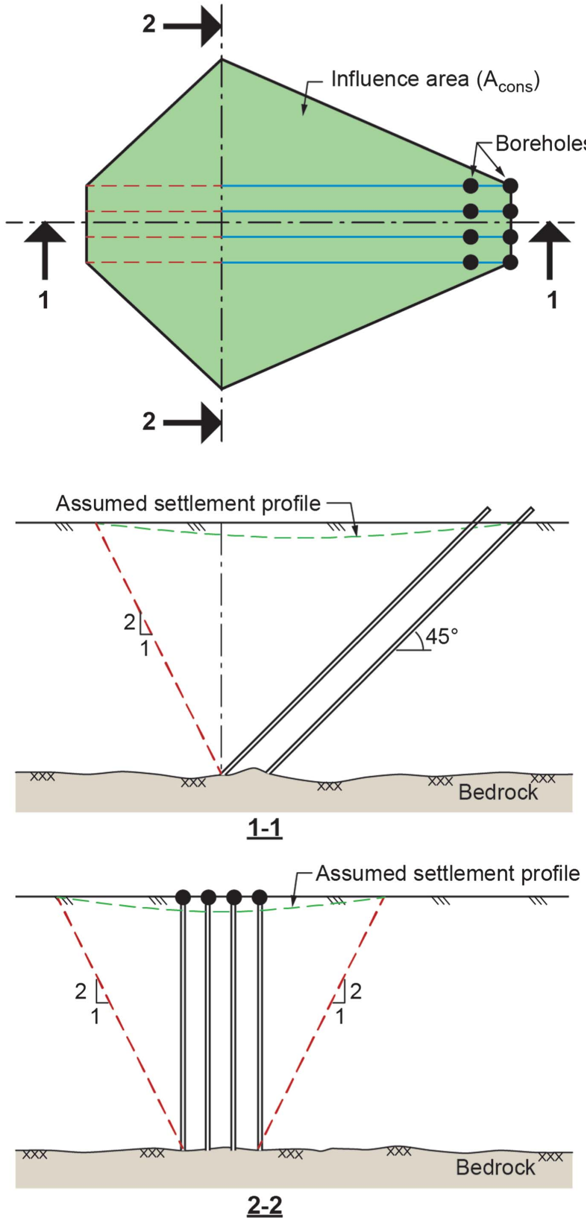

Fig. 11. Assumed influence area from reconsolidation of remolded clay around anchors.

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.

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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 $= p'_c/\sigma'_{v0}$;
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.

References

Ahlund, R., and O. Ögren. 2016. "Pore pressures and settlements generated from two different pile drilling methods." Master of Science thesis, Dept. of Civil and Architectural Engineering, Royal Institute of Technology.

Baardvik, G., A. Engen, B. Kalsnes, K. Karlsrud, E. J. Lande, J. Langford, A. Simonsen, G. Tvedt, and G. Veslegard. 2016. *Begrensnings av skader som følge av grunnarbeider, sluttrapport [Limitation of damage due to ground works, final report]*. [In Norwegian.] BIA-Project 219951/030, Final Rep. Oslo, Norway: Norwegian Research Council.

Baligh, M. M. 1985. "Strain path method." *J. Geotech. Eng.* 111 (9): 1108–1136. https://doi.org/10.1061/(ASCE)0733-9410(1985)111:9(1108).

Behringer, H. 1930. *Die Flüssigkeitsförderung nach dem Prinzip der Mammutpumpe [Pumping liquids according to the principle of the mammoth pump]*. [In German.] Doctoral dissertation, Technical Univ. of Karlsruhe.

Bredenberg, H., M. Jönsson, R. Isa, M. Larsson, and E. L. Larsson. 2014. *Borrteknik för minimering av marksättninger vid borrad grundläggning [Drilling technique for minimizing ground settlements from drilling of foundation piles]*. [In Swedish.] Tyresö, Stockholm: Bygg & Teknik.

Edstam, T., and A. Küllingsjö. 2010. "Ground displacements due to pile driving in Gothenburg clay." In *Proc., 7th European Conf. on Numerical Methods in Geotechnical Engineering (NUMGE), Trondheim, Norway*, 25–630. London: Taylor and Francis.

FHWA (Federal Highway Administration). 2005. *Micropile design and construction*. Publication No. FHWA NHI-05-039. Washington, DC: FHWA.

Finnish Road Authorities. 2003. *Instructions for drilled piling. Design and execution guide. Guidelines for design and implementation*. Helsinki, FL: Finnish Road Authorities.

Geokon. 2019. "Settlement points (Borros type) | Model 1950." Accessed March 14, 2019. http://www.geokon.com/1950.

Karlsrud, K. 2012. "Prediction of load displacement behavior and capacity of axially loaded piles in clay based on interpretation of load test results." Ph.D. thesis, Dept. of Civil and Environmental Engineering, Norwegian Univ. of Science and Technology.

Karlsrud, K., and L. Andresen. 2008. "Design and performance of deep excavations in soft clays." In *Proc., 6th Int. Conf. on Case Histories in Geotechnical Engineering*. Arlington, VA: Missouri Univ. of Science and Technology.

Karlsrud, K., and T. Haugen. 1984. . "Cyclic loading of piles and pile anchors-phase II: Final report." NGI Rep. No. 40018-11. Oslo, Norway: Dept. for Onshore Foundations, Norwegian Geotechnical Institute.

Karlsrud, K., T. G. Jensen, V. Gardå, E. K. W. Lied, and F. Nowacki. 2014. . "Time effects on pile capacity—Summary and evaluation of pile test results." NGI Rep. No. 20061251-00-279-R. Oslo, Norway: Dept. for Onshore Foundations, Norwegian Geotechnical Institute.

Kato, H., S. Tamiya, and T. Miyazawa. 1975. "A study of an air-lift pump for solid particles and its application to marine engineering." In *Proc., 2nd Symp. on Jet Pumps, Ejectors and Gas Lift Techniques*, G3-37–G3-49. Cambridge, UK: Churchill College.

Kempfert, H. G., and B. Gebresellassie. 1999. "Effect of anchor installation on settlement of nearby structures in soft soils." In *Proc., Int. Symp. on Geotechnical Aspects of Underground Construction in Soft Ground*, 665–670. Rotterdam, Netherlands: A.A. Balkema.

Konstantakos, D. C., A. J. Whittle, C. Regalado, and B. Scharner. 2004. "Control of ground movements for a multi-level-anchored, diaphragm wall during excavation." In *Proc., 5th Int. Conf. on Case Histories in Geotechnical Engineering*, 37. New York: Univ. of Missouri—Rolla.

Kullingsjø, A. 2007. "Effects of deep excavations in soft clay on immediate surroundings—Analysis of the possibility to predict deformations and reactions against the retaining system." Ph.D. thesis, Dept. of Civil and Architectural Engineering, Royal Institute of Technology.

Lande, E. J., and K. Karlsrud. 2015. "Full scale field test—Drilling of anchors to bedrock in soft clay." In *Proc., 16th European Conf. on Soil Mechanics and Geotechnical Engineering, Edinburgh*, 625–630. London: ICE Publishing. https://doi.org/10.1680/ecsmge.60678.

Langford, J., and G. Baardvik. 2016. "Pore pressure reductions and settlements induced by deep supported excavations in soft clay." In *Proc., 17th Nordic Geotechnical Meeting, Reykjavik*, 993–1002. Reykjavik, Iceland: Icelandic Geotechnical Society.

Langford, J., K. Karlsrud, E. J. Lande, A. Ø. Eknes, and A. Engen. 2015. "Causes of unexpectedly large settlements due to deep excavations in clay." In *Proc., 16th European Conf. on Soil Mechanics and Geotechnical Engineering, Edinburg*, 1115–1120. London: ICE Publishing. https://doi.org/10.1680/ecsmge.60678.vol3.156.

Lehane, B. M., and R. J. Jardine. 1994. “Displacement-pile behavior in a soft marine clay” *Can. Geotech. J.* 31 (2): 181–191. https://doi.org/10.1139/t94-024.

Li, G., A. C. Amenuvor, Y. Hou, X. Lu, J. Wu, and T. N. Nguyen. 2019. “Effect of open-ended phc pile installation during embankment widening on the surrounding soil.” *J. Geotech. Geoenviron. Eng.* 145 (2): 05018006. https://doi.org/10.1061/(ASCE)GT.1943-5606.0002016.

Mana, A. I., and G. W. Clough. 1981. “Prediction of movements for braced cuts in clays.” *J. Geotech. Geoenviron. Eng.* 107 (6): 759–777.

Ni, Q., C. C. Hird, and I. Guymer. 2009. “Physical modelling of pile penetration in clay using transparent particle image velocimetry.” *Géotechnique* 60 (2): 121–132. https://doi.org/10.1680/geot.8.P.052.

Peck, R. B. 1969. “Deep excavations and tunneling in soft ground.” In *Proc., 7th Int. Conf. on Soil Mechanics and Foundation Engineering*, 225–290. London: International Society for Soil Mechanics and Geotechnical Engineering.

Randolph, M. F., and C. P. Wroth. 1979. “An analytical solution for the consolidation around a driven pile.” *Int. J. Num. Anal. Methods Geomech.* 3 (3): 217–229. https://doi.org/10.1002/nag.1610030302.

Rønning, S. 2011. “E6 Trondheim-Stjørdal, Parsell Trondheim—Dagsone vest. Geotekniske aspekter knyttet til rørspunt, noen erfaringer ved utførelsen” [E6 Trondheim-Stjørdal, section Trondheim West. Geotechnical aspects related to the drilled tube wall, some experiences with the execution]. [In Norwegian.] In *Proc., 49th Geoteknikkdagen*. Oslo, Norway: Norwegian Geotechnical Society.

Veslegard, G., E. J. Lande, and A. S. Simonsen. 2015. *Forbedring og videreutvikling borede stag og peler—Metoder, utførelse og dokumentasjon [Improvements and developments bored anchors and piles—Methods, execution and documentation]*. [In Norwegian]. BIA-Project 219951/030, Rep. No. 3.4. Oslo, Norway: Norwegian Research Council.