Field measurements of pore water pressure changes in very high plasticity stiff clays adjacent to driven piles

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ABSTRACT
The pore pressure distribution surrounding driven piles in heavily overconsolidated high plasticity stiff clays is a complex matter and affected by a range of factors such as soil permeability, soil strength, distance from piles and number of piles. In this paper pore pressures measured in Søvind Marl before, during and after driving of numerous HE300B steel piles at a construction site are presented and discussed. The complexity in pore pressure distribution during pile driving and subsequent dissipation is highlighted from the results and the fully grouted method for installation of piezometers is discussed.

Keywords: Pore water pressure, Field tests, Pile driving, High plasticity stiff clays, Fully grouted method

1 INTRODUCTION

When a pile penetrates into a clay deposit it will cause very large shear strains in the surrounding clay, and as a consequence induce large changes in the total stress and pore pressure regime (Karlsrud, 2012).

Driving of displacement piles into a cohesive soil generate large excess pore pressures close to the pile (Figure 1) and at the pile-soil interface – a phenomenon that have been thoroughly described in the literature. The excess pore pressures may be positive or negative dependent on soil type. In the past both field and model studies have been performed but relatively few accurate field data are available on the distribution of excess pore water pressures between driven piles. Especially for piles in highly overconsolidated clay and so far no data have been published on the distribution of pore water pressure near driven piles in highly overconsolidated Danish clay of Palaeogene origin. From the literature, it is clear that no known numerical model can predict the induced excess pore pressure regime and subsequent dissipation in highly overconsolidated clays in an operational way. Hence, additional field measurements are required for different soil types to be able to correlate a given model with full scale field conditions. Figure 1 shows a principle sketch of the influence of pile driving on the movement and pore pressure changes in the adjacent soil.

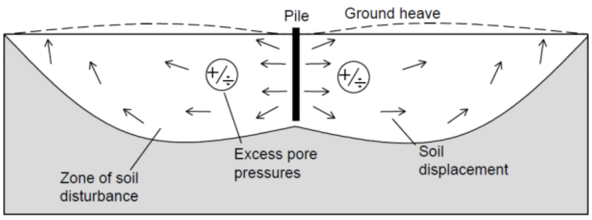

Figure 1. Influence of pile driving on the movement and pore pressure changes in the adjacent soil. Principle sketch.

In this paper, a field study of pore water pressure measurements using fully grouted vibrating wire piezometers adjacent to clusters of driven steel piles are presented. The piles are driven through made ground into a highly overconsolidated high plasticity Palaeogene clay – the so-called Søvind Marl.

Rarely have pore water pressure been measured under realistic conditions at actual construction sites during pile driving in highly overconsolidated clays. The aim of the study is to present and highlight the complex pore pressure conditions adjacent to a cluster of HE300B steel piles at a construction site. Focus are on the pore pressure profile before, during and up to 200 days after driving in highly overconsolidated high plasticity clay.

2 PREVIOUS STUDIES

Prediction of pore pressures in relation to driven piles is not a straightforward matter. As shall be seen in the following section, the pore pressure regime around a driven pile can vary a lot and is strongly dependent on factors such as soil type, distance from the pile, stress history, degree of fissuring, time, permeability and other factors. In fact excess pore pressures measured at any position along the shaft of a pile (or a CPT cone) can be positive or negative immediately after installation (Burns and Mayne, 1999).

2.1 Pore water pressure measurements in relation to pile driving

Previous studies show that the excess pore pressure build up around a driven pile in clay soil is very much dependent on the soil type. The excess pore pressure response is quite different for soft clays and normal consolidated (NC) clays compared to highly over consolidated (OC) clays. The majority of studies have focused on NC and slightly OC clays whereas only few studies have been performed on heavily OC clays.

Poulus and Davis (1980) have presented a summary of measured excess pore pressure developed due to pile driving from 11 case histories. Their findings are shown in Figure 2 which shows that in the vicinity of the pile, very high excess pore pressures build up. The figure also shows that beyond a radial distance of 4 – 8 pile radii there is a rapid decrease in pore pressure and that beyond 30 pile radii, the excess pore pressure can be considered negligible for a single driven pile.

Randolph and Wroth (1979) conclude that attempts to measure the pore pressure distribution in the vicinity of a driven pile usually produce scattered and inconclusive results. However, for NC clays the excess pore pressure appears to decrease approximately linearly with the logarithm of the radius from the pile axis.

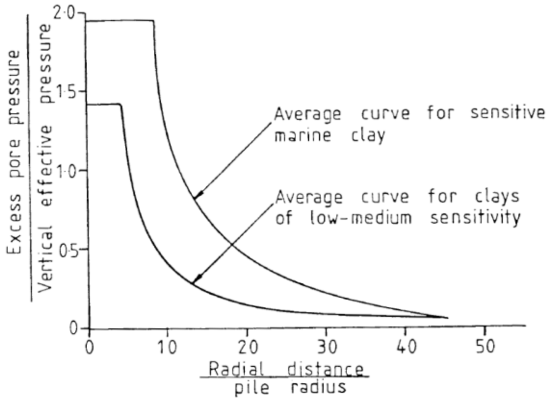

Figure 2. Typical measured excess pore pressure developed due to pile driving. Data from 11 case histories. Gue (1984) based on data from Poulus and Davis (1980)

Through numerical modelling Bergset (2015) demonstrated this trend for OC clays as well. He also showed that for OC clays negative excess pore pressures were likely to appear close to the pile due to dilation in the clay when sheared in contrast to the typical behavior.

Bond and Jardine (1991) and Coop and Wroth (1989) did field experiments using instrumented close-ended steel piles with a length of up to 6 m in heavily OC London Clay confirm this. They conclude that their results differ in several important respects from the response predicted by current theories of pile behaviour and they found among other things that negative excess pore pressures are generated at the pile surface during installation.

From the literature it has been found that pile driving can influence the pore pressures in a zone of up to approx. 100 pile radii from a driven pile dependent on the soil type. In NC clays the distance is approx. 14 – 35 pile radii (Chandra et al., 1993 and Robertson et al., 1990) while it is 30 – 90 pile radii for slightly OC clays (Lo et al., 1965 and Eigenbrod et al., 1996). For a sensitive marine clay, the influence of pile driving on the pore pressures were seen to be negligible beyond a distance of 100 pile radii from a pile group (Bozozuk et al., 1978).

The large variation in the extent of the influence zone (14 to 100 pile radii) corresponds to a distance between 2 and 15 meters for a 300 mm x 300 mm precast concrete pile (which are often used at Danish construction sites). The lateral distance from the pile within which excess pore pressures are build up depends on the soil type as a likely result of differences in permeability, strength, stiffness, stress state (e.g. K_0) etc. However, no distinct pattern is found even when comparing similar soil types. Consequently, it is difficult to predict the lateral distribution of excess pore pressures around driven piles and especially in highly OC Palaeogene clays, as the available field data from this specific soil type is very sparse.

3 SITE CHARACTERIZATION

Field measurements were conducted at a construction site at the harbour of Aarhus in the central part of Denmark. This earlier industrial harbour area at the city centre, comprising more than 400000 m^2 of area, is currently undergoing a major urban re-development. Common for all building projects at Aarhus harbour are, that soil conditions necessitate deep foundations, which in Denmark normally mean pile foundations with driven piles. This field study is based on data obtained at the construction site for the building project referred to as Z-huset. A building of up to 10 storeys with a 2-storey underground parking facility is to be constructed at the site.

3.1 Previous construction activities

The construction of Z-huset, was commenced in 2008 with an approx. 7 m deep excavation (locally up to 8.5 m) followed by driving of hundreds of precast quadratic concrete piles. Due to the financial crisis, the construction ceased in 2009 after completion of only the 2-storey concrete basement structure.

3.2 Current construction activities

In 2014 the construction of the building recommenced and at that time the project had undergone significant design modifications. The modifications meant a redistribution of loads at the foundation level, which called for additional approx. 400 piles to be driven. The basement structure was demolished, apart from the outer basement walls and the pile-founded concrete bottom plate (500 mm thick), so the additional piles had to be driven through cut open sections of the bottom plate. For a principle sketch of the construction see Figure 3. Steel piles (HE300B) were chosen in favour of concrete piles due to a smaller cross-sectional area. The purpose was to minimize damage of the existing pile founded bottom plate due to soil displacement and heave from driving the new piles. Nevertheless, pile driving resulted in heave of the concrete bottom plate between zero and 27 mm (based on information from the contractor).

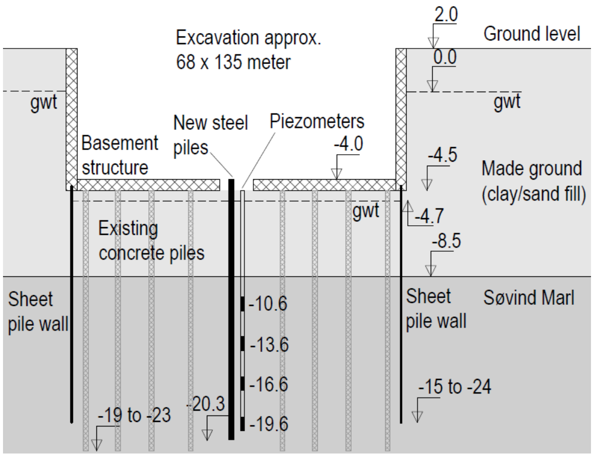

Figure 3. Principle sketch of the basement structure and piezometer installation. (gwt = groundwater table).

3.3 Ground and groundwater conditions

At this site the ground conditions are quite uniform. The site area was reclaimed 25 years ago for which reason the ground consist of approx. 8 – 10 meter of made ground (primarily sand and clay fill) and directly hereunder Søvind Marl to approx. 70 meters below sea level over similar high plasticity clays to several hundred meters depth.

The groundwater level is under influence from the water level in the adjacent sea and is normally found in level 0 ± 0.5 m. After construction of the basement, drains below the bottom plate in approx. level -4.7 govern the groundwater level.

Geotechnical investigations including several geotechnical borings and cone penetration tests (CPT) were carried out in 2007 and earlier. Previous construction activities have resulted in an assumed non-hydrostatic pore pressure profile at the site.

3.4 Geotechnical characterization of Søvind Marl

The Søvind Marl is a sedimentary marine calcareous clay of extreme plasticity deposited in an Eocene ocean covering the Danish area from around 45 to 35 million years ago. The clay is highly OC due to previous glaciations and geological erosion events. Søvind Marl is a fissured clay with slickensides and clay can be assumed to be fully saturated. Typical index properties and geotechnical parameters of the clay are given in table 2.

Table 1. Index properties and geotechnical parameters of Søvind Marl (From the archive of Geo and previous site investigations at Z-huset)

Property	Value/Range
Natural water content	35-55 [%]
Liquid Limit	80-350 [%]
Plasticity Index	50-250 [%]
Unit weight	17-19 [kN/m^3]
$CaCO_3$	0-75 [%]
Undrained shear strength	80 – 170 [kPa]
Coefficient of permeability (k_{lab})	$3{\cdot}10^{-12}$ – $5{\cdot}10^{-13}$ [m/s]
Oedometer modulus (E_{oed}) between level -10 and -30	~85 – 340 [MPa]
Coefficient of consolidation (c_v)	$2.9{\cdot}10^{-7}$ – $3.6{\cdot}10^{-9}$ [m^2/s]

4 RECENT INVESTIGATIONS

4.1 Field instrumentation and installation process

Installation of the piezometers (Geosense multilevel vibrating wire piezometers (VWP), 690 kPa HAE) took place on the 24th of January 2015. A string with piezometers in four levels was installed in a borehole that was subsequently filled with a cement-bentonite grout (Figure 3). This method, termed the fully grouted method, has been described by e.g. Vaughan (1969), McKenna (1995), Mikkelsen and Green (2003) and Simeoni et al. (2011) and its applicability in heavily OC London Clay has been illustrated by Wan and Standing (2014). The equipment was installed according to the principles described by Mikkelsen and Green (2003). Due to a fault in delivery of the bentonite, a pre-mixed cement/bentonite mix was used instead of pure bentonite. This led to a grout with a very low bentonite content almost comparable with a neat cement grout. Grout details are given in Table 2. The mistake was not realised until after the installation was completed.

Table 2. Grout properties (w: water, c: cement, b: bentonite)

Mix proportion by mass (w: c: b)	Marsh Funnel viscosity [sec.]	Permeability (curing time 37 days) [m/s]
2: 1: 0.014	27	$9.8{\cdot}10^{-10}$

4.2 Grout permeability

Previous studies have shown that the grout permeability is crucial for successful measurements when using fully grouted piezometers. (Vaughan 1969, McKenna 1995, Mikkelsen and Green 2003, Contreras et al. 2008). When choosing a grout with an appropriate permeability the horizontal hydraulic gradients adjacent to the piezometer will be much higher than the vertical gradients inside the grouted borehole. Therefore, the piezometer will read the correct formation pore pressure (Mikkelsen and Green, 2003). Different studies show that the grout permeability can be some orders of magnitude greater than the permeability of the surrounding soil without introducing significant errors Vaughan 1969, Contreras et al. 2008, Marefat et al., 2014).

Recently constant head and falling head permeability tests have been performed for 5 different grout mixtures at different curing stages in connection to this study. They indicated a very low permeability for a mixture with a very low bentonite content. The grout accidently used in this study had one of the lowest permeabilities measured. At the same time however, it showed significant shrinkage during curing. McKenna (1995) describes that the shrinkage of neat cement grouts during hydration may potentially result in the grout pulling away from the borehole. Based on the field measurements it is later discussed if the very low bentonite content in the applied grout is sufficient to meet the requirements for the relevant purpose.

4.3 Permeability of Søvind Marl

The permeability of Søvind Marl is, similar to all other Palaeogene clays, very low and the permeability of such clays are normally derived indirectly from oedometer tests. Oedometer tests in relation to previous investigations in the area have yielded a coefficient of permeability (k_{lab}) for Søvind Marl around $3 \cdot 10^{-12} - 5 \cdot 10^{-13}$ m/s.

However, Thorsen (2015) did a numerical (2D Plaxis) comparison of k_{lab} with settlements observations for a large structure over a period of years and concluded that the insitu permeability for a Palaeogene clay may be up to 30 times larger than k_{lab} derived by indirect methods.

In order to get an estimate of the coefficient of permeability without having to derive it from an oedometer test, a constant head permeability test was performed on a sample of Søvind Marl that was built into a triaxial cell. The soil sample was taken during drilling for the installation of the piezometers at level -10.6. The permeability test yielded a coefficient of permeability $k \approx 4 \cdot 10^{-11}$ m/s which is between 13 and 80 times higher than the previous indirect results from oedometer tests. The findings by Thorsen (2015) may indicate that the coefficient of permeability derived from constant head test in the lab may give a better representation of field permeability. More tests should be performed to confirm this.

5 PILE DRIVING

The 15.6 m long HE300B steel piles were driven by a Junttan PM20 pile driver with a 60 kN hammer. The tip of the piles after driving were approx. at level -20,3 i.e. approx. 0.7 meter deeper than the lowest piezometer. Driving started on the 19th of January 2015 and was completed 49 days later. In total approx. 400 steel piles were driven during that period. The sequence and number of piles driven each day after the start of pile driving is shown in Figure 4 together with existing piles. Note that the sketch is only a minor section of the construction site as only the piles within the zone around the piezometer where pore pressure response were registered during pile driving are shown.

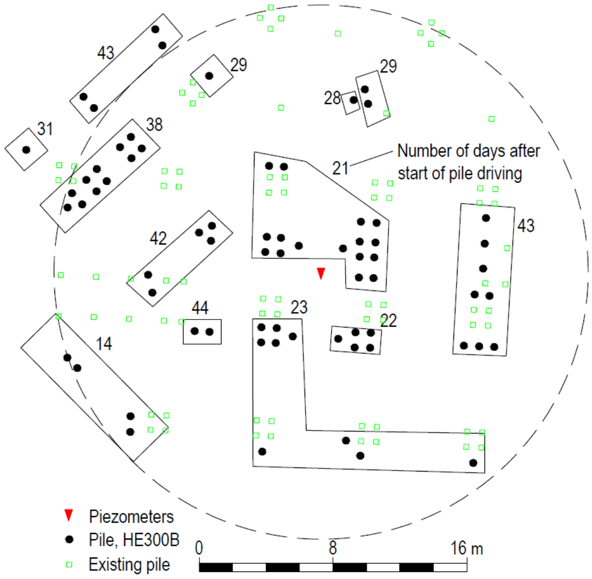

Figure 4. Location of new and existing piles and the piezometers. Numbers show the sequence of pile driving. The dashed circle indicates 100 pile radii from the piezometers. The piezometers were installed on day 5. (*equivalent radius = 160 mm for a HE300B steel pile assuming a plug at the pile tip)*

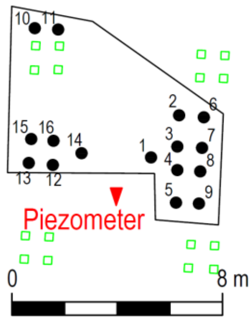

Figure 5. Piles driven at day 21. Numbers refer to the sequence of driving.

6 PORE PRESSURE MEASUREMENTS

6.1 Expected pore pressure profile before recent pile driving

In order to evaluate the pore pressures measured right after installation of the piezometers an estimation of the present pore pressure regime should be made for critically assessment of the measured data.

However, the pore pressure profile in the Søvind Marl at Z-huset has proven very difficult to predict due to several activities at the site over the years. Activities that would entail changes in pore pressure. Thus the area was reclaimed and filled up 25 years ago inducing excess pore pressures in the clay and subsequent recompression. In 2008 the excavation for the basement was commenced inducing negative changes in the pore pressures leading to swelling in the upper zone. Subsequently hundreds of concrete piles were driven into the Søvind Marl leading to further changes in pore pressures (positive or negative).

An estimate of the pore pressures after these loading/unloading events can be derived from Terzaghi's theory of consolidation assuming one-sided drainage (Okkels and Bødker, 2008). A calculation has been conducted under the assumptions that pore pressures were hydrostatic before the loading 25 years ago and with a coefficient of permeability k ~ $9 \cdot 10^{-11}$ m/s for Søvind Marl. The calculated deviation in pore pressures from hydrostatic conditions after the loading and unloading events (without taking the pile driving in 2008 into consideration) is in the order of -15 to -50 kPa from level -10.6 to -19.6. With present analytical models it is not possible to make a more accurate estimation of the pore pressure distribution with depth before the recent pile driving. Uncertainties mainly related to the permeability of Søvind Marl and insufficient knowledge about the influence from driving the concrete piles. However, owing to the mentioned activities the pore pressure regime prior to driving of the new piles is assumed non-hydrostatic.

6.2 Pore pressures measured before driving steel piles

The measurements of pore pressure started immediately after backfilling of the borehole with grout and connection of the piezometer string to a data logger. Figure 6 show the changes in pore pressure profile with depth after installation of the piezometers. The excess pore pressure measured at all levels right after installation is approx. 25 % higher than hydrostatic pressure due to the weight of the liquid grout. As the grout hardens, the piezometers show more or less hydrostatic conditions which is in contrast to the expected non-hydrostatic conditions. After the initial drop in pore pressure during curing a slight increase in pore pressure is observed between 21 and 36 hours after installation. This could indicate a minor effect of stress relief in the borehole or be due to ongoing curing of the grout. A similar effect, although considerably more pronounced and prolonged, have been seen for pore pressure measurements in London Clay (Wan and Standing, 2014). The authors also showed that the initial pore pressure profile was non-hydrostatic with locally induced negative excess pore pressures. The latter has not been observed in this study.

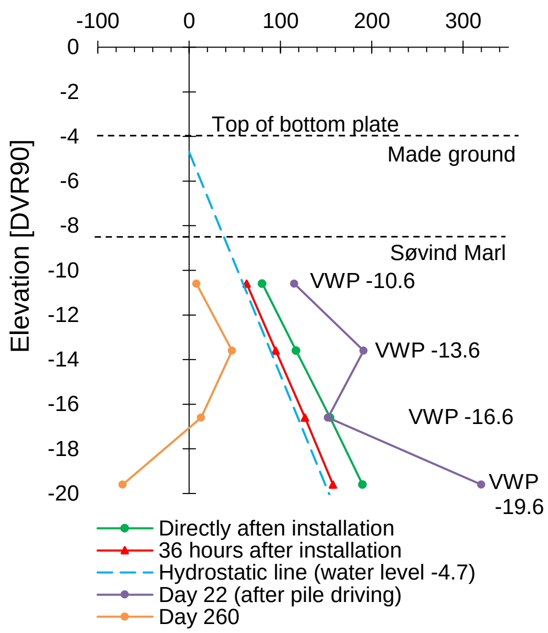

Figure 6. Pore pressure profiles.

6.3 Pore pressures measured during pile driving

The first piles to yield measurable changes in pore pressures were driven at day 14. On this day 4 piles were driven at a distance of 100 pile radii from the piezometers (Figure 4).

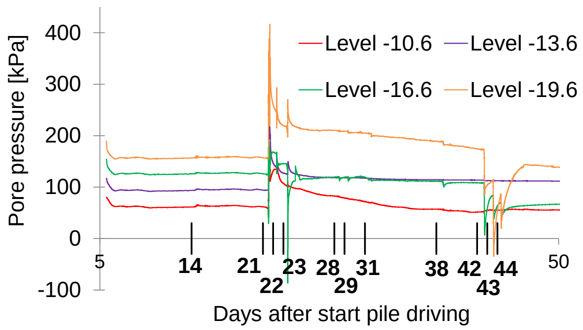

Figure 7. Pore pressure measurements in the period of pile driving. The numbers in bold refer to the days where piles near the piezometer was driven (see Figure 4)

Only minor pore pressure changes were registered from these four piles (about 2% increase) (Figure 7). The pore pressure increase were uniform in all 4 piezometers and the excess pore pressure dissipated in around 24 hours. At day 21, 22 and 23 clusters of 16, 5 and 9 piles respectively were driven. These were the piles closest to the piezometers and the influence on the measured pore pressures was seen to be very significant. The nearest piles (day 21) were in a distance of only 1.8 meters (~11 pile radii) from the piezometers. The pore pressure measurements during day 21 – 23 are presented in Figure 9. Excess pore pressures measured were in the range of 40% – 160% of the initial value. In the minutes after completion of a pile driving sequence a quick dissipation of excess pore pressures is observed. Within hours, the dissipation seem to follow a more gradual dissipation course.

The highest excess and the most fluctuations in pore pressures were measured with the lowest piezometer in level -19,6 near pile tips. It is out of scope for this paper to go into details with every driven pile and its influence on the measured pore pressure, however the pile sequences at day 21 are shown in detail in Figure 8 for illustration of the complexity in pile driven induced pore pressures.

The figure shows how the build-up of excess pore pressure progress differently with depth. While the pore pressure build up is relatively smooth at the upper levels (-10.6 and -13.6 in particular) the pore pressure response in the lower piezometers closer to the pile tip (-16.6 and -19.6) are more irregular. Regular abrupt drops in pore pressure (level -19.6) are observed after driving of piles 4, 5, 8, 9, 12, 14 and 16. It seems to be coinciding

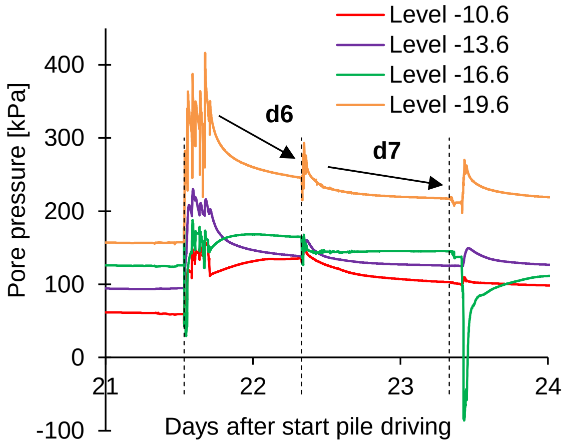

Figure 9. Pore pressure measurements day 21 - 23. (dashed lines indicate start of pile driving).

with the pile passing the piezometer and is probably due to local disturbance in the grout and clay surrounding the piezometers. The pore pressure is seen to undergo a quick reversion (within minutes).

The arrows in Figure 8 show pore pressure build-up and dissipation trends for the lowermost piezometer. It is seen how the piles near (N) the piezometers result in pore pressure build-up whereas distant piles (D) and piles experiencing a shadow effect from already driven piles (S) do not affect the pore pressures much.

A somehow atypical pore pressure response was seen at day 42 – 44 (Figure 10). Although a profound shadow effect would be expected due to the amount of already driven piles (cf. Figure 4), the pore pressure in level -16.6 and -19.6 was severely affected by the piles whereas level -10.6 and -13.6 was unaffected. The response was solely negative changes in pore pressures directly associated with each pile driving.

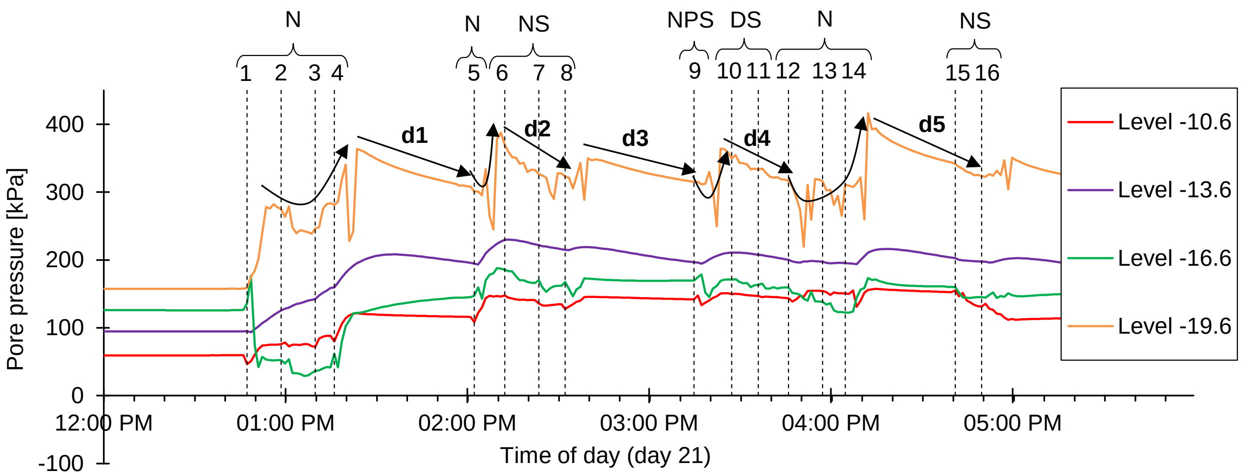

Figure 8. Pore pressures measured during day 21. Dashed lines indicate a starting point of a pile driving sequence. (Pile numbers refer to Figure 5). N = near, S = shadowed, PS = partly shadowed, D = distant.

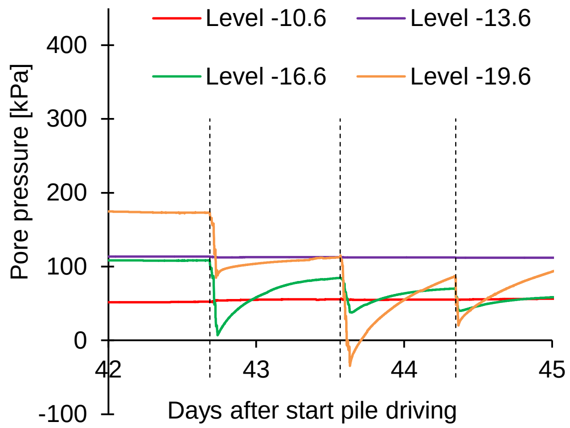

Figure 10. Pore pressure measurements day 42 – 44. (dashed lines indicate start of pile driving).

6.4 Dissipation and pore pressures measured after pile driving

As pile driving ceased, the pore pressures started to dissipate. The dissipation is clearly seen at Figure 11.

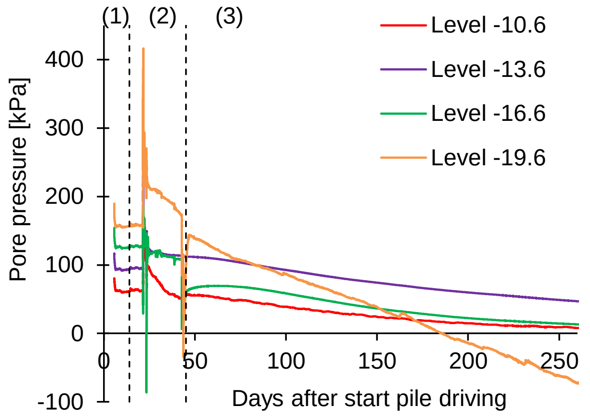

Figure 11. Pore pressure measurements over the entire period (day 250 = 25th of September 2015). Stage (1), (2) and (3) = before, during and after pile driving near piezometers.

The induced excess pore pressures are expected to dissipate towards the initial values over time. However, at this site the pore pressures in all four levels have continued to decrease for more than 200 days and have so far reached values far below the measured initial pore pressures. The pore pressure profile at day 260 after pile driving is presented at Figure 6. The pore pressure in level -19.6 is dissipating linearly over time with a constant rate and do not yet seem to move towards a steady state. The pore pressures in the other three levels display a slight decrease in dissipation rate but have not yet reached constant levels.

Different mechanisms are believed to control the dissipation and in Figure 12 dissipation events are illustrated with plot of the normalized dissipation with time where Δu is the decrease in pore pressure (dissipation) and u_0 is the pore pressure start value (peak value) before dissipation starts. d1 – d7 (cf. Figure 8 and Figure 9) illustrate the quite rapid and short-termed decrease of large excess pore pressures immediately after pile driving, whereas a long-term and more gradual dissipation trend is seen in stage (3) at Figure 11. The short-term dissipation courses are believed to be controlled by local conditions e.g. permeability of the grout around the piezometers, whereas the long-termed dissipation supposedly is governed by soil permeability.

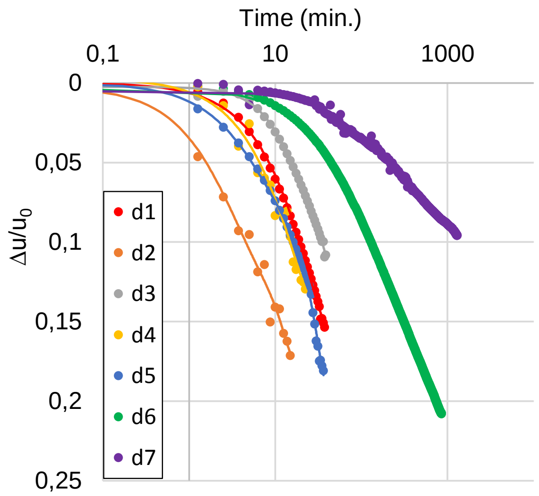

Figure 12. Dissipation d1 – d7 (cf. Figure 8 and Figure 9). From piezometer in level -19.6

7 DISCUSSION

Diverse steady-state pore water pressure profiles immediately after piezometer installation have been found between this study and the study by Wan and Standing (2014). This is likely to be a result of differences in grout and soil properties. The grout used in the present study may have experienced significant shrinkage during hydration due to a very low bentonite content, which could have led to a vertical flow path between the borehole wall and the grout column or even cracks that could have increased the macro-permeability of the grout. These factors could have caused pseudo-hydrostatic conditions in the borehole, leading to an initial observed pore pressure profile which is near hydrostatic. From the continued measurements, it seems though that pile driving processes may have closed such shortcuts in the drainage paths in the grout, as the continued measurements show no signs of interconnection between piezometer levels.

During the initial stage of pile driving (day 21), large positive excess pore pressures are build up. But in the latter stage of the driving process (day 42 – 44) large negative changes in pore pressures are observed – solely in the clay near the pile tip. The pore pressures in the upper part of the clay were apparently unaffected by the piles, maybe due to shadow effects. The rapid dissipation indicate that the conditions are locally induced and hence controlled by grout properties. Dilation in the grout due to shearing can have caused the negative changes in pore pressure.

Dissipation of excess pore pressures begin immediately after pile driving. After a rather rapid dissipation in the minutes after pile driving a prolonged and gradual dissipation trend is observed in the days and monts after pile driving. The measurements show an almost linear dissipation trend continuing for more than 200 days after pile driving with pore pressures significantly lower than before pile driving and even negative values near the level of the pile tips. This tendency is not fully understood, but according to the theory, the pore pressures should dissipate over time and adjust to the initial values. However, as the initial pore pressure values are not accounted for due to presumably misleading initial pore pressure measurements, it is unknown at which stage the pore pressures will display steady state conditions. The pore pressure measurements will continue in the following years in order to shed light on this.

8 CONCLUSION

Pile driving in highly overconsolidated clay lead to the build-up of large excess pore pressures. Through pore pressure measurements with multilevel vibrating wire piezometers near clusters of piles at a construction site it has been found, that pile driving lead to a redistribution of pore pressures and that magnitude and distribution of pore pressures in relation to pile driving in highly OC clays is very complex. The interplay between grout and soil permeability and shadow effects from existing piles makes it very difficult to make a reliable estimate of the pore pressure conditions without direct measurements.

A summary of findings from this study is listed below:

- Pile driving in highly OC clays induce large excess pore pressures in the soil close to the pile. In this study pore pressures are influenced by pile driving within 100 pile radii from the pile. Beyond this distance no influence on pore pressures from pile driving has been observed. However, the distance depends on the distribution of already driven piles. Piles near a piezometer can generate a shadow effect so that no pore pressure changes are registered from piles placed behind these even though they may be within 100 pile radii from the piezometer.
- Pile driving is seen to not only induce positive pore pressure changes. Several pile driving events have caused locally negative changes in pore pressure controlled by the grout properties.
- The pore pressure response differs with depth in the soil. The most irregular pore pressure progress is seen near the pile tip where excess pore pressures of up to 160% are induced. In the remaining levels, a more smooth development is detected with excess pore pressures of 40% – 100%.
- General soil and local grout permeability conditions are believed to play a role in dissipation of long term and short term excess pore pressures respectively. The long term dissipation is observed to reach values far below the initial values.
- The importance of using a suitable grout for equipment installation with the fully grouted method has to be considered to obtain the desired properties. In this study it is likely that shrinkage of the grout has influenced the pore pressures measured before pile driving.

One of the goals with this study was to shed light on the complexity of the pore pressure distribution in heavily OC high plasticity clays during pile driving. Additional pore pressure measurements are planned at neighboring construction sites and test fields with similar ground conditions. Results from these tests will hopefully help to further shed light on the mechanisms of pore pressure development due to pile driving in heavily OC high plasticity clay. The results are to be discussed in future publications.

The significance of grout properties when using the fully grouted method in heavily OC high plasticity clays require further examination.

9 ACKNOWLEDGEMENTS

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10 REFERENCES

Bergset, K. H. (2015). Radial consolidation of pore pressure induced by pile driving. NTNU Trondheim, master degree thesis.

Bond, A. J. & Jardine, R. J. (1991). Effects of installing displacement piles in a high OCR clay. *Geotechnique* 41, No. 3, pp. 341-363.

Bozozuk, K., Fellenius, B. H. & Samson, L. (1978). Soil disturbance from pile driving in sensitive clay. Canadian Geotechnical Journal 15, pp. 346 – 361

Burns, S. E. & Mayne, P. W. (1999). Pore Pressure Dissipation Behavior Surrounding Driven Piles and Cone Penetrometers. Transportation research record 1675, paper no. 99-0423, pp. 17-23.

Chandra, S. & Hossain, M. I. (1993). Prediction and Observation of Pore Pressure Due to Pile Driving. Proceedings: Third International Conference on Case Histories in Geotechnical Engineering, St. Louis, Missouri, June 1-4, Paper No. 1.66.

Contreras, I. A., Grosser, A. T. & Ver Strate, R. H. (2008). The Use of the Fully-grouted Method for Piezometer Installation. Geotechnical News, Bi Tech Publishers, pp. 30 - 40, June.

Coop, M. R. & Wroth, C. P. (1989). Field studies of an instrumented model pile in clay. *Geotechnique* 39, No. 4, pp. 679-696.

Eigenbrod, K. D. & Issigonis, T. (1996). Pore water pressures in soft to firm clay during driving of piles into underlying dense sand. Canadian Geotechnical Journal, 33, pp. 209 – 218.

Gue, S. S. (1984). Ground heave around driven piles in clay. PhD Thesis, University of Oxford

Karlsrud, K. (2012). Prediction of load-displacement behaviour and capacity of axially loaded piles in clay based on analyses and interpretation of pile load test results. NTNU Trondheim. PhD Thesis.

Lo, K. Y. & Stermac, A. G. (1965). Induced pore pressures during pile-driving operations. Proc. 6th International Conference on soil mechanics and foundation engineering, Montreal, Vol. II, pp. 285-289

Marefat, V., Chapuis, R. P., Duhaime, F., 2014. Piezometric error for fully grouted piezometers Installed in clay layers. GEO Regina 2014, 67th Canadian Geotechnical Conference

Martin, R. E., Seli, J. J., Powell, G. W. & Bertoulin, M (1987). Concrete pile design in tidewater Virginia. Journal of geotechnical engineering, vol. 113, no. 6, pp. 568-585.

McKenna, G. T. (1995). Grouted-in installation of piezometers in boreholes. Can. Geotech. J. 32, pp. 355-363.

Mikkelsen, P. E. & Green, G. E. (2003). Piezometers in Fully Grouted Boreholes Symposium on Field Measurements in Geomechanics, FMGM 2003, Oslo, Norway, September.

Okkels, N. & Bødker, K. (2008). Beregning af hævninger under antagelse af retlinjet dræning. NGM 2008.

Poulus, H. G. & Davis E. H. (1980), Pile foundation analysis and design, John Wiley and Sons, New York.

Randolph, M. F. & Wroth, C. P. (1979). An analytical solution for the consolidation around a driven pile. Int. Journal for numerical and analytical methods in geomechanics, vol. 3, pp. 217-229.

Robertson, P. K., Woeller, D. J. & Gillespie, D. (1990). Evaluation of excess pore pressures and drainage conditions around driven piles using the cone penetration test with pore pressure measurements. Canadian Geotechnical Journal, JV27, N2, pp. 249 - 254, April.

Simeoni, L., De Polo, F., Caloni, G. & Pezzetti, G. (2011). Field performance of fully grouted piezometers. FMGM, Berlin.

Thorsen, T. & Okkels, N. (2015). The New Lillebælt Bridge – a settlement model. XVI European Conference on Soil Mechanics and Geotechnical Engineering, Edinburgh, September.

Vaughan, P. R. (1969). A note on sealing piezometers in boreholes. *Geotechnique* 19, No. 3, pp. 405-143.

Wan, M. S. P. & Standing, J. R. (2014). Field measurements by fully grouted vibrating piezometers. Proceedings of the Institution of Civil Engineers. Geotechnical engineering. Vol. 167, Issue 6, pp. 547-564.