Field measurements of pore-water pressure changes in a stiff fissured very high plasticity Palaeogene clay during excavation and pile driving.

Mesures sur le terrain des changements de pression de l'eau interstitielle dans une argile Paléogène très rigide hautement plastique lors des traveaux de creusement et de battage de pieux.

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ABSTRACT: Deep excavations induce stress relief of the underlying substratum. In areas of Denmark where the near-surface substratum consists of heavily overconsolidated, Palaeogene clays of very high plasticity this poses a specific challenge. Stress relief from excavation will typically over time cause the clay to heave – a heave that is governed by the dissipation of negative excess pore-water pressures induced by unloading. However, if foundation piles are driven immediately after excavation – which is common practice in connection to deep basement excavations in Denmark – large positive excess pore water pressures may develop that may fully or partly equalize the negative excess pore-water pressures. In order to shed light on pore-water pressures during excavation and pile driving, piezometers were installed at a construction site in various depths and at different distances to driven piles. This paper presents and discusses pore-water pressure measurements obtained during excavation and pile driving. As expected, the observations show that excavation leads to a drop in pore-water pressure and pile driving induce large excess pore-water pressures in the clay.

RÉSUMÉ: Les excavations profondes entrainent un soulagement du substrat sous-jacent. Dans les régions du Danemark où le substrat proche de la surface est constitué d'argiles Palaeogene très fortement surconsolidées de très grande plasticité, cela pose un défi particulier. L'annulation des contraintes causée par une excavation entraînera habituellement, au fil du temps, un soulevement de la couche argile - soulèvement qui est provoqué par la dissipation des surpressions négatives de l'eau interstitielle suite à ce déchargement. Cependant, si des pieux sont enfoncées immédiatement après les travaux de creusement - ce qui est une pratique courante au Danemark lors d'excavations profondes – une forte surpression positive des eaux interstitielles peut se développer et ainsi égaliser totalement ou partiellement les surpressions négatives d'eaux interstitielles. Afin de faire la lumière sur les pressions de l'eau interstitielle lors d'excavation et battage de pieux, des piézomètres ont été installés sur un chantier de construction à différentes profondeurs et distances des pieux enfoncés. Cet article présente et discute les mesures de pression de l'eau interstitielle obtenues lors des travaux d'excavation et de battage. Les observations montrent que l'excavation entraîne une chute de la pression de l'eau interstitielle et que le battage induit une surpression des eaux interstitielles dans l'argile.

KEYWORDS: High plasticity stiff overconsolidated clays, pile driving, field testing, pore water pressure, fully grouted method

1 INTRODUCTION

The present work is part of an ongoing research project about pore pressure development in stiff fissured, highly overconsolidated clays of very high plasticity in relation to deep excavations and pile driving. The purpose is to gain a better understanding of the complex pore pressure development caused by excavation induced unloading in combination with subsequent loading from driving of displacement piles.

From basic soil mechanics theory, it is well known that pore-water pressures will drop instantly in underlying clay strata as a response to excavation (i.e. unloading). Assuming elastic theory and fully saturated conditions, the drop in pore pressure corresponds to the magnitude of change in the mean stress, $\Delta p = 1/3(\Delta\sigma_V+2\Delta\sigma_H)$. The subsequent dissipation of negative excess pore pressures is controlled by e.g. drainage paths and soil permeability and can last for decades in very low permeable clays ($k < 10^{-10}$ m/s). The clay type presented in this study is rich in highly expansive minerals (smectite), and the dissipation process may cause very significant swelling of the clay and heave of the excavation level (Okkels and Bødker 2008). Examples of heave of other types of stiff, highly overconsolidated clays due to unloading can be found in the literature (e.g. May 1975 and Fehmarnbelt (Fixed Link) 2013).

The build-up of excess pore pressures at the pile surface during pile installation and subsequent dissipation have been intensively studied in the past for a variety of soil types, including stiff, highly overconsolidated clays (e.g. Bond and Jardine 1991 and Coop and Wroth 1989). However, for Danish soil types and stiff, highly overconsolidated clays in particular, the available data are very sparse. The only reported case involves a instrumented single open-ended steel tube pile (OD 508x22.2 mm) driven 25 meters into the underlying clay at an offshore test site (Fehmarnbelt (Fixed Link) 2013).

Generally, studies found in the literature have focused on the build-up of pore water pressures at the pile surface, while the changes in pore pressures further away from the pile surface have received much less attention. No published studies have been found which presents measurement of pore pressures during unloading in combination with subsequent pile driving.

This paper presents pore pressure measurements in a Danish Palaeogene clay type called Søvind Marl as obtained at a construction site from piezometers installed in boreholes and at the surface of an instrumented test pile. Measurements were taken before, during and after excavation and pile driving.

2 SITE CONDITIONS

Field measurements took place at a construction site at the harbour of Aarhus starting in the early summer of 2016. The building project (called AARhus) consist of an approx. 8000 m^2, 20-storey building complex with parking basement. The building is supported on more than 800 precast 300mm x 300mm concrete foundation piles driven up to 34 meters into

the ground. In order to monitor pore-water pressures changes, a total of 17 piezometers were installed in two measurement areas approx. 40 meters apart (Figure 1). 13 piezometers were installed in 5 boreholes and 4 piezometers were installed in one precast concrete pile with filtertips in level with the pile surface.

Three boreholes were carried out prior to excavation and the two others between the period of excavation and pile driving. Furthermore, inclinometer casings and extensometer anchors were installed, in order to correlate soil deformation and pore pressure changes. These deformation data are currently being processed and will be presented in future publications. After lowering the groundwater approx. 4 meters, excavation started southeast of measurement area (M.A.) 1 and moved towards M.A.2 lasting 15 days. Pile driving commenced successively after excavation.

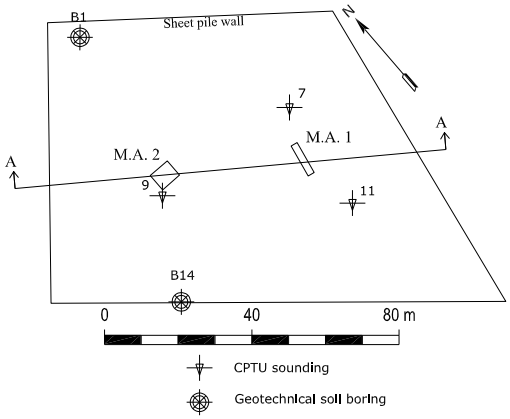

Figure 1. Site plan. (M.A = Measurement Area)

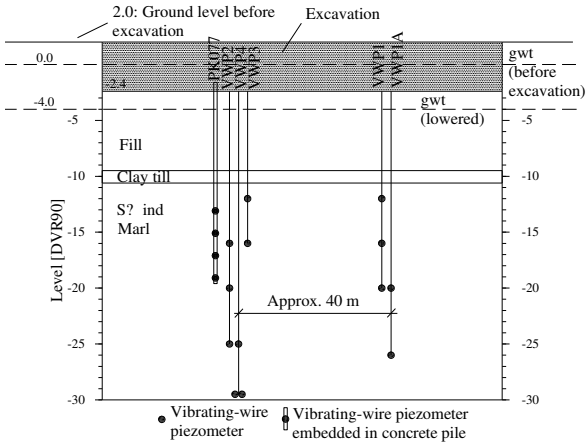

Figure 2. Cross section A-A. Overview of installed piezometers.

2 .1 *Geology*

The harbour area was reclaimed in 1971, 45 years ago. The upper soil strata consist of approx. 12 meters of sand fill with interbedded clay layers. Below there is a thin layer of clay till overlying the Søvind Marl formation which is extending to great depth (~70 meters). The Søvind Marl is a very fine-grained marine sediment deposited around 45 to 35 million years ago (middle to late Eocene). The clay fraction (up to around 70% of the soil matrix) is very rich in smectite minerals. Further geological information about the Søvind Marl can be found in (Grønbech et al. 2015 and Simonsen et al. 2017).

2 .2 *Geotechnical properties*

Compared to other younger clay types Søvind Marl exhibit unusual geotechnical properties which is described by e.g. (Grønbech et al. 2015). This being primarily due to its high content of smectite and varying carbonate content (0 – 60 % $CacCO_3$). The natural water content is between 34 and 55 %. The clay is extremely plastic with liquid limits W_L between 100 and 294 % and plasticity indexes I_p = 70 to 250 %. It is furthermore heavily overconsolidated due to the weight of eroded younger layers and numerous glaciations in the Quaternary period. The overconsolidation ratio is well beyond 20. Undrained shear strength between level -10 and -30 have been found from CPTU's to be 100 kPa $\leq c_u \leq$ 150 kPa with N_{kt} ~ 23. The permeability has been determined at $\leq 3 \cdot 10^{-11}$ m/s from oedometer and permeability tests and the effective strength parameters c´ ~ 20 kPa and ϕ` ~ 16°. The coefficient of consolidation, c_v has been found to vary between $1 \cdot 10^{-7}$ – $1 \cdot 10^{-9}$ m^2/s. Samples of Søvind Marl often appear fissured with slickensides.

3 METHODS

Vibrating wire (VW) piezometers (Geosense VWP-3001, 690 kPa HAE) were chosen in favour of other piezometer types due to their proved long-term accuracy and reliability under different installation conditions as well as long-term reliability (Bayrd 2011 and McRae et al. 1991). The functional requirements for the piezometers were high, as the sensors should function under harsh dynamic testing conditions during pile driving. Previous field testing by (Bullock et al. 2005 and Hajduk et al. 2000) have shown that VW piezometers are capable at measuring directly on the pile surface during pile driving with only little risk of sensor failure. Furthermore, VW piezometers contain a diaphragm sensor and requires only a very small equalization volume (Wan and Standing 2014) which makes it an obvious choice for installation in very low permeable clays where water flow is very restricted. High air entry ceramic filters were chosen to limit desaturation.

3 .1 *Fully grouted method*

13 piezometers were installed in boreholes back-filled with a suitable cement-bentonite grout. (Vaughan 1969) first described this method, which is referred to as the fully grouted method, and over the recent years, the geotechnical community worldwide has adopted it. The most obvious advantages of this installation method is that it allows installation of several piezometers in different levels in the same borehole.

An effective filter saturation and a hermetic grout seal are essential prerequisites for a successful installation. Thus, the high air entry piezometer filters were saturated by boiling for at least two hours and then kept under de-aired water until installation. Grout was then applied through a tremie-pipe from the bottom of the borehole and up, and piezometer cables were collected within the 1.25" PVC host pipe, which the piezometers were mounted on. The performance of the grout was tested in a laboratory trial setup. The test verified that the grout when set would make a perfect seal and not introduce any voids or leakages along cables or pipes in the grout column.

The grout should represent the soil which it replaces in terms of permeability and strength properties. (Contreras et al. 2008) demonstrated that grout permeability could be 1000 times higher than soil permeability without introducing significant errors in the measurements of pore water pressure. Based on laboratory tests the most suitable grout was found to have a water/cement/bentonite ratio of 2.5/1.0/0.35 by weight. Hence, this grout composition was applied for the fieldwork.

3 .1 *Piezometers cast into concrete test pile*

4 piezometers were mounted in a 18m long precast concrete test pile. As shown in Figure 3, the piezometers were secured to the

pile reinforcement with the filter tip upwards and in level with the pile surface. The piezometers were covered and sealed during pile construction and uncovered and saturated just before pile driving at the construction site.

Figure 3. Piezometer in test pile (left/right: before/after casting).

4 RESULTS

One of the aims of the piezometer installation was to observe the steady-state pore pressures prior to excavation and observe the undrained pore pressure response during unloading. However, due to external constraints in relation to the construction work, it was not possible to have the first piezometers installed until 10 days prior to excavation start. As described below, this is probably inadequate time for the piezometers to adjust to the far-field steady-state pore-water pressure in very high plasticity and very low permeable clays.

Selected pore pressure measurements have been chosen for presentation in this paper. The ongoing measurements yield an extensive amount of data, which will be included in future publications.

4 .1 *Pore-water pressures before, during and after excavation*

During borehole drilling in stiff, overconsolidated clay, negative excess pore pressures can be generated along the borehole wall due to stress relief and shearing from the drilling equipment. Measurements in stiff, overconsolidated London Clay have shown pore pressures due to stress relief that were in the order of 15 – 85 % less than steady-state conditions (Wan and Standing 2014). The time needed for dissipation until pore pressures balance with the far-field steady-state values, is controlled mainly by the soil permeability and have been observed to last from a few weeks up to two months after installing piezometers in fully grouted boreholes in London Clay (Wan and Standing 2014).

The pore pressures in the clay are controlled by e.g. K0 conditions and as the Søvind Marl has undergone a complex stress history, the K0 conditions are difficult to predict. This makes it complicated to predict the expected pore pressures before construction start. However, from simple analytical calculations the expected pore pressures have been estimated.

The immediate response to the up-fill 45 years ago is a pore pressure increase of around 136 kPa assuming K0 = 1 (Figure 4). This excess pore pressure will dissipate at a rate that is primarily dependent on the permeability. The laboratory coefficient of permeability for Søvind Marl is $3 \cdot 10^{-11}$ m/s and lower. However, according to (Thorsen and Okkels 2015 and Simonsen et al. 2017) and others, the field permeabilities of very low permeable clays are most likely higher than the laboratory values. Hence, a best estimate of $k = 3 \cdot 10^{-10}$ m/s ($\sim 10 \cdot k_{lab}$) and $c_v = 2 \cdot 10^{-7}$ m^2/s was used in the calculations.

From the calculations, it was found that after 45 years (in 2016) the excess pore pressures cannot be expected to be fully dissipated which can be seen as U_predicted_45y in Figure 4. The figure also shows that the pore pressures measured with the piezometers VWP1 – VWP3 before excavation were below U_predicted_45y and even below the hydrostatic pore pressures. This clearly indicates that the piezometers were still not measuring the correct far-field pore-water pressure when excavation started, possibly due to unequalised stress relief as described above.

Figure 5 shows the change in pore pressures as measured in piezometer boreholes VWP1 and VWP2 during the period of excavation. At day 5 and 9 respectively, excavation took place around the piezometer boreholes in measurement areas (M.A.) 1 and M.A.2. The pore pressures are seen to drop around 10 kPa as response to the ongoing unloading as the excavation front approaches each measurement area. As soon as the excavation front has passed the measurements area, pore pressures decrease further. Similar responses were seen in each measurement area with a total drop in pore pressure from unloading of 30 – 40 kPa. Around day 10, pile driving started near M.A.1 and heavily influenced the pore pressures, which is why the rest of the graphs of VWP1 are left out of Figure 5.

If K0 is assumed to be around 1.0, the theoretical drop in pore pressure should be approx. 80 kPa corresponding to the weight of the removed soil (4.4 m · 18 kN/m^3). The measured drop in pore pressure is merely around 40 kPa indicating that the loading from the past land reclamation has led to a decrease of K0 in the clay as described by (Burland et al. 1979).

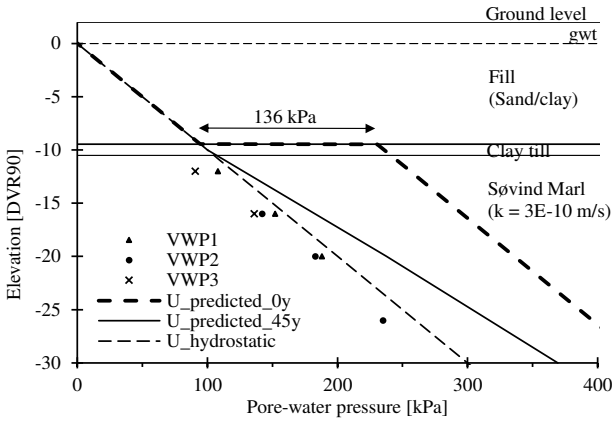

Figure 4. Pore pressure profile before excavation.

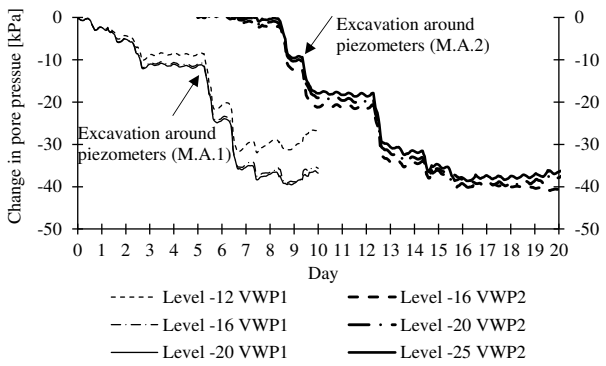

Figure 5 Changes in pore pressures during excavation

4 .2 *Pore pressures during and after pile driving*

Pore pressures measured at the surface of the instrumented pile during pile driving are shown in Figure 6. Large excess pore pressures build up in all levels (note that piezometer P2 show unrealistic low values as it accidently was placed behind a steel lifting-ring sticking out from the surface of the pile).

Dissipation of excess pore pressures happened faster for the upper piezometers than for the lower piezometers (Figure 7). The dissipation data show a linear correlation between dissipation rates and depth below the clay surface, which indicate that drainage is not just radially away from the pile wall as described by (Bond and Jardine 1991) but is also likely to be dependent on the drainage path along the pile surface. Piezometer P1, P2 and P3 were fully dissipated in approx. 4, 14 and 24 days respectively, whereas the piezometer P4 in level -

19.1 had reached a dissipation rate of around 96% at day 25. From day 25 and onwards, nearby pile driving affected the pore pressure measurements and hence interrupted the dissipation process (which is why the graphs in Figure 7 stop at day 25).

Dissipation rates are remarkably faster than observed in the tests by (Fehmarnbelt (Fixed Link) 2013). At that site, excess pore pressures following pile driving were still dissipating after one year of measuring at the pile surface. This may be explained by use of another pile material (steel), location of the piezometers at greater depth and due to a lower permeability of the clay compared to this study.

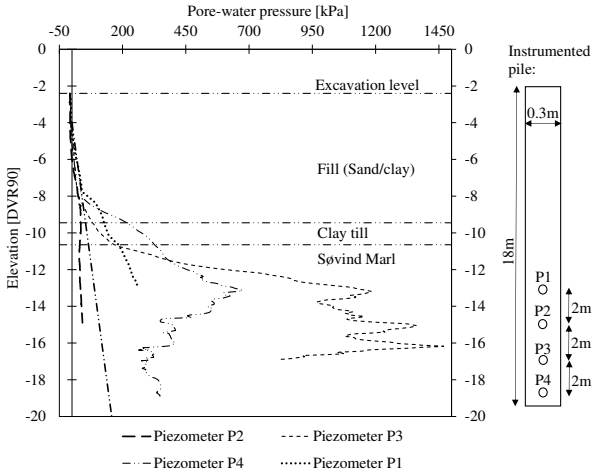

Figure 6. Pore pressures during driving of instrumented test pile.

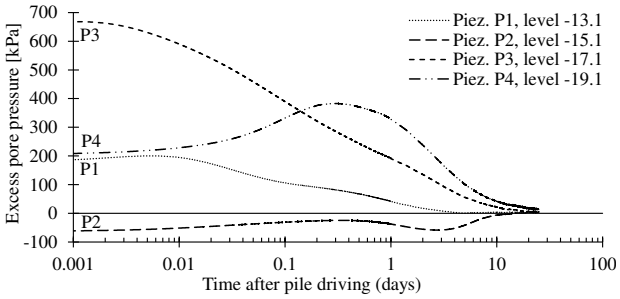

Figure 7. Dissipation of excess pore pressures after driving of instrumented test pile (P2 was influenced by excessive soil disturbance).

5 MAIN FINDINGS AND CONCLUSIONS

Field measurements of pore-water pressures in stiff overconsolidated clay at a construction site during combined excavation and pile driving, have documented that excavation (unloading) lead to an instant decrease of pore pressure. Pile driving induces large excess pore pressures at the pile/soil interface, which start dissipating when pile driving is ceased.

Data from piezometers measuring pore pressures in the soil body around driven piles show complex pore pressure distributions in the clay with build-up of both positive and negative excess pore pressures and different dissipation rates depending on depth and distance to the driven piles. This paper did not leave space for these data but they will be presented in future presentations together with deformation data from extensometer and inclinometer measurements.

The study has shown that in very high plasticity clays:

(1) Piezometers should be installed prior to construction work in adequate time for the pore pressures to equalize to far-field steady-state pressures.

(2) Vibrating wire piezometers can survive harsh conditions such as pile driving.

(3) It is possible to measure the undrained response from unload due to excavation.

(4) The fully grouted method for piezometer installation seems to work well even in very low permeability clays when measuring pore pressure response from excavation. Future publications will discuss the applicability of this method when measuring pore pressures near driven piles.

(5) The magnitude of pore-water pressure decrease as a response to excavation (unloading) is less than expected, which indicates that loading from the up-fill 45 years ago has led to a decrease of K0 in the clay.

(6) It is recommended to measure the far-field steady-state pore pressure in conventional standpipe piezometers in order to confirm the VW piezometers readings (not done in this study).

6 ACKNOWLEDGEMENTS

The authors would like to thank Geo and Innovation Fund Denmark for proving funding for this study and Centrum Pæle A/S and Aarsleff A/S for help with production and driving of the instrumented test pile. MT Højgaard A/S is sincerely acknowledged for letting us use their site for research and their engagement in keeping the equipment safe during heavy on-going construction work.

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