Permeability of a stiff fissured very high plasticity Palaeogene clay - direct and indirect measurement methods and scale effects.

Perméabilité des argiles Paléogène fissurés et rigides de très haute plasticité - méthodes de mesure directes et indirectes et les effets d'échelle.

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ABSTRACT: Different methods of measuring the direct and indirect permeability of clays exists, both in the laboratory and in the field. However, traditionally the determination of permeability of heavily overconsolidated and stiff Danish Palaeogene clays often rely only on the oedometer test method, which is an indirect test method. However, this method has previously been shown to underestimate the in situ permeability of this type of clay. This paper explores the differences in permeabilities derived from indirect and direct permeability tests in the laboratory and the field through a series of oedometer tests, constant head permeability tests, rising head tests and CPTU dissipation tests. The results show that the indirect permeability tests yield the lowest permeabilities whereas the direct measuring methods yield only slightly higher permeabilities. Laboratory tests are prone to underestimate the in situ permeability of the clay due to the presence of macro structures.

RÉSUMÉ: Différentes méthodes de mesure, directes et indirectes, de la perméabilité des argiles existent, tant en laboratoire que sur le terrain. Cependant, traditionnellement, la détermination de la perméabilité des argiles Paléogène danoises très surconsolidées et rigides repose souvent uniquement sur la méthode d'essai de l'oedomètre, qui est une méthode d'essai indirecte. Cependant, il a été déjà démontré que cette méthode sous-estimait la perméabilité in situ de ce type d'argile. Cet article explore les différences dans les perméabilités calculées à partir de tests de perméabilité indirectes et directes en laboratoire et sur le terrain à travers une série d'essais oedométriques, de tests de perméabilité sous charge d'eau constante, de slug tests (essai d'injection-relaxation) et de tests de dissipation par pénétration statique (CPTU). Les résultats montrent que les tests de perméabilité indirecte donnent les plus faibles perméabilités alors que les méthodes de mesures directes ne donnent que des perméabilités légèrement plus élevées. Les tests de laboratoire sont susceptibles de sous-estimer la perméabilité in situ de l'argile en raison de la présence de macro-structures.

KEYWORDS: Permeability, high plasticity stiff overconsolidated clays, laboratory testing, field testing, CPTU dissipation tests

1 INTRODUCTION

Knowledge about pore pressure build-up and dissipation is of high relevance in a wide variety of geotechnical problems. Permeability of the soil is a governing factor in relation to dissipation of excess pore pressures. When dealing with stiff Danish Palaeogene clays of very high plasticity, which are known to be very low permeable ($k < 1 \times 10^{-10}$ m/s) it can be challenging to make an accurate direct determination of permeability (coefficient of permeability) based on flow tests. Hence, this parameter is often derived indirectly from standard oedometer tests based on the recorded consolidation behaviour.

In this study, a series of laboratory test have been carried out on specimens of a Danish high plasticity Palaeogene clay called Søvind Marl. Based on the results the relationship between laboratory values for indirectly derived permeabilities from oedometer tests and directly measured permeability from constant head flow tests in a triaxial cell is investigated. In addition, the relationship between laboratory values and field values of permeability from rising head tests and CPTU dissipation tests is assessed.

1.1 Previous Studies

(Skempton and Henkel 1960) were among the first to address permeability of stiff, high plasticity and very low permeable clays. From laboratory permeability tests and oedometer tests on samples of London Clay, they found that measuring the permeability directly from permeability tests yielded approx. four times higher values of permeability than when deriving the permeability indirectly from oedometer tests. When compared to rising head field tests, the values were about the same as measured from laboratory permeability tests. Thus indicating, insignificant scale effects between results obtained from flow tests performed in the laboratory and field. In contrast, (Chandler et al. 1990) found that the field permeability to laboratory permeability of London Clay was about 4 times higher when measured with a self-boring permeameter and up to 50 times higher when determined from constant-head tests in piezometers. They also found that the permeability anisotropy factor $r_k$ was 2.1 ($r_k = k_h/k_v$).

Studies on the permeability of Danish Palaeogene clays are sparse. However, the available studies indicate, that the directly measured and indirectly derived laboratory values of permeability (from oedometer tests) are in good agreement (Jacobsen 1967), and that the anisotropy factor is around 1. Back-calculations of heave data (i(Fehmarnbelt (Fixed Link) 2013)) and settlement data (e.g. Jacobsen 1967 and Thorsen et al. 2015)) indicate that the in situ permeability can be between 10 to 500 times higher than indirect laboratory values. Part of this major variation shall most probably be found in choice of calculation model.

(Tavenas et al. 1989a and 1989b) did a comprehensive study on the permeability of soft to firm natural clays. They found...
that permeability testing in the triaxial cell and falling-head tests in a modified oedometer cell were preferable ways of doing laboratory permeability measurements. On the contrary, they advise against the use of indirect methods of evaluating the permeability from consolidation tests due to the errors that lies in the assumptions contained in Terzaghi’s consolidation theory. Their studies did however not cover any clay types similar to the one addressed in the present study.

Based on the previous studies, there is reason to believe that the coefficient of permeability is underestimated when derived indirectly from oedometer tests.

2 SITE CONDITIONS

In the main parts of Denmark, Palaeogene clays are found at great depths outside normal geotechnical interest. In some areas in the central part of Denmark however, glacial events during the Quaternary Period have dislocated the clay from its original location or removed younger soil layers and therefore the clay can be found near terrain. The test site (approx. 3000 m²) used in this study is located close to a clay pit near the town of Randers. The ground investigations consist of 11 borings and CPTU tests (Figure 1) of which SP6 and SP9 are presented in this study. Ground level is roughly at 58 m.O.D. (DVR90) over the entire test field.

2.1 Geology

The ground conditions in the test field are uniform and consist of approx. 3 – 4 m of clay till with a sharp boundary to the underlying Søvind Marl which extends to at least 15 m below terrain (no boreholes have been taken deeper). The Søvind Marl Formation was deposited in a deep ocean during the middle and late Eocene from around 45 to 35 million years ago. It is a light grey to almost white, very fine-grained marl or calcareous clay. The carbonate content can vary from zero to around 70%. The Søvind Marl Formation includes thin beds of darker coloured, non- or slightly calcareous clay. Apart from these layers, bedding is indistinct due to heavy bioturbation. Although sand or silt lenses never will be found in Søvind Marl a few horizons within the formation are found to be rich in sand-sized glauconite (Heilmann-Clausen et al. 1984).

Investigations from a nearby clay pit show that the clay fraction account for 65-70% of the soil mass and that there are practically no particles larger than 0.01mm (medium grained fraction account for 65-70% of the soil mass and that there are coarse-grained glauconite. These layers were not observed in the other boreholes across the test field.

The boreholes were made with dry rotary drilling technique (6” dimension) and both SP6 and SP9 were equipped with casagrande type (standpipe) piezometers. Water level measurements in the clay till, show that the free water level in the area can be assumed at or very close to ground level.

Table 1. Properties of Søvind Marl (effective strength parameters are estimated based on data from (Geo, 2016))

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
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<td>Plasticity index</td>
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<tr>
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<tr>
<td>CaCO₃</td>
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<td>ϕ’</td>
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2.2 Geotechnical properties

Compared to other younger clay types Søvind Marl exhibit unusual geotechnical properties which is described by e.g. (Granbech 2014). This being primarily due to its high content of smectite and varying carbonate content. It shows extremely high plasticity and it is furthermore heavily overconsolidated due to the weight of eroded younger layers and numerous glaciers in the Quaternary period. The overconsolidation ratio has not been determined for the Søvind Marl in the test field, but expected to be in the order of 10 to 20. Samples of Søvind Marl often appear fissured and with slickensides. Properties of Søvind Marl as determined on samples from the test field are listed in Table 1 and CPTU profiles with geological interpretations are shown in Figure 2.

The CPTU profiles show the ground profile to be rather uniform across the site. The CPTU profile SP6 are representative for the test field as a whole, whereas CPTU SP9 show significant local peaks in the tip resistance below 10 m depth indicating local variations in the clay. Borehole SP9 confirms that the peaks derive from decimeter thick layers of coarse-grained glauconite. These layers were not observed in the other borings or removed younger soil layers and therefore the clay is furthermore heavily overconsolidated (Grønbech 2014). This being primarily due to its high content of smectite and varying carbonate content. It shows extremely high plasticity and it is furthermore heavily overconsolidated due to the weight of eroded younger layers and numerous glaciers in the Quaternary period. The overconsolidation ratio has not been determined for the Søvind Marl in the test field, but expected to be in the order of 10 to 20. Samples of Søvind Marl often appear fissured and with slickensides. Properties of Søvind Marl as determined on samples from the test field are listed in Table 1 and CPTU profiles with geological interpretations are shown in Figure 2.

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The boreholes were made with dry rotary drilling technique (6” dimension) and both SP6 and SP9 were equipped with casagrande type (standpipe) piezometers. Water level measurements in the clay till, show that the free water level in the area can be assumed at or very close to ground level.

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3 METHODS AND RESULTS

Field permeability was measured using two rising head tests in SP6 and SP9 and two CPTU dissipation tests in SP6. Samples for laboratory tests were obtained using 75mm Shelby tubes in borehole SP6 and SP9. In order to determine the laboratory values of permeability, eight IL oedometer tests (indirect measurements) and three constant head permeability tests in triaxial cells (direct measurements) on vertically oriented samples have been carried out. Two of the oedometer tests were run on specimens that were taken directly from the triaxial cell after testing. All specimens can be assumed fully saturated and the void ratios of the specimens before testing in the oedometer cell were between 1.2 and 1.5 (for specimens taken directly from the Shelby tube). The specimens that were taken directly from the triaxial cell had void ratios of around 1.7 before installation in the oedometer cell. The specific gravity of solids were between 2.6 and 2.8 in all specimens. Laboratory and field results are compared in Figure 6.
4.1 Constant head permeability tests

The direct permeability measurements were made on \( d = 70 \) mm and \( h = 30 \) mm specimens in a simplified triaxial cell permeability setup. Each specimen was initially left to consolidate under a constant cell pressure of 200 kPa where after a gradient over the specimen and hence a flow was generated by applying a backpressure at the bottom. The volume of the outflow from the top of the specimen was measured using a precise digital scale with 0.001 g accuracy. To minimize evaporation, the outflow water was collected in a small container with a layer of oil on top. Tap water was used in the backpressure system.

For very low permeable clays, high gradients are often required in order to get a sufficient flow that is measurable. The permeability was obtained assuming the validity of Darcy’s theory of steady state seepage. To validate Darcy’s law initial tests were run with varying gradients from 42.5 to 340 and over this interval, insignificant discrepancies in permeability were observed. A gradient of around 170 was chosen for the subsequent tests.

The flow test implies observation of flow of very small volumes over a long period of time, which is of critical importance for accurate permeability determination. For a meticulous description of sources of errors the reader is referred to (Tavenas et al. 1989a).

4.2 Oedometer test

The indirect permeabilities were derived from oedometer tests on \( d = 60 \) mm and \( h = 20 \) mm specimens. Initially a load was applied large enough to prevent the specimen from swelling after which the specimen was loaded incrementally using a load ratio of 2. Time curves were plotted and the coefficient of consolidation \( c_v \) was derived for each load step using Taylor's curve fitting method. The derivation of the coefficient of permeability from the load step following the initial required load to prevent the specimen from swelling was done assuming the validity of Terzaghi’s theory of one dimensional consolidation. The permeability was taken from the load step following the initial required load to prevent the specimen from swelling after the addition of tap water to the cell (at a vertical effective stress of around 240 kPa).

4.3 Rising head test

Each of the boreholes used for rising head tests, were equipped with 0.7 m filtertips between 11 and 12 m below ground level. The lowermost meter of the borehole was backfilled with sand and a bentonite seal placed above the sand pocket. Bentonite seals were at least 4m thick and Mikolit®B pellets were used for sealing. The pellets have a water absorption capacity of 80-90% and a permeability below 10^{-12} m/s (declared by the manufacturer). Position of the standpipe piezometers are shown in Figure 2.

The rising head field tests were commenced 5 months after installing the standpipe piezometers. They were started by bailing out water of the standpipes where after water level measurements were obtained at regular intervals with water level data loggers, as well as with regular dip meters. Water inflows were measured for around 90 days, at which time only one of the piezometers (SP9) had reached a water level of 12.6 m depth, the penetration was halted leading to pore pressure built up and subsequently dissipation (Figure 4). It is referred to as dilatary pore pressure response as opposed to monotonic response in most other soil types and is most likely due to redistribution of pore pressures around the tip (e.g. Sully et al. 1999). Two dissipation tests were carried out at 12 and 12.6 m depth. The tests lasted 14 days and 5 days respectively.

The dissipation data was interpreted using Hvorslev’s basic time lag theory (Hvorslev 1951) according to which the coefficient of permeability can be determined as a function of the cross sectional area of the standpipe, the basic time lag, \( T \) and the shape factor which is a function of the dimensions of the sand pocket around the standpipe (Hvorslev 1951).

The rising head data from SP6 and SP9 are shown in Figure 3 and the associated calculated permeability can be read from Figure 6. The permeability of the Søvind Marl in SP9 is approx. 13 times higher than in SP6. The difference in permeability is attributed to the relatively coarse-grained layer of glauconite that is found within the section of the porous filter tip of SP9 (see also Figure 2).

The CPTU tests were carried out using a 10 cm² piezocene which was brought down at a constant speed of 2 cm/s while measuring the pore water pressure at the cone tip shoulder (\( u_s \)).

During penetration the measured pore pressures were around zero or slightly negative which is a typical response in stiff and overconsolidated clays (e.g. Sully et al. 1999). At the desired depth, the penetration was halted leading to pore pressure built up and subsequently dissipation (Figure 4). It is referred to as dilatary pore pressure response as opposed to monotonic response in most other soil types and is most likely due to redistribution of pore pressures around the tip (e.g. Sully et al. 1999). Two dissipation tests were carried out at 12 and 12.6 m depth. The tests lasted 14 days and 5 days respectively.

The dissipation test data (\( u_s \) is the estimated equilibrium pore pressure at 12 m and 12.6 m depth).

The dissipation of pore pressures during a CPTU dissipation test is controlled by the coefficient of consolidation in the horizontal direction (Robertson 2010). The dissipation data was interpreted using the method of (Teh and Houlbys 1989) with use of the correction procedures for overconsolidated clay as described by (Sully et al. 1999). Hence the dissipation data were normalized using the change in excess pore pressure at time \( t > 0 \) in proportion to the initial excess pore pressure at \( t = 0 \), where the time of the pore pressure peak is set as the new zero time point (\( t = 0 \)).

In this way, the horizontal coefficient of consolidation can be evaluated as:

\[
c_h = \frac{T^* \cdot R^2 \cdot I \cdot 0.5}{t_{50}}
\]

where \( T^* \) is the modified time factor (Teh and Houlbys 1991), \( R \) is the radius of the cone, \( I \) is the rigidity index of the soil and \( t_{50} \) is the time for 50% dissipation (Figure 5).
The rigidity index is defined as the ratio between the shear modulus \((G)\) and the undrained shear strength \((s_u)\). An accurate determination of \(I_R\) requires advanced laboratory testing or seismic CPT measurements (Krage et al. 2014) – none of which have been available in this study. Experiments from (Fehmarnbelt (Fixed Link) 2013) indicate that \(I_R\) might be around 500. Hence, \(I_R\) has been assumed equal to 500 in this study.

When \(c_s\) is known, the horizontal soil permeability is readily determined from Terzaghi’s consolidation theory where the stiffness has been derived from oedometer tests \((E_{vod} = 8\) MPa for SP6).

![Figure 6. Comparison of laboratory and field permeability](image)

4.5 Comparison of field and laboratory data

Figure 6 show a comparison of laboratory and field permeability values. At SP6, where the soil samples for laboratory tests as well as the field measurements are taken in a homogeneous soil body, the variation in permeability is within a factor of four. Where the soil mass is inhomogeneous due to coarse-grained glauconitic layers (SP9) the field permeability is between four and 110 times higher than the laboratory value. The scatter of the laboratory values is more pronounced for SP9 than for SP6 although measured on samples from the same Shelby tube. When comparing vertical permeability values (laboratory tests) with horizontal (dissipation tests) from Figure 6 there are no indication of significant permeability anisotropy.

5 MAIN FINDINGS AND CONCLUSIONS

A study of the laboratory and in situ permeability of a stiff and highly plastic clay – the so-called Søvind Marl, has shown that:

1. There are no clear indications of permeability anisotropy.
2. Direct and indirect laboratory permeability tests yield approx. the same values of permeabilities. When conducted on the exact same specimens, the direct measurement yield merely 1.2 – 1.8 times higher permeability than the indirect method.

(3) Laboratory tests do not seem to underestimate the in situ permeability considerably when the soil sample is representative of the in situ conditions (SP6).

(4) Macro structures (in this case strongly glauconitic layers) can lead to significantly higher in situ permeabilities than compared to laboratory values. Other macro structures could be fissures, sand/silt lenses, faults etc.

The findings in this study support the findings by (Jacobsen 1967) and partly support the conclusions drawn by (Skempton and Henkel 1960) and Tavenas et al. (1989a and 1989b). However, from the present study it seems that indirect determination of permeability from oedometer tests should not necessarily be disqualified in favour of direct measurements.

The permeability of stiff clays is believed to be partly influenced by plasticity index and void ratio. Further studies are planned in order to identify these relationships.

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7 REFERENCES


