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Kaufmann, Kristine Lee; Nielsen, Benjaminn Nordahl; Augustesen, Anders Hust

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September 2010

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Strength and Deformation Properties of Tertiary Clay at Moesgaard Museum

K. L. Kaufmann[1], B. N. Nielsen[2] and A. H. Augustesen[3]

Aalborg University, September 2010

Abstract

The tertiary clay at Moesgaard Museum near Aarhus in the eastern part of Jutland in Denmark is a highly plastic, glacially disturbed nappe of Viborg Clay. The clay is characterised as a swelling soil, which could lead to damaging of the building due to additional heave of the soil. To take this characteristic, as well as the strength and deformation properties, into account during the design phase, two consolidation tests and one triaxial test have been conducted. This paper evaluates the results of the laboratory tests leading to the preconsolidation stress, the deformation parameters consisting of the swelling pressure, the constrained modulus and the compression index, and the strength parameters comprising the undrained shear strength, the drained shear strength and the effective angle of internal friction.

Keywords: Tertiary clay, strength properties, deformation properties, Moesgaard Museum.

1 Introduction

10 km south of Aarhus, Moesgaard Museum is situated, cf. Fig. 1. The tertiary clay at Moesgaard Museum is a highly plastic, glacially disturbed nappe of Viborg Clay. A geotechnical boring near Viborg known as the Viborg-1-boring gives an overview of the stratigraphy of the tertiary deposits.

Figure 1: Location of Moesgaard Museum near Aarhus marked by a circle.

The tertiary clay at Moesgaard Museum is a swelling soil, i.e. highly ex-

[1]Graduate Student, Dept. of Civ. Eng., Aalborg University, Denmark.
[2]Assistant Professor, M.Sc., Dept. of Civ. Eng., Aalborg University, Denmark.
[3]Specialist in Geotechnical Engineering, Ph.D., M.Sc., COWI A/S. Part-time lecturer, Dept. of Civ. Eng., Aalborg University, Denmark.

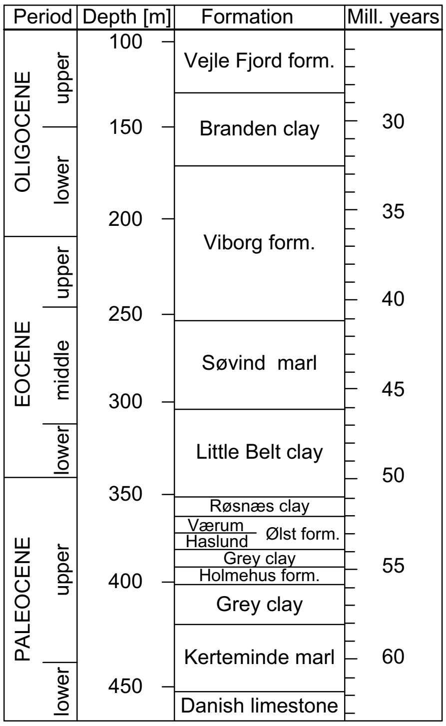

Figure 2: Profile of tertiary deposits based on the Viborg-1-boring near Viborg. According to (Thøgersen, 2001).

pansive in combination with water, which could lead to damaging of the building due to heave of the soil. Thus, it is important to determine at which stress levels swelling is expected to occur, represented by the swelling pressure.

Additionally, deformation and strength parameters of the clay are necessary for a detailed design of the foundation of new buildings. The parameters are, in addition to in situ tests, determined by two consolidation tests and one triaxial test. The results of the three laboratory tests are outlined in this paper.

2 Geological Description

The geological description of the area at Moesgaard Museum is based on two boring records and soil description in connection with the laboratory tests. Boring B106 is more sandy than boring B105. Segments of the boring records in the depths of the soil specimens can be seen in Fig. 4.

The tertiary clay at Moesgaard Museum is a highly plastic, glacially disturbed nappe of Viborg Clay from the Lower Oligocene period. The clay is a glacier deposit where the glacier has transported a large nappe of the Viborg Clay. Viborg Clay is a sea deposit which supersedes Søvind Marl and Little Belt Clay, cf. Fig. 2. The sea deposits are illustrated by the very homogeneous clay from about 12 to 26 m below ground surface. The sign of glacier deposits can be seen by the sand strias and gravel grains at 26–26.5 m and 28–29 m below ground surface. (COWI A/S, 2009)

The clay of the specimens from the two borings is characterised as highly plastic, greyish olive green and slightly micaous. Through the entire layer illustrated by the geotechnical borings, the clay is characterised as medium to very calcareous. This implies the greyish tone of the olive green colour. (COWI A/S, 2009)

Samples of the clay can be seen in Fig. 3. Both a homogenised, wet sample used for the classification tests and the dried, traversed soil specimen for the first consolidation test is shown.

Figure 3: Homogenised soil used for the classification tests (left) and traversed dried soil specimen used in the first consolidation test (right).

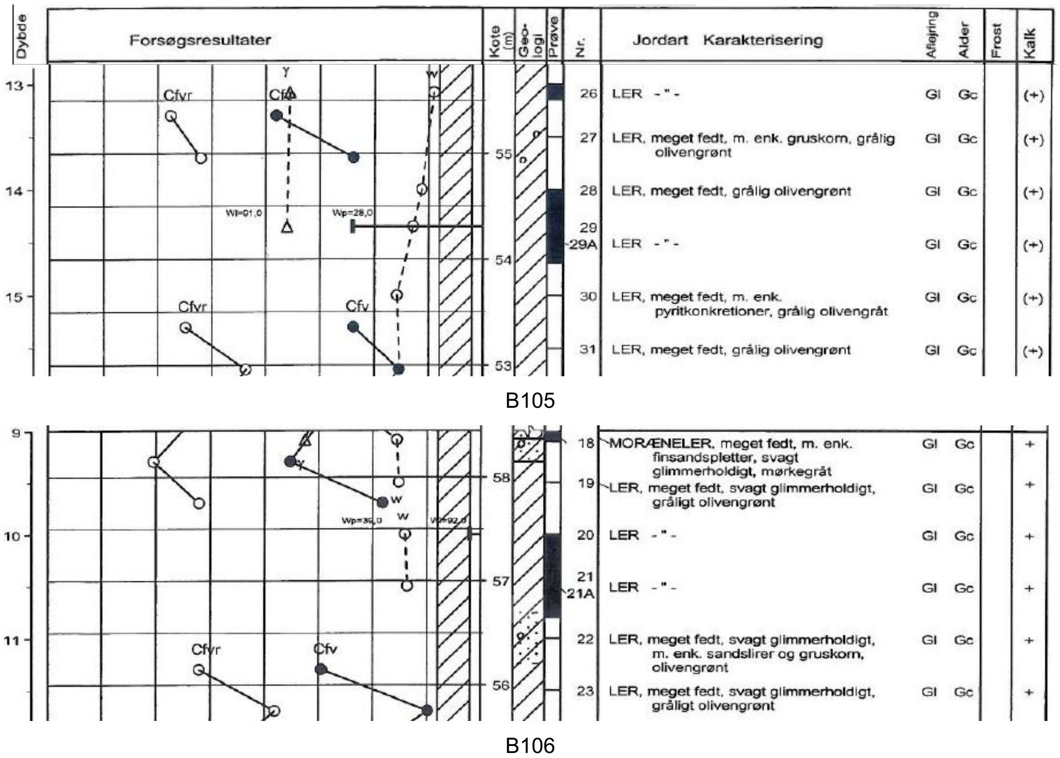

Figure 4: Segments of boring records B105 and B106 in depths of the soil specimens (29 and 21, respectively).

3 Classification

The tertiary clay at Moesgaard Museum is classified by the results of two geotechnical borings and the laboratory tests comprising the consolidation and triaxial tests. Following classification tests are conducted:

- Rolling out,
- The Casagrande method,
- The fall cone method.

The plastic limit is determined by rolling out, cf. Laboratoriehåndbogen (DGF's Laboratoriekomité, 2001). The liquid limit is determined by both the Casagrande method and the fall cone method, cf. Laboratoriehåndbogen (DGF's Laboratoriekomité, 2001). The classification parameters can be seen in Tab. 1.

Table 1: Classification parameters of the tertiary clay at Moesgaard Museum. The mean value is the arithmetic mean.

	w [%]	γ [kN/m^3]	w_P [%]	w_L [%]	I_P [%]
Results of boring records					
Min	19.0	18.8	28.0	91.0	53.0
Max	35.5	19.4	39.0	92.0	63.0
Mean	30.1	19.1	33.5	91.5	58.0
Results of laboratory test					
Min	32.8	18.1	34.5	95.0	60.0
Max	34.1	18.7	35.0	105.6	71.1
Mean	33.2	18.4	34.8	100.3	65.5

The results of the Casagrande method and the fall cone method differ about 2–9 %, cf. Fig. 5.

The liquid limit and the plasticity index determined by the two boring records and the mean values of the laboratory tests are plotted according to the Casagrande Chart, cf. Fig. 5.

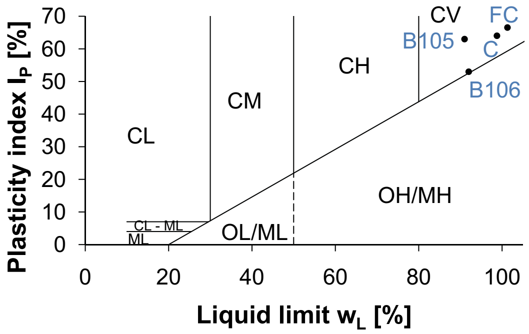

Figure 5: Classification of the tertiary clay at Moesgaard Museum according to the Casagrande Chart. *FC* is fall cone and *C* is Casagrande. According to Krebs Ovesen et al. (2007, Fig. 1.13).

Hence, the clay is characterised as very high plasticity clay marked by *CV* in the figure in accordance to the geological description of the boring records. For comparison, the classification of Søvind Marl and Little Belt Clay are plotted in Fig. 6 together with the tertiary clay at Moesgaard (Johannesen et al., 2008; Thøgersen, 2001).

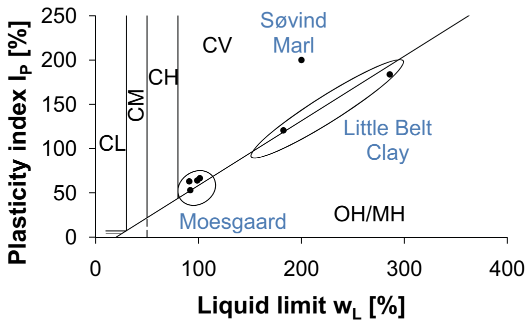

Figure 6: Classification of Søvind Marl, Little Belt Clay and the tertiary clay at Moesgaard Museum according to the Casagrande Chart. According to Krebs Ovesen et al. (2007, Fig. 1.13).

4 Consolidation Tests

The deformation properties of the tertiary clay at Moesgaard Museum are determined by two consolidation tests conducted according to the instructions in Laboratoriehåndbogen (DGF's Laboratoriekomité, 2001). The first consolidation test is conducted to determine the preconsolidation stress σ'_{pc} and the swelling pressure σ'_S. The second consolidation test is used to determine constrained modulus E_{oed}. However, various parameters can also be determined by the second consolidation test, e.g. σ'_{pc}, the creep strain ε_S, the coefficient of permeability k and the coefficient of consolidation c_k. Both consolidation tests are conducted by incremental loading.

In the first consolidation test, the initial dimensions of the specimen are $H = 2D = 60$ mm. The reason for the small dimensions are so that it is possible to apply stresses which are sufficiently large to observe a linear tendency of the primary path. The testing program consists of increasing loading to determine the virgin curve and one unloading step. Additionally, σ'_S is determined to be in the interval of the load step at which no swelling occurs and the step prior to this. The preconsolidation stress is preliminary estimated to 1120–1400 kPa corresponding to 4–5 times the vane shear strength c_v which is found to approximately 280 kPa at nearby depths (DGF's Laboratoriekomité, 2001).

To ensure that secondary consolidation has occurred, the duration of each load step is two–three days depending on the progress of the strain–time curve. For example, the strain–time curve for load step 6, which is an increase of the loading from 600 kPa to 1200 kPa, can be seen in Fig. 7. The duration of the load step is two days by which the progress of the secondary consolidation can be described by a linear regression line.

In the second consolidation test, the initial dimensions of the specimen are $H = 2D = 70$ mm. The testing program

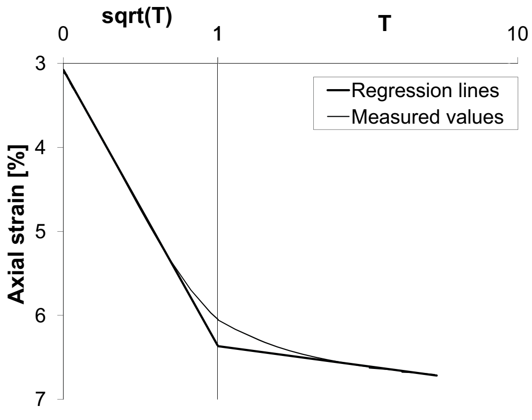

Figure 7: Strain–time curve for load step 6. $T = 1$ is after about 0.5 day and the final T is after two days.

consists of two times three unloading–reloading paths. The first three cycles include unloading to 650, 338 and 129 kPa, each followed by reloading to 1000 kPa. The next three cycles includes similar unloadings followed by reloading to 2000 kPa. After the last three cycles, the specimen is loaded to 4000 kPa and finally unloaded to zero loading.

The effective in situ stress is estimated to $\sigma_0' = 200$ kPa (COWI A/S, 2010).

4.1 Deformation Properties

The swelling pressure, the preconsolidation stress and the compression index are determined on the basis of the first consolidation test.

The swelling pressure σ_S' is determined as the load interval at which swelling no longer occurs. It is found to be between 150.8 and 303.7 kPa.

The preconsolidation stress σ_{pc}' is estimated by the Terzaghi method (Moust Jacobsen, 1993; Thøgersen, 2001). In Terzaghi's method, the primary path, i.e. the curved line, of the primary loading curve is used to determine σ_{pc}'. According to Terzaghi, the primary path can be described by Eq. 1.

$$\varepsilon = C_{c\varepsilon} \cdot \log\left(1 + \frac{\sigma'}{\sigma_\kappa'}\right) + \varepsilon_0 \qquad (1)$$

Where $C_{c\varepsilon}$ is the compression index, σ_κ' is a reference stress and ε_0 is the start value of the consolidation strain (not equal to the initial strain ε_i). The reference stress σ_κ' is the addition to σ' by which the primary path gets linear in a semi-logarithmic depiction, named the virgin curve, cf. Fig. 8.

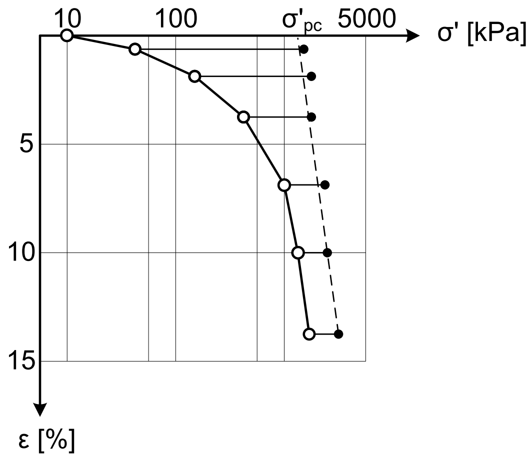

Figure 8: Determination of the virgin curve (the dashed line) used for Terzaghi's method.

σ_{pc}' is then estimated by Eq. 2.

$$\sigma_{pc}' \approx 2.0 \cdot \sigma_\kappa' \qquad (2)$$

The primary path and the virgin curve of the load-displacement curve can be seen in Fig. 9.

The preconsolidation stress is found to $\sigma_{pc}' = 1350$ kPa which implies an overconsolidation ratio of approximately $OCR = 6.8$. Additionally, estimations based on Akai's method, Janbu's method and the coefficient of consolidation, cf. Figs. 10–12, have been investigated leading to σ_{pc}' in the range $1200 - 1800$ kPa.

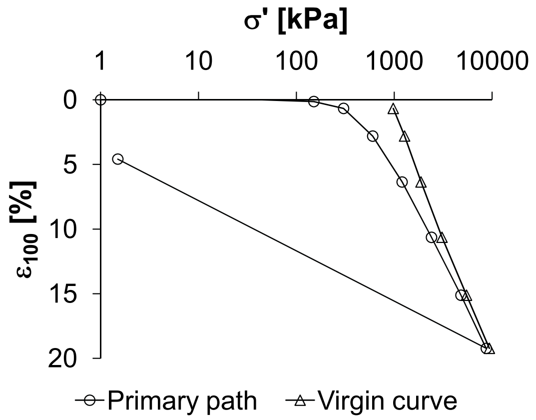

Figure 9: Primary path and virgin curve of the first consolidation test.

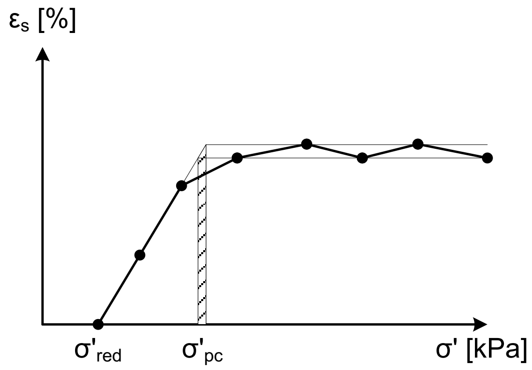

Figure 10: Akai's method of determining σ'_{pc}. σ'_{red} is the lowest effective stress of the soil. According to Moust Jacobsen (1993, Fig. 5.13).

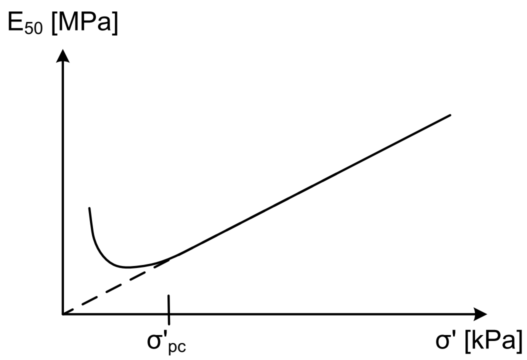

Figure 11: Janbu's method of determining σ'_{pc} based on the secant modulus E_{50}.

The compression index is determined as the slope of the linear part of the primary loading path as $C_{c\varepsilon} = 15.9$ %. For comparison, the compression index determined by the approximative approach in

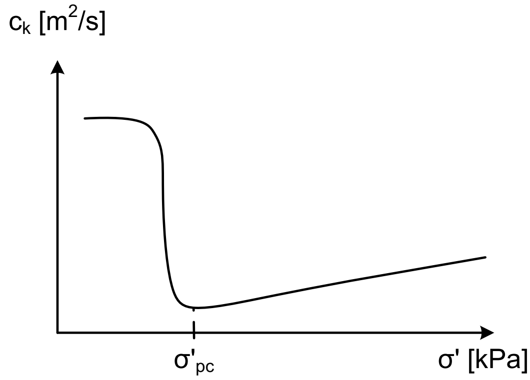

Figure 12: Determination of σ'_{pc} based on the coefficient of consolidation c_k. According to Thøgersen (2001, Fig. 5.9).

Eq. 3 for $w = 34.1$ % from the first consolidation test is found to $C_{c\varepsilon} = 12.3$ % (Steenfelt et al., 2007).

$$C_{c\varepsilon} = \frac{w - 25\ \%}{w + 40\ \%} \tag{3}$$

The primary path of the second consolidation test can be seen in Fig. 13.

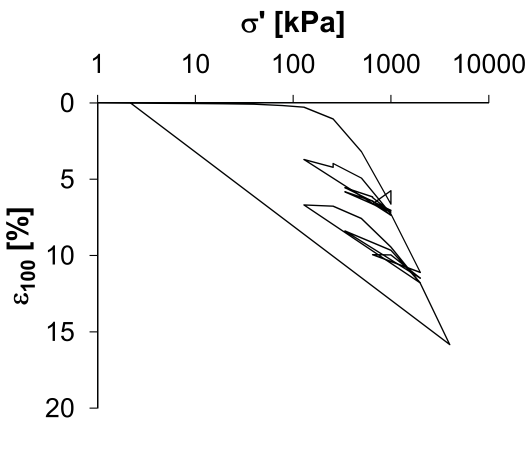

Figure 13: Primary path and unloading/reloading paths of the second consolidation test.

The constrained modulus is estimated based on the second consolidation test. It is determined by Eq. 4 according to Laboratoriehåndbogen (DGF's Laboratoriekomité, 2001).

$$E_{oed} = E_{oed,0} + \Delta E_{oed} \cdot \sigma'_{red} \tag{4}$$

Where $E_{oed,0}$ is the constrained modulus for zero effective stress, ΔE_{oed} the addition to E_{oed} per stress unit, and σ'_{red} the lowest effective stress of the soil. For each reloading path plotted in an (ε_{100}, σ')-diagram, the constrained modulus can be found as the reciprocal value of the initial slope. Plotting E_{oed} for each reloading path combined with σ'_{red} yields an estimate of Eq. 4.

The reloading paths can be seen in Fig. 15. As seen in the figure, the magnitude of E_{oed} for each reloading path depends on the choice of considered points. Choosing for example the first two points of the first reloading path for the second three cycles, i.e. (650 kPa, 9.9 %) and (999 kPa, 10.0 %), leads to E_{oed} = 1748 MPa, whereas the first and last point of this reloading path, i.e. i.e. (650 kPa, 9.9 %) and (1999 kPa, 11.5 %), leads to $E_{oed} = 87$ MPa. The constrained moduli for alle parts of the reloading paths are listed in Tab. 3.

As seen in Fig. 15 and Tab. 3, the constrained modulus depends significantly on the investigated stress level. Hence, multiple relations based on Eq. 4 for different stress levels could be a method of obtaining realistic values of E_{oed} in the future.

As an approximation of the constrained modulus, a linear regression between the first and last points of each reloading path is applied. This leads to the relation in Eq. 5 for the first three cycles and in Eq. 6 for the last three cycles, cf. Fig. 14.

$$E_{oed,1} = -3818 + 132 \cdot \sigma'_{red} \quad \text{[kPa]} \quad (5)$$
$$E_{oed,2} = 4560 + 128 \cdot \sigma'_{red} \quad \text{[kPa]} \quad (6)$$

In Eq. 5, the constrained modulus starts as a negative value. This is not realistic and Eq. 6 is used as an approximation of E_{oed}. For comparison, the constrained modulus of the tertiary Søvind Marl is approximately $E_{oed} = 10000 + 150 \cdot \sigma'_{red}$ (Johannesen et al., 2008).

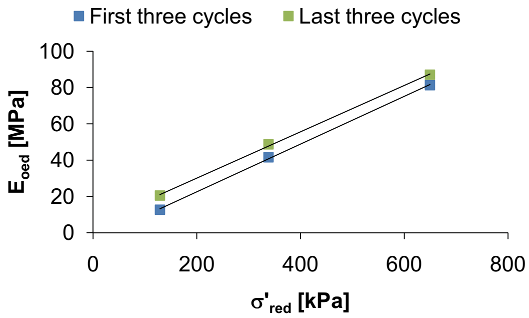

Figure 14: Initial slopes for the first three cycles and the last three cycles based on the first and last points of each reloading path.

The deformation parameters of the tertiary clay at Moesgaard Museum are listed in Tab. 2.

Table 2: Deformation parameters of the tertiary clay at Moesgaard Museum by the consolidation tests.

σ'_S	150.8-303.7 kPa
σ'_{pc}	1350 kPa
$C_{c\varepsilon}$	15.9 %
E_{oed}	$4560 + 128 \cdot \sigma'_{red}$ kPa

5 Triaxial Test

The strength properties of the tertiary clay at Moesgaard Museum are determined by a $CAU_{u=200}$ triaxial test with $H = D =$ 70 mm and smooth pressure cells. The test is conducted according to Laboratoriehåndbogen (DGF's Laboratoriekomité, 2001). The loading phase is conducted as an anisotropic loading controlled by the coefficient of horizontal earth pressure at rest K_0. The specimen is unloaded with constant K_0. The effective mean stress p' as a function of the deviatoric stress q can be seen in Fig. 16.

During the failure phase, the specimen is loaded to failure with a strain rate of 0.5 %/h.

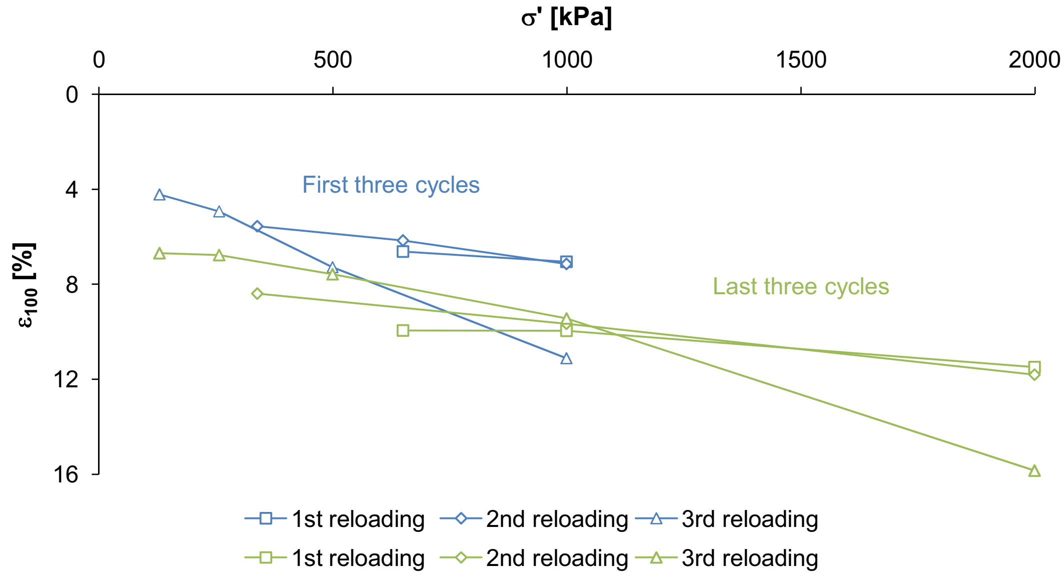

Figure 15: Reloading paths for the second consolidation test.

Table 3: Constrained moduli E_{oed} in MPa for all parts of the reloading paths.

	First three cycles			Last three cycles		
Points	1st reloading	2nd reloading	3rd reloading	1st reloading	2nd reloading	3rd reloading
1–2	81	52	18	1748	52	159
1–3		42	12	87	49	42
1–4			13			32
1–5						20
2–3		35	10	65	47	30
2–4			12			28
2–5						19
3–4			13			27
3–5						18
4–5						16

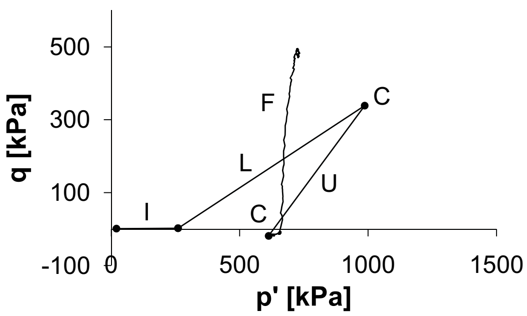

Figure 16: The effective mean stress p' as a function of the deviatoric stress q. I: Installation, L:Loading, C: Consolidation, U: Unloading, F: Failure.

The reader should be aware that the pore pressure was not measured during the test. This does, however, not have any significant consequence because the effective stress path and not the total stress path is followed during an undrained triaxial test, cf. Fig. 17. Problems appeared with the unloading equipment. This implied that the specimen was not unloaded to the desired stress level, cf. Fig. 18, and, thus, with a varying K_0.

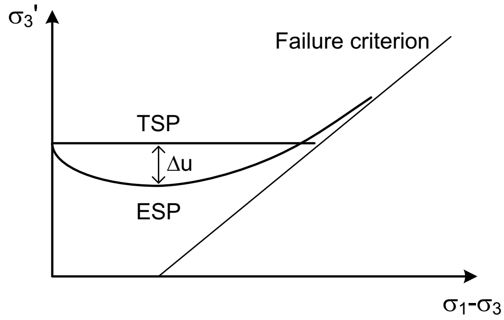

Figure 17: Total stress path (TSP), effective stress path (ESP) and failure criterion for undrained triaxial tests.

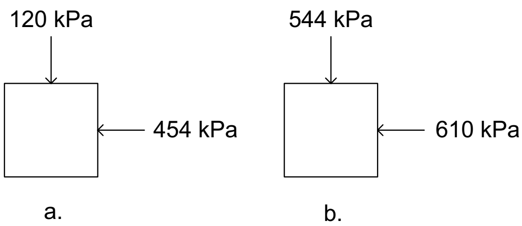

Figure 18: a. Desired stress levels after unloading. b. Actual stress levels after unloading.

5.1 Strength Properties

The axial strain ε_1 is plotted as a function of the effective deviatoric stress q' in Fig. 19.

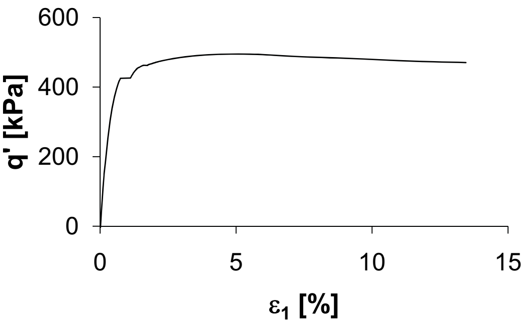

Figure 19: The axial strain as a function of the deviatoric stress.

The undrained shear strength c_u is determined as half the maximum deviatoric stress q, cf. Eq. 7.

$$c_u = \frac{q}{2} = \frac{\sigma_1 - \sigma_3}{2} \tag{7}$$

The maximum deviatoric stress for the failure criterion corresponding to 10 % additional strain is found to $q = 495$ kPa leading to $c_u = 247.5$ kPa. This corresponds well with the vane shear strength of $c_v = 280$ kPa which indicates a non-fissured clay.

For preconsolidated clays, the soil tends to dilate during loading. As seen in Fig. 20, this is not the case for the present triaxial test.

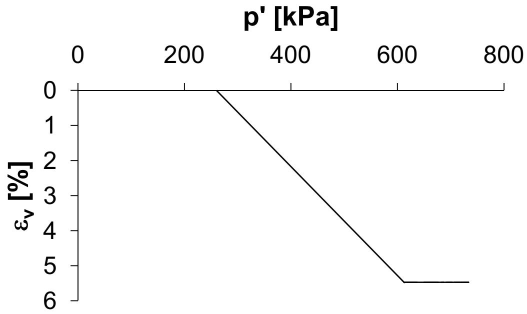

Figure 20: Volumetric strains ε_v as a functions of the effective mean stress p'.

The drained strength parameters c'_{tr} and φ'_{tr} are determined by Eqs. 8 and 9.

$$\sin \varphi'_{tr} = \frac{3}{1 + 6 \cdot \tan \alpha} \tag{8}$$

$$c'_{tr} = a \cdot \tan \alpha \cdot \tan \varphi'_{tr} \tag{9}$$

The parameters α and a are found by the failure criterion, cf. Fig. 21.

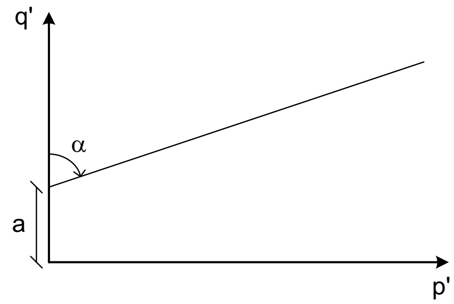

Figure 21: Failure criterion of triaxial test.

Because only a single failure point is found by the triaxial test, it is difficult to plot a unique failure criterion, cf. Fig. 21. Thus, both a solution including a cohesion defined by the parameter a

and a solution intersecting with $(0, 0)$ are used. This leads to $(c'_{tr}, \varphi'_{tr}) = (0, 17.8°)$ and $(c'_{tr}, \varphi'_{tr}) = (47.1 \text{ kPa}, 14.5°)$, respectively. It should be noted, though, that for the simple relation $c' = 0.1 \cdot c_u$, the value of $c'_{tr} = 47.1$ kPa implies $c_u = 471$ kPa. This value is almost twice the undrained shear strength determined earlier as $c_u = 247.5$ kPa. Hence, the second set of drained strength parameters including a cohesion is considered unrealistically high.

The strength parameters of the tertiary clay at Moesgaard Museum are listed in Tab. 4.

Table 4: Strength parameters of the tertiary clay at Moesgaard Museum by the triaxial test.

c_u	247.5 kPa
c'_{tr}	0 kPa/47.1 kPa
φ'_{tr}	17.8°/14.5°

6 Conclusions

The strength and deformation properties of the tertiary clay at Moesgaard Museum have been evaluated in this paper. The properties are found by means of two consolidation tests and one triaxial test.

The first consolidation test led to estimates on the swelling pressure, the preconsolidation stress and the compression index. The parameter were found to $\sigma'_S = 150.8 - 303.7$ kPa, $\sigma'_{pc} = 1350$ kPa and $C_{c\varepsilon} = 15.9$ %, respectively. The second consolidation test led to an estimate on the constrained modulus to $E_{oed} = 4560 + 128 \cdot \sigma'_{red}$ kPa.

The triaxial test provided the undrained shear strength and two sets of drained strength parameters: one set where the failure criterion intersects with $(0, 0)$, i.e. excluding a cohesion, and a set where a cohesion is included. The undrained shear strength was found to $c_u = 247.5$ kPa. The first set of drained strength parameters was found to $(c'_{tr}, \varphi'_{tr}) = (0, 17.8°)$. The second set was found to $(c'_{tr}, \varphi'_{tr}) = (47.1 \text{ kPa}, 14.5°)$. The drained shear strength of the second set is, however, considered high compared to the undrained shear strength when using the approximation $c' = 0.1 \cdot c_u$.

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