The Effects of Different Earth Pressure Coefficient at Rest in Triaxial Shear Tests on Clay

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AF263X Degree Project
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Preface

This Degree Project in Soil and Rock Mechanics is part of the Master's program in Civil and Architectural Engineering at The Royal Institute of Technology. It was carried out during the autumn semester of 2017 in cooperation with The Norwegian University of Science and Technology (NTNU) and the engineering consultancy company Multiconsult. The original idea for the project was proposed by Anders Gylland at Multiconsult after the author came in contact with the company and wanted to collaborate about a degree project with geotechnical laboratory work.

Trondheim, 2017-12-23

Jo Forseth Indgaard
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Last but not least I would like to thank fellow MSc student Pernille Rognlien for many valuable and educational discussions throughout the work with this thesis. It has been helpful to share different ponderings and frustrations with someone in a similar position.

J.F.I.
Abstract

Triaxial shear test is the most accurate test for deciding the undrained shear strength of clay. In every test the ratio between the horizontal and vertical stresses, the coefficient of earth pressure at rest ($K'_0$), has to be decided. It's widely believed that the choice of this parameter will influence the results, but it's not known to what extent.

In this thesis 20 consolidated undrained active triaxial shear tests on clay has been conducted with a $K'_0$ at 0.6 and 0.8. The clay was collected with a 54 mm piston sampler at the Norwegian Geo-Test Site in Trondheim, Norway, on depth of 3.0 to 7.8 meters. Besides the triaxial testing, index tests and oedometer tests was conducted on every cylinder to do a general classification of the clay. The clay has a sensitivity of 9-20, a water content of 35-51 %, a plasticity index of 27-65 % and an over consolidation ratio of 2.6-6.8.

The consolidation phase of the triaxial test was conducted in five loading steps with a rest time in-between to monitor the amount of pore water expelled at each stress level. The loading steps was 50 %, 75 % and 100 % of maximum cell pressure and thereafter at 50 % and 100 % of the maximum deviator stress. The shear phase was done at a speed of 1.5 % per hour to a total of 10 % axial strain.

It is not possible to reach an unambiguous conclusion from the results, but the maximum shear strength of tests with a $K'_0$ at 0.8 is 17 % higher, while the total amount of pore water extortion is equal between the two values. The amount of creep in the latest steps is on the other hand smaller for a $K'_0$ at 0.8. This indicates that the soil is handling the stress level better than with a $K'_0$ at 0.6.
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\( w \) Water content %
\( w_L \) Liquid limit %
\( w_p \) Plastic limit %

**Greek Symbols**

\( \Delta \sigma_r \) Change in cell pressure in triaxial cell \( kPa \)
\( \Delta u \) Change in pore pressure in triaxial cell \( kPa \)
\( \Delta V \) Change in volume of triaxial test specimen during consolidation \( cm^3 \)
\( \Delta V(\Delta \sigma') \) Change in volume of triaxial test specimen caused by increase in stress \( cm^3 \)
\( \Delta V(\Delta t') \) Change in volume of triaxial test specimen caused by creep \( cm^3 \)
\( \delta \) Deformation \( mm \)
\( \epsilon \) Strain %
\( \epsilon_{axial} \) Axial strain %
\( \gamma \) Unit weight \( kN/m^3 \)
\( \phi \) Friction angle °
\( \rho_s \) Density of soil grains \( t/m^3 \)
\( \sigma'_1 \) Major principal effective stress \( kPa \)
\( \sigma'_2 \) Middle principal effective stress \( kPa \)
\( \sigma'_3 \) Minor principal effective stress \( kPa \)
\( \sigma_d \) Deviatoric stress \( kPa \)
\( \sigma_h \) Total horizontal stress \( kPa \)
\( \sigma'_h \) Effective horizontal stress \( kPa \)
\( \sigma_v \) Total vertical stress \( kPa \)
\( \sigma'_v \) Effective vertical stress \( kPa \)
\( \sigma'_{v0} \) Effective vertical overburden stress \( kPa \)
\( \tau_{\epsilon 2\%} \) Shear strength at 2% strain \( kPa \)
\( \tau_{max} \) Maximum shear strength \( kPa \)

**Acronyms**

*NGI* the Norwegian Geotechnical Institute

*NGS* Norwegian Geotechnical Society

*NGTS* Norwegian Geo-Test Site

*NTNU* Norwegian University of Science and Technology
\textbf{OC} Over consolidated

\textbf{OCR} Over consolidation ratio

\textbf{SVV} Public Roads Administration (Statens vegvesen)
Chapter 1

Introduction

1.1 Background

To find the undrained shear strength of a soil the use of triaxial shear test is considered to be the most accurate method. It is commonly used and the results is important for the evaluation of the soil properties and behaviour. When a triaxial test is carried out the overburden effective pressure ($\sigma'_{vo}$) and earth pressure coefficient ($K'_{0}$) is the only parameters that usually is varied between each test, in addition to what type of triaxial test.

The overburden pressure is fairly easy to evaluate based on the weight of the overburden soil and the hydrostatic conditions, while the $K'_{0}$ is harder to determine. The chosen $K'_{0}$ is mostly based on a gut-feeling from the engineer that booked the test. This gut-feeling can in some cases be backed by different correlations models, but it’s in most cases just based on experience. Even though the engineers use time and carefully consider what $K'_{0}$ to chose, I have not been able to find any studies to prove that it matters at all.

In the biggest annual conference about geotechnical engineering in Norway in the autumn of 2017, one of the themes was how to chose the right $K'_{0}$ (L’Heureux et al. [2017]). A regression analysis had been done to bring forward new formulas that can be used to predict $K'_{0}$ based on other soil parameters. One of the recommendations after this report was to investigate the impact of $K'_{0}$ on the behavior of clay material in triaxial tests.

The most important part of this study is to investigate how the sample is handling the different stress states. It’s believed that this is connected to the amount of pore water extorted from the sample. In the triaxial tests conducted in this thesis continuous monitoring of pore
CHAPTER 1. INTRODUCTION

Water extortion will be utilized to investigate the impact different stress states has on the soil behaviour.

The soil used for these triaxial tests is from the newly created test site in Trondheim called The Norwegian Geo-Test Sites (NGTS). Besides the investigations on the effect of different $K'_0$ in triaxial tests, another objective is the contribution of data for the characterization of this clay.

1.2 Objectives

The main objectives of this project can be summarized as the following:

1. To investigate the impact of different $K'_0$ in undrained active triaxial shear tests.

2. To investigate when in the consolidation phase the sample get disturbed by monitoring the extortion of pore water.

3. To contribute with data to the Norwegian Geo-Test Sites.

1.3 Limitation

In the choice of doing a few triaxial shear tests on many different $K'_0$ and making many tests on two different $K'_0$ the latter was chosen. It would be interesting to see the effect of a larger variation of $K'_0$, but in this thesis a prioritization to focus on just two was made.

1.4 Approach

First, a literature study was conducted to find relevant literature. Relevant sources were found and the laboratory work was carried out. The data was processed, and this report written.

1.5 Structure of the report

This report consists of four main parts. First the theoretical background for the different concepts is presented together with a description of the test site. Then the laboratory work is described and the calculation methods presented, before the results is presented. The report concludes with a discussion leading to a conclusion.
Chapter 2

Background

2.1 Definition of the Coefficient of Earth Pressure at Rest

The term coefficient of earth pressure at rest, $K_0$, is defined as the relationship between the horizontal stresses $\sigma_h$ and the vertical stresses $\sigma_v$, as seen in equation 2.1:

$$K_0 = \frac{\sigma_h}{\sigma_v}$$  \hspace{1cm} (2.1)

To obtain the actual effective relationship the in situ pore pressure $u$ is subtracted and the effective coefficient of earth pressure at rest $K_0'$ is obtained.

$$K_0' = \frac{\sigma_h - u}{\sigma_v - u} = \frac{\sigma_h'}{\sigma_v'}$$  \hspace{1cm} (2.2)

Most empirical equation leads to $K_0$. To find $K_0'$ equation 2.1 and 2.2 is combined and equation 2.3 is obtained.

$$K_0' = \frac{(K_0 \cdot \sigma_v) - u}{\sigma_v - u}$$  \hspace{1cm} (2.3)

2.2 Triaxial Testing

In the triaxial cell the minor effective stresses, $\sigma_h' = \sigma_2' = \sigma_3'$, is related to the major effective stress, $\sigma_v' = \sigma_1'$ as following based on equation 2.2:

$$\sigma_3' = \sigma_v = K_0' \cdot \sigma_1'$$  \hspace{1cm} (2.4)
CHAPTER 2. BACKGROUND

The deviator stress, $\sigma_d$, is defined as the difference between $\sigma'_1$ and $\sigma'_3$ as seen in equation 2.5:

$$\sigma_d = \sigma'_1 - \sigma'_3$$  \hspace{1cm} (2.5)

The average effective stress, $p'$, is calculated with equation 2.6. With the connection presented it’s easy to see that a lower $K'_0$ also represent a lower average stress level for the specimen.

$$p' = \frac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3} = \frac{\sigma'_1 + 2\sigma'_3}{3} = \frac{\sigma'_1 + 2K'_0 \cdot \sigma'_3}{3}$$  \hspace{1cm} (2.6)

2.3 Correlation Methods Between Soil Parameters and $K'_0$

To accurately predict $K'_0$, extended research has been done through the years. Numerous empirical formulas is proposed on both normal consolidated and over consolidated clay. Two of the most widely used correlations for guidance when choosing $K'_0$ for OC-clay for triaxial tests at Multiconsult (Gyllan [2017]) is seen in figure 2.1 and 2.2. In addition many formulas from many different researchers exists, as for example Schmidt [1966], Mayne and Kulhawy [1982], Hamouche et al. [1995] and Fioravante et al. [1998].

In November 2017 L’Heureux et al. [2017] presented similar work based on data collected on Norwegian clay. This regression analysis lead to two new equation (equation 2.7 and 2.8) that is recommended for use on on clay from Norway.

Figure 2.1: Relationship between $K'_0$, $I_p$ and OCR by Brooker and Ireland [1965]
2.3. CORRELATION METHODS BETWEEN SOIL PARAMETERS AND $K'_0$

Figure 2.2: Relationship between $K'_0$, $\phi'$ and OCR by Brooker and Ireland [1965]

$$K_0 = 0.48 \cdot I_p^{0.03} OCR^{0.47}$$  \hspace{1cm} (2.7)

$$K_0 = 0.53 \cdot OCR^{0.47}$$  \hspace{1cm} (2.8)

Where $I_p$ is the plasticity index and OCR the over consolidation ratio.

These equations together with figure 2.1 and 2.2 is used to see what results these relationships gives on the clay treated in this thesis. For each relation and sample cylinder the value for $K_0$ is found and transformed to $K'_0$ by equation 2.3. These results, together with in-data, is presented in table 2.1.

Table 2.1 indicates that both 0.8 and 0.6 would have been a reasonable choice for $K'_0$ on this specific clay. On the other hand, a $K'_0$ at 0.6 is based on talks with the staff at Multiconsult

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<td>7-7.8</td>
<td>8</td>
<td>3.5</td>
<td>30</td>
<td>0.5</td>
<td>0.6</td>
<td>0.9</td>
<td>0.7</td>
<td>0.7</td>
</tr>
</tbody>
</table>

Table 2.1: Suggestion for $K'_0$ based on proposal from Brooker and Ireland [1965] and L’Heureux et al. [2017].
(Gyllan [2017]) seen as too low. The concern is that the higher shear stress would cause the soil specimen to yield even before the shearing phase has started. If the specimen yields this should cause significant increase in pore water extortion according to section 2.4.

2.4 Sample Quality

To decide if the quality of the conducted tests is adequate, an investigation of the shape of the stress curves can be informative. An illustration of the difference between good and bad samples is seen in figure 2.3. Besides these non-quantifiable properties the amount of extorted water from the sample and the B-value can tell more about the quality. They are further explained in this section.

The change in void ratio \(e_0\) is another way used to decide the quality of the test conducted (Lunne et al. [1997]). The numbers needed to calculated \(e_0\) was just collected experimentally on a few tests in this thesis and are therefore not further explained in this text.

Water Extortion

In the consolidation phase pore water is extorted from the sample. The amount of pore water is monitored through the burette and logged as all other parameters. The amount of pore water expelled depends on the type of soil, type of consolidation, the consolidation stress level, the amount of sample disturbance and the duration of the consolidation phase (Berre [1985]).

![Figure 2.3: The difference between good quality (250 mm block) and worse quality (54 mm). From NGS [Published 1997, edited 2013].](image)
In this work the soil has fairly similar properties, the consolidation is done in the same manner with the same consolidation time, while the consolidation stress level alternates with different $K_0'$. The different soil samples is taken from the same cylinder, and should therefore be equally disturbed before the pressure in the triaxial cell was applied. With this set-up it’s reasonable to assume that the effect of the way the stress was applied (see section 3.4.2) would be the main cause of the varying amount of pore water expelled.

Berre [1985] came up with quality criteria based on the volumetric strain, $\epsilon_{vol}$ (eq. 2.9) and over consolidation ratio ($OCR$). His criteria is that $\epsilon_{vol}$ under 1.0 % if the OCR is 1-2 is acceptable quality, while $\epsilon_{vol}$ at 2-3 % is acceptable if the OCR is 2-3.

$$\epsilon_{vol} = \frac{\Delta V}{V_0}$$  \hspace{1cm} (2.9)

Where $\Delta V$ is the total amount of pore water extorted and $V_0$ the initial specimen volume.

**B-test**

To investigate how well the system was saturated (Skempton [1984]) the pore-pressure coefficient B, or B-value, was found with a B-test. The B-test was done about one hour after the back pressure was applied. After the burette was closed and the pressure in the sample had stabilized $u$ and $\sigma_3$ was documented. The cell pressure was then increased with 10 kPa and after 2 minutes $u$ was recorded again, and $\sigma_3$ lowered back to the same level as before the B-test. The B-value was calculated with the formula 2.10.

$$\text{B-value} = \frac{\Delta u}{\Delta \sigma_r}$$  \hspace{1cm} (2.10)

According to SVV [2014] a B-value over 0.95 is a sign of a well saturated system, while a value between 0.80 and 0.95 is acceptable.

## 2.5 Time Resistance Concept

According to basic consolidation theory the consolidation of soil can be divided in two parts (see figure 2.4). The deformation occurring first, called primary consolidation, and the secondary consolidation dominated by creep. In the triaxial tests this concept should be valid.
even though the time span is much shorter than what is usually the case when creep is dis-
cussed. The part in the figure marked period of pore pressure dissipation corresponds to what
in this thesis will be called Change in volume of triaxial test specimen caused by increase in
stress, $\Delta V(\Delta \sigma')$. While the part marked no excess pore pressure corresponds to change in vol-
ume of triaxial test specimen caused by creep, $\Delta V(\Delta t')$.

To make it possible to quantify the amount of primary and secondary settlements in each
loading step the time resistance concept according to Janbu [1969] is introduced. A resis-
tance is defined as given incremental cause divided by incremental measured effect. In this
case the cause is time and the effect deformation, or more precise volume change. Based on

Figure 2.5: Definition of the resistance of a material by Janbu [1985].
2.5. TIME RESISTANCE CONCEPT

the illustration made by Janbu [1969], this concept is applied as seen in figure 2.6.

For each step \( n \) a tangent of the curve closest to the next step is is drawn. The incline of this tangent represent the resistance \( R = \frac{dT}{d\epsilon} \). The intersection between this tangent and the vertical line from where the current loading step starts indicates the size of \( \Delta V(\Delta \sigma') \) and \( \Delta V(\Delta t') \).

The amount of creep, or size of \( R \), is believed to be connected to how well the soil is handling the current stress level, especially in overconsolidated clays. In figure 2.7 this is illustrated by the differences of \( R \) between disturbed (2) and non-disturbed (1) far from the preconsolidation stress \( p_c' \).

For the tests conducted with a step time of 60 minutes the concept is applied in the the same manner but the tangent is drawn at 30 minutes after the last increase in \( \sigma_3 \).

Figure 2.7: The differences of R over the \( p' \). (1) and (2) indicates low subsequently high disturbance. [adapted from Geotechnical-Division, 2015]
2.6 Test Site

Norwegian Geo-Test Sites (NGTS) is a research project led by the Norwegian Geotechnical Institute (NGI) which consist of five different test sites around Norway with different soil types (NGI [2017]). The Trondheim site represent the quick clay site, while the other sites are for soft clay, silt, sand and permafrost. The test sites are used as field laboratories for development, verification and testing for site investigations and testing procedures. The site was opened in June 2016 and is part of the International Geo-Test Sites Network. As part of the early work a precise soil characterization is important and that is what this thesis will contribute with.
The test site is located at Flotten about 10 km south of Trondheim. The location of the site can be seen in figure 2.8 together with a quaternary geology map in figure 2.9. As seen on these maps the Flotten test site is situated within an area with thick marine deposits where the old sea floor is basically intact. However, the site is not far from the dark blue color which indicates eroded areas. These areas are affected by erosion caused by the river Nideleven which is seen running south-north on the map.

Piezometers were installed on the site (at 10 and 15 meters depth) not far from where the soil was retrieved. They were installed in July and registered data every third hour. After the first week the shallowest piezometer showed a stable pore pressure at about 155 kPa. The ground water level was found at 1.4 meters depth. These values together with the calculated densities from the laboratory tests formed the basis for the soil profile that was used to calculate $\sigma'_{v0}$ for the triaxial tests. Because the soil profile was updated after a new laboratory tests was completed, two tests at the same depth from the first and second hole would not have the exact same $\sigma'_{v0}$. 

Figure 2.9: Quaternary geology map. Light blue colour represent old seabed, while darker blue colour represent eroded marine deposits. The red arrow indicates the location of NGTS. Figure from Reite et al. [1999].
Chapter 3

Laboratory Investigations

The laboratory investigations was done at the geotechnical laboratory at NTNU Gløshaugen. After the first three cylinders was tested in a manner that differed a bit from cylinder to cylinder a clearer procedure was found and followed thereafter. This procedure is described in the next section. Even though the procedure was mainly the same after the first three cylinders the operator got more experience for each completed test. It’s therefore reasonable to believe that the quality of the tests also improved and that the most reliable results came towards the end of the work.

3.1 Test Procedure

The soil specimens was extracted from the ground by the laboratory engineers at NTNU Karl Ivar Volden Kvisvik and Espen Andersen. They made the samples on demand and the goal was to do the laboratory investigations as soon as possible after the samples was extruded, within one week. After extrusion the samples was handled with care and stored in a refrigerated room with a temperature of 4 °C.

Before the sample was extruded from the cylinder the triaxial apparatus was made ready in a way that made it possible to build in a sample in each of the triaxial apparatus in under 15 minutes after the cylinder was extruded. After the extrusion the sample was visually examined and what part to use for what test was decided. Three clay samples for deciding the water content was cut off and weighted before the clay for the triaxial tests was separated from the rest of the sample. While the soil for the triaxial test was built in, the rest of the sample was covered with plastic. After the consolidation of the triaxial tests was started,
the sample for the odometer test was built in and the test was started. Thereafter the index
tests followed in this order: falling cone, Attenberg, density, salinity, grain density, grain size
distribution and organic content.

3.2 Index Testing

As part of NGTS a general classification of the soil was completed. The index test has mainly
followed the recommendations from the handbook R210 by the Norwegian Public Roads
Administration (SVV [2014]). Any special procedures not following this recommendation, or
other specialties, is mentioned in the following section.

3.2.1 Density

The density was calculated for the whole cylinder, for each of the triaxial test specimen, and
by a small density ring. The average value is the one that is presented in the results.

3.2.2 Water Content

The procedure for finding the water content followed the recommendation except that the
cooling before the second weighing was not done in a desiccator. Three small samples was
dried, from the top, bottom and middle. The average value of these three are presented.

3.2.3 Attenberg Limits

The Attenberg limits was decided by Casagrande and the rolling out a thread-method.

3.2.4 Fall Cone

Normal fall cone method was used for deciding undrained shear strength, remoulded shear
strength and sensitivity.

3.2.5 Salinity

The salinity was calculated based on the electric conductivity of the pore water extorted by a
centrifuge.
CHAPTER 3. LABORATORY INVESTIGATIONS

3.2.6 Grain Size Distribution

The grain size distribution was decided by hydrometer analysis. This analysis was conducted with procedures from NTNU that differs from the R210, but not in any significant manner. Because of the very high clay content and not very accurate hydrometer readings, the results presented are clipped in the beginning until the change from reading to reading was significant.

3.2.7 Grain Density

The grain density was decided with newly calibrated pycnometers. For each cylinder three tests was conducted. The average values is presented in the results.

3.2.8 Organic Content

The same sample used for water content was grounded after the water content was calculated and used for loss on ignition.

3.3 Oedometer

To decide $p_c$ and OCR of the soil at least one oedometer tests was conducted for every cylinder. The tests was conducted on two different apparatuses at the laboratory at NTNU and was run over night.

The test equipment log $\delta$, $\sigma'_v$, and $u$ every 5th second. This leads to a very large dataset with scattered results. To smooth out the data a moving average of 100 elements was conducted before every data point except every 16th was deleted before further calculations. The calculation was conducted in Matlab.

First an approximation of the average effective stress calculated using equation 3.1:

$$\sigma'_v = \sigma_v - \frac{2}{3} u_b$$  \hspace{1cm} (3.1)

Where $\sigma_v$ is the vertical stress and $u_b$ is the pore pressure at the sample base.

The strain was calculated with equation 3.2:

$$\epsilon = \frac{\delta}{h_0}$$  \hspace{1cm} (3.2)
Where $\delta$ is the deformation and $h_0$ the initial specimen height at 20 mm.

The modulus $M$, was calculated based on the differences between two consecutive data points as seen in equation 3.3:

$$M = \frac{\sigma_{vi} - \sigma_{vi-1}}{\epsilon_i - \epsilon_{i-1}}$$  \hspace{1cm} (3.3)

Finally the coefficient of consolidation $c_v$ was calculated using equation 3.4:

$$c_v = \frac{\sigma_{vi} - \sigma_{vi-1} \left[ h_0(1-\epsilon) \right]^2}{t_i - t_{i-1} \frac{2u_b}{}}$$  \hspace{1cm} (3.4)

### 3.4 Triaxial Shear Test

To make it possible to compare the effect of different $K'_0$ it was attempted to keep all other parameters as constant as possible. The procedure for building in the sample, together with the procedures throughout the test was completed in a similar manner. These procedures are further explained in the following section.

Side drain paper soaked in distilled water was used on all tested samples to increase the speed of the consolidation.

#### 3.4.1 Build-in

First the porous discs and the distilled water used for flushing was put under vacuum. The triaxial apparatus was prepared with flushing of deaired water through the system. The membrane stretcher was closed with electrical tape and a rubber membrane was installed.
Figure 3.2: Diagrammatic layout of the triaxial apparatus (Alan Bishop and Henkel [1957]).

with four o-rings. The desired $\sigma'_v$ and $\sigma'_h$ was decided. The most recent soil profile was used to calculate $\sigma'_v$ based on the density found in the overlying cylinders. The sample was put in the cutting crib, trimmed to exactly 10 cm and weighted, before the sample was placed on the pedestal (see figure 3.1) in the same direction as in the ground.

A wetted filter paper was wrapped around the sample before the rubber membrane was carefully put in place. The cell was installed and tightly screwed before the water level was raised to the middle of the sample. With water at this level all pressure sensors was zeroed out. Further on the cell was filled to the top with water and oil, the loading ram was put in place (see figure 3.2), and the load cell was put in contact with the top cap. To flush out any air in the system a cell pressure of about 10 kPa was applied before deaired water was sent through the top and bottom porous discs.

### 3.4.2 Consolidation Phase

The stress was applied in five steps to be able to evaluate where in the consolidation phase the soil sample get to a stress situation it can't handle. The rest time was usually 30 minutes, but 60 minutes was experienced with in some cases to see the effect of a longer consolidation
3.4. TRIAXIAL SHEAR TEST

phase. The steps was at 50 %, 75 % and 100 % of maximum $\sigma_3$, and 50 % and 100 % of the maximum $\sigma_d$. The application of pressure at each step was done over about 30 seconds. The loading steps is illustrated through the stress path in a NTNU-plot seen in figure 3.3.

In the consolidation phase the level in the burette was monitored. The level in the burette represent the amount of pore water that is expelled from the sample. On the later triaxial shear tests $\delta$ was even monitored in the isotropic consolidation phases. This was done with requiring a $\sigma_d$ at 1 kPa. In this way the step-wise motor would seek contact and monitor $\delta$ in the process.

**Back Pressure**

To make the system more saturated a back pressure was applied after the consolidation phase. The back pressure was in most cases set to 300 kPa and applied over about 30 minutes, in shorter steps. When the back pressure was applied the $\sigma_3'$ was watched and kept close to the level the sample was consolidated to.
3.4.3 Shearing Phase

The shearing phase was started about one hour after the B-test was finished. The speed was constant for all tests at 1.5% \( \epsilon \) per hour.

After the shearing was finished the sample was built out, and dried before it was weighted to find the water content.

3.4.4 Different Triaxial Apparatuses

The tests conducted for this thesis was done on two different triaxial apparatuses, called \textit{Triax 1} and \textit{Triax 2}. Both triaxial apparatuses is similar in appearances and use, but they have some minor special characteristics that could lead to systematic errors. To reduce this effect different \( K'_0 \) was used in both apparatuses.

3.4.5 Calculations

The calculations done on the raw-data is presented in the following section.

Correction for change in area after consolidation was done:

\[
A_a = A_0 \left( 1 - \frac{\Delta V}{V_0} \right) / \left( 1 - \frac{\Delta V}{3V_0} \right)
\]  

(3.5)

\( A_a \) = specimen area after consolidation
\( A_0 \) = build-in specimen area, 23.2 \( cm^2 \)
\( h_0 \) = height of sample at build-in, 100 mm
\( V_0 \) = volume at build-in, \( V_0 = A_0 \cdot h_0 = 232 \ cm^2 \)
\( \Delta V \) = change in volume after consolidation (extorted pore water)

In the shearing phase \( \delta \) is recorded and \( \epsilon_{axial} \) is calculated as following:

\[
\epsilon_{axial} = \frac{\delta}{h_0}
\]  

(3.6)

Correction for change in area during shear phase is included as following:

\[
A_s = A_a (1 - \epsilon_{axial})
\]  

(3.7)
3.4. TRIAXIAL SHEAR TEST

From the NTNU-plot a failure line can be drawn that corresponds with the different tests. From this line the friction angle \( \phi \), is found with equation 3.8 and attraction \( a \) is found where this line intersect with the x-axis.

\[
\tan \phi = \frac{S_f}{\sqrt{1 + 2S_f}} \tag{3.8}
\]

The effects of filter paper, rubber membrane, change in volume of triaxial cell during application of cell pressure, and the submerged eigenweight of the loading piston is assumed to be negligible.
Chapter 4

Results

In this chapter summary plots are presented. The results from each of the triaxial and oedometer tests are found in appendix A and B.

4.1 Triaxial Tests

The data from the triaxial tests are presented in the following section.

4.1.1 Overview

In table 4.1 an overview over every triaxial test that was conducted is found.

<table>
<thead>
<tr>
<th>Depth</th>
<th>Apparatus</th>
<th>$K_0$</th>
<th>$\sigma'_V$ [kPa]</th>
<th>$\Delta V$ [cm$^2$]</th>
<th>$\tau_{max}$ [kPa]</th>
<th>$\tau_{2%}$ [kPa]</th>
<th>$\tau_{max}$/$\tau_{2%}$ [%]</th>
<th>$\sigma'<em>V$/$\tau</em>{2%}$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-1.1</td>
<td>Triax1</td>
<td>0.8</td>
<td>50.1</td>
<td>4.1</td>
<td>49</td>
<td>47</td>
<td>96</td>
<td>0.97</td>
</tr>
<tr>
<td>2-1.2</td>
<td>Triax1</td>
<td>0.6</td>
<td>45.5</td>
<td>2.4</td>
<td>42</td>
<td>42</td>
<td>100</td>
<td>0.93</td>
</tr>
<tr>
<td>2-2.1</td>
<td>Triax1</td>
<td>0.8</td>
<td>51.9</td>
<td>6.9</td>
<td>41</td>
<td>40</td>
<td>99</td>
<td>0.78</td>
</tr>
<tr>
<td>2-2.2</td>
<td>Triax1</td>
<td>0.6</td>
<td>54.7</td>
<td>3.7</td>
<td>42</td>
<td>42</td>
<td>99</td>
<td>0.77</td>
</tr>
<tr>
<td>2-3.1</td>
<td>Triax1</td>
<td>0.8</td>
<td>58.9</td>
<td>8.1</td>
<td>40</td>
<td>40</td>
<td>99</td>
<td>0.68</td>
</tr>
<tr>
<td>2-3.2</td>
<td>Triax2</td>
<td>0.6</td>
<td>59.9</td>
<td>5.3</td>
<td>37</td>
<td>37</td>
<td>98</td>
<td>0.62</td>
</tr>
<tr>
<td>2-4.1</td>
<td>Triax1</td>
<td>0.8</td>
<td>67.2</td>
<td>-</td>
<td>40</td>
<td>40</td>
<td>99</td>
<td>0.59</td>
</tr>
<tr>
<td>2-4.2</td>
<td>Triax2</td>
<td>0.6</td>
<td>69.0</td>
<td>4.9</td>
<td>37</td>
<td>37</td>
<td>98</td>
<td>0.54</td>
</tr>
<tr>
<td>2-5.1</td>
<td>Triax1</td>
<td>0.8</td>
<td>75.3</td>
<td>7.2</td>
<td>60</td>
<td>49</td>
<td>82</td>
<td>0.79</td>
</tr>
<tr>
<td>2-5.2</td>
<td>Triax2</td>
<td>0.6</td>
<td>77.8</td>
<td>5</td>
<td>46</td>
<td>40</td>
<td>86</td>
<td>0.59</td>
</tr>
<tr>
<td>3-1.1</td>
<td>Triax1</td>
<td>0.8</td>
<td>47.4</td>
<td>4.1</td>
<td>42</td>
<td>42</td>
<td>98</td>
<td>0.89</td>
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<tr>
<td>3-1.2</td>
<td>Triax2</td>
<td>0.6</td>
<td>46.2</td>
<td>-</td>
<td>34</td>
<td>27</td>
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<td>0.74</td>
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<tr>
<td>3-2.1</td>
<td>Triax1</td>
<td>0.6</td>
<td>54.8</td>
<td>6.6</td>
<td>30</td>
<td>28</td>
<td>94</td>
<td>0.55</td>
</tr>
<tr>
<td>3-2.2</td>
<td>Triax2</td>
<td>0.8</td>
<td>57.0</td>
<td>4.7</td>
<td>46</td>
<td>46</td>
<td>98</td>
<td>0.81</td>
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<tr>
<td>3-3.1</td>
<td>Triax1</td>
<td>0.6</td>
<td>66.2</td>
<td>5.9</td>
<td>35</td>
<td>34</td>
<td>99</td>
<td>0.52</td>
</tr>
<tr>
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<td>Triax2</td>
<td>0.8</td>
<td>69.6</td>
<td>5.3</td>
<td>44</td>
<td>43</td>
<td>96</td>
<td>0.64</td>
</tr>
<tr>
<td>3-4.1</td>
<td>Triax1</td>
<td>0.6</td>
<td>77.2</td>
<td>6.2</td>
<td>38</td>
<td>37</td>
<td>99</td>
<td>0.49</td>
</tr>
<tr>
<td>3-4.2</td>
<td>Triax2</td>
<td>0.8</td>
<td>80.6</td>
<td>5</td>
<td>54</td>
<td>51</td>
<td>94</td>
<td>0.67</td>
</tr>
<tr>
<td>3-2.1X</td>
<td>Triax1</td>
<td>0.6</td>
<td>59.7</td>
<td>4.7</td>
<td>48</td>
<td>47</td>
<td>99</td>
<td>0.80</td>
</tr>
<tr>
<td>3-1.2X</td>
<td>Triax2</td>
<td>0.6</td>
<td>49.1</td>
<td>2.7</td>
<td>39</td>
<td>35</td>
<td>90</td>
<td>0.79</td>
</tr>
</tbody>
</table>
4.1. TRIAXIAL TESTS

4.1.1 Table 4.2: Average numbers from table 4.1

<table>
<thead>
<tr>
<th>$K'_0$</th>
<th>$\tau_{max}$ [kPa]</th>
<th>$\tau_{\epsilon 2%}$ [kPa]</th>
<th>$\frac{\tau_{max}}{\tau_{\epsilon 2%}}$ [%]</th>
<th>$\frac{\sigma'<em>{V0}}{\tau</em>{max}}$ [-]</th>
<th>$\phi$ [$^\circ$]</th>
<th>a [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.6</td>
<td>40.3</td>
<td>38.8</td>
<td>96.5 %</td>
<td>0.67</td>
<td>30</td>
<td>-12</td>
</tr>
<tr>
<td>0.8</td>
<td>47.1</td>
<td>45.5</td>
<td>96.6 %</td>
<td>0.80</td>
<td>30</td>
<td>-14</td>
</tr>
</tbody>
</table>

Figure 4.1: NTNU-plot of every test with $K'_0$ at 0.6. Curves clipped after yielding. A failure line is drawn with corresponding attraction $a$ and $S_f$.

4.1.2 Shear Strength

In table 4.1 $\tau_{max}$ and $\tau_{\epsilon 2\%}$ is found. The average numbers are reported in table 4.2. The most important figures is the significant higher (+17 %) $\tau_{max}$ for $K'_0$ at 0.8. Another interesting finding is that the ratio of $\tau_{max}$ to $\tau_{\epsilon 2\%}$ is the same for both 0.6 and 0.8. The differences between the ratio of $\sigma'_{V0}$ to $\tau_{max}$ is in the same order of magnitude with +19%.

The results from every triaxial test from both bore holes is plotted in figure 4.1 and figure 4.2. As seen, the stress paths is placing itself towards almost the same failure line for both 0.6 and 0.8.

In figure 4.3 $\tau_{max}$ is divided on $\sigma'_{V0}$ and plotted against $\Delta V$. The tendency described earlier is also seen in this plot with 0.6 dominating the lower left part of the figure while 0.8 dominate the uppermost part.
CHAPTER 4. RESULTS

Figure 4.2: NTNU-plot of every test with $K'_0$ at 0.8. Curves clipped after yielding. A failure line is drawn with corresponding attraction $a$ and $S_f$.

4.1.3 Pore Water Extortion

The samples that are believed to be disturbed is plotted with a dashed line. They are not included in the average plots. See more about this in section 5.1.4.

Figure 4.4 displays the amount of pore water extorted in total for every test, plotted

Figure 4.3: $\tau_{max}$ divided on $\sigma_{vo}$ plotted against $\Delta V$. 
4.1. TRIAXIAL TESTS

Figure 4.4: Amount of pore water extortion increases with depth.

against the depth. The trend line is based on a simple linear regression and has a coefficient of determination ($R^2$) of 0.4.

Figure 4.5 displays the cumulative amount of pore water extorted in each step for every test. The tests with $K'_0 = 0.8$ extort more water in the early steps, but this gap is almost closed

Figure 4.5: The cumulative amount of pore water extruded at each step on all tests.
after the complete consolidation.

In figure 4.6, $\Delta V$ for each step is divided on $\Delta p'$ for each step and plotted. The average lines follow each other closely, but a small shift is seen after step 3. This is closely connected to the fact that when $K'_0$ is 0.8, $\Delta p'$ is very small in the two last loading steps.

Figure 4.7 displays $\Delta V(\sigma)$ for each step divided on $\Delta p'$ for each step. This figure displays very similar behavior as seen in figure 4.6.

Figure 4.8 and figure 4.9 displays $\Delta V(t)$ for each step in two different manners. In figure 4.8, $\Delta V(t)$ is divided on $\Delta p'$. This has the same effect as described earlier that $K'_0$ at 0.8 gives a higher value in the latest step because $\Delta p'$ is small in these steps.

Figure 4.9 on the other hand displays in reality the absolute values because the time com-
4.1. TRIAXIAL TESTS

Figure 4.8: Amount of pore water extorted in each step from the time effect divided on the change in average stress.

ponent (30 min) is the same for almost every data-point. A high $R$ indicates that the soil has little creep and is withstanding the actual pressure well. The upper part of figure 4.9 displays the difference between the average lines seen in the lower part of the figure. A clear trend of higher increase in resistance for a higher $K'_0$ from the start to the last step.

The percentage of the total volume change which comes from creep and the stress-component is displayed in figure 4.10. While the lines is following each other closely until step 3, the R-value for $K'_0$ at 0.8 makes a clear shift. This is connected to the small change in deviator stress and that the creep then will naturally have a bigger impact.
Figure 4.9: Bottom figure: overview over $R$. Top figure: The difference between the average values presented together with the trend line for the data.

Figure 4.10: Overview over the share of creep from the total volume change in every step.
4.2 Oedometer Test

In figure 4.11 $p'_c$ is plotted together with the final soil profile. The knee in the line represents the ground water level at 1.4 meter.

![Figure 4.11: Overview over $p'_c$ found in oedometer tests and estimated effective overburden pressure.](image-url)

Figure 4.11: Overview over $p'_c$ found in oedometer tests and estimated effective overburden pressure.
4.3 Index Tests

The data from the index-tests is found in table 4.3. The most noteworthy information is the clear change in the clay properties from about 7 meters depth. As seen in figure 4.12, the clay content is way lower than at shallower depths. The sensitivity, $S_t$, is a bit higher, while the plasticity index ($I_p$) and liquidity index ($I_L$) is respectively significantly higher and lower for this soil layer.
### 4.3. INDEX TESTS

Table 4.3: Overview over results from every index-test

<table>
<thead>
<tr>
<th>Cylinder</th>
<th>Depth</th>
<th>Atterberg Limits</th>
<th>Density</th>
<th>Falling cone</th>
<th>Salinity</th>
<th>Organic Content</th>
<th>OCR</th>
<th>Clay Content</th>
</tr>
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<tr>
<td></td>
<td></td>
<td>$w$</td>
<td>$w_t$</td>
<td>$w_p$</td>
<td>$I_p$</td>
<td>$I_c$</td>
<td>whole $\rho$</td>
<td>triax1 $\phi$</td>
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<tr>
<td>3-1</td>
<td>3-3.8</td>
<td>49</td>
<td>56</td>
<td>27</td>
<td>29</td>
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<td>1.75</td>
<td>1.77</td>
</tr>
<tr>
<td>3-2</td>
<td>4-4.8</td>
<td>50</td>
<td>65</td>
<td>29</td>
<td>36</td>
<td>59</td>
<td>1.75</td>
<td>1.77</td>
</tr>
<tr>
<td>2-2</td>
<td>4-4.8</td>
<td>48</td>
<td>62</td>
<td>29</td>
<td>33</td>
<td>58</td>
<td>-</td>
<td>1.77</td>
</tr>
<tr>
<td>2-3</td>
<td>5-5.8</td>
<td>56</td>
<td>63</td>
<td>31</td>
<td>32</td>
<td>78</td>
<td>1.80</td>
<td>1.79</td>
</tr>
<tr>
<td>3-3</td>
<td>5-5.8</td>
<td>48</td>
<td>59</td>
<td>29</td>
<td>29</td>
<td>64</td>
<td>1.69</td>
<td>1.77</td>
</tr>
<tr>
<td>3-4</td>
<td>6-6.8</td>
<td>43</td>
<td>34</td>
<td>20</td>
<td>14</td>
<td>160</td>
<td>1.82</td>
<td>1.82</td>
</tr>
<tr>
<td>2-4</td>
<td>6-6.8</td>
<td>48</td>
<td>39</td>
<td>22</td>
<td>17</td>
<td>156</td>
<td>1.87</td>
<td>1.84</td>
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<tr>
<td>2-5</td>
<td>7-7.8</td>
<td>44</td>
<td>27</td>
<td>19</td>
<td>8</td>
<td>318</td>
<td>1.93</td>
<td>1.98</td>
</tr>
</tbody>
</table>
Chapter 5

Discussion

5.1 Possible Errors

There are many possible sources for errors in the laboratory. The most influential for the results from the triaxial test are presented and discussed in the following section.

5.1.1 Apparatus

A triaxial test cell is a complex apparatus with many components. Critical failures would be detected by the operator, but small failures in some of the components could appear without being noticed.

It's believed the sensors in the triaxial apparatus are precise, but no test to check this was conducted. There could therefore be both systematic and random errors in the raw files received from the apparatus.

During the triaxial testing there have been some bigger incidents detected with different consequences. They are mentioned by the results in appendix A and includes:

1. Burette would not measure extorted pore water. Tests were run, but with no results from the consolidation phase. The laboratory engineers fixed the burette before new tests were conducted.

2. Electricity outage right after the consolidation phase had started. The sensors did not respond and the sample was probably disturbed with too high cell pressure. Cell was drained and sensors zeroed out before the test was restarted.

3. Leakage in different cables during the application of back-pressure. The back-pressure
was lowered and cable fixed. The shearing phase was then executed with a lower level of back-pressure.

4. Numerous leakages in cables in both the consolidation and shear phase. Was fixed without any other impact than small time delays.

5. The pedestal was not in place when the sample was built in and the pedestal with sample fell about 5 mm before an abrupt stop when the cell was installed. The test was thereafter conducted in a normal manner, but the sample was probably highly disturbed from the accident.

6. Error with the pressure sensors. There was water pressed from the burette into the sample from the beginning. Could have been that they wasn't zeroed out. The test was executed, but no results from the consolidation phase was retrieved and the result from the shear phase are also very questionable.

5.1.2 Cylinder Extortion

When the clay is retrieved from the field it's stored in steel cylinders in a refrigerated room at 4 °C. The first operation before the test can be conducted is to remove the top and bottom protection cover. Afterwards the piston head in the top of the cylinder is removed. This is where a potential problem have occurred when the first five cylinders was opened. The vacuum that in some cases are present in the sample was not removed and the piston head was just pulled out by brute force. This could have lead to tension in the sample that could have disturbed it, especially in the upper part of the cylinder. The amount of force used to remove the piston head varied and depends on the level of pressure in the cylinder. It's not known to what extent this has caused problems, but based on the quality criteria mentioned in section 2.4 the tests conducted on the uppermost soil in these cylinders are believed to be too disturbed.

5.1.3 Operational Skills

As described earlier the clay can easily be disturbed if it's not handled with care. Every interaction with the specimen could potentially cause disturbances if they are not done in a gentle way. The best way to do the different operations, from cylinder extortion, to cutting and build-in can only be learned through practise. The operator of the tests, and author of
this paper, had prior to this work very limited experience with laboratory work in general and triaxial testing in particular. To reduce the impact on the lack of practical skills it was conducted several triaxial tests on random clay samples before the start of this work. This improved the quality of the first real samples, but it is still reasonable to think that the quality would further increase throughout the work.

Based on an analysis of the results it was not possible to quantify this effect. Apart from the samples disturbed from the cylinder extortion the effect of better operational skills is not taken into further consideration.

5.1.4 Included In the Average Values

The samples that is, based on the discussion above, believed to be more than accepted disturbed is not included in the calculated average numbers. That includes, with reasons:

- 3-1.2 - not zeroed out.
- 3-2.1 - fell down after installed on pedestal
- 2-2.1, 2-3.1, 2-5.1 - believed to be disturbed from the extrusion.
- 2-4.1 - burette not working

The tests with a rest-time at each step on 1 hour is included based on the fact that the amount of extortion in the extra 30 minutes is insignificant.

5.2 What $K'_0$ to Chose

When a triaxial test is ordered the chosen $K'_0$ will vary based on the type of soil and topography, but also on the purpose of the test. It’s important to remember this, and to keep in mind that the following discussion is based on the condition found on overconsolidated clay with relatively low sensitivity in a flat area at a depth shallower than 8 meter.

With the small amount of soil samples, and even smaller number of samples with accepted quality, the data material is inadequate for a clear conclusion. Combined with the scattered results from every individual test and the embedded unavoidable fact that it’s impossible to do undisturbed tests of the same soil twice, and the conclusion get’s even weaker. On the other hand it is possible to see some trends in the average values that can be explained by accepted theories.
5.2. WHAT $K'_0$ TO CHOOSE

The clearest tendency seen is the higher $\tau_{max}$ for a higher $K'_0$. A $K'_0$ at 0.8 gives in average 17% higher $\tau_{max}$ than at 0.6. This corresponds with earlier experience and is not surprising. This higher $\tau_{max}$ is not in itself a argument for choosing a higher $K'_0$, it’s more important that the value is representative for the actual $s_u$ of the soil.

The short version of the comparison of the amount of extorted pore water is that it doesn’t matter if the $K'_0$ is 0.6 or 0.8. Even though the amount of pore water extortion in the different steps vary, the total sum after the consolidation phase is almost the same. This means that the fear of choosing a low $K'_0$ seems a bit overrated.

Arguments against $K'_0$ as low at 0.6 can be found in the results in figure 2.7. A higher resistance number indicates that the sample is less disturbed as explained in section 2.5. The results is not indisputable, but it’s a clear trend.

Compared to the suggestions for $K'_0$ based on proposal from Brooker and Ireland [1965] and L’Heureux et al. [2017] seen in table 2.1 the results from this thesis confirms that these approximations could be useful when deciding what $K'_0$ to use in a triaxial test, especially on a depth greater than five meters.
Chapter 6

Conclusion and Further Work

6.1 Conclusion

It is not possible to reach an unambiguous conclusion from the results, but the maximum shear strength of tests with a $K'_0$ at 0.8 is significant higher while the total amount of pore water extortion is equal between the two values. The amount of creep in the last step is on the other hand smaller for a $K'_0$ at 0.8. This indicates that the soil is handling the stress level better than with a $K'_0$ at 0.6.

6.2 Recommendations for Further Work

Based on the uniformity in the material used in this work it would be interesting to see if the tendency in the results would also be found in other types of clay. The most obvious continuation would be to investigate a normal consolidated clay. Besides investigating if the main conclusion in this thesis is applicable for normal consolidated soil it would be appealing to see if the amount of creep in the later loading steps would follow the pattern found in this work.

Based on the small differences in results between a $K'_0$ at 0.6 and 0.8 found in this work it would be interesting to see how the impact would have been on other values of $K'_0$, both higher and lower.
Bibliography


Appendix A

Results Triaxial Shear Tests
Triaxial test 2-1.1

Depth: 3.35 m
Sampling date: 20.09
Opening date: 25.09
Testing date: 25.09
Triax apparatus: Triax1
Comment: the first test

- $\sigma'_{v0} = 50.1$ kPa
- $K_0' = 0.8$
- $\gamma = 17.3$ kN/m$^3$
- $B$-value = 0.82

Volume change and deformation during consolidation

\[ \Delta V \text{ [cm}^3\text{]} \]

\[ t \text{ [min]} \]

NTNU-plot

Pore pressure and shear stress under shear phase

\[ \frac{1}{3}(\sigma_1' - 2\sigma_3') \text{ [kPa]} \]

\[ \sigma_3' \text{ [kPa]} \]

\[ \varepsilon_{axial} \text{ [%]} \]

\[ u \text{ [kPa]} \]

Figure A.1: Results from Triaxial shear test at hole 2, depth 3.35 meter.
Triaxial test 2-1.2

| Depth: 3.50 m | $\sigma'_{v0}$ | 54.6 kPa |
| Sampling date: 20.09 | $K_0'$ | 0.6 |
| Opening date: 25.09 | $\gamma$ | 17.3 kN/m³ |
| Testing date: 26.09 | $w$ | - |
| Triax apparatus: Triax1 | $B$-value | 0.96 |

Volume change and deformation during consolidation

$\Delta V$ [cm³]

$\delta$ [cm]

t [min]

Pore pressure and shear stress under shear phase

$\sigma'_{1} - \sigma'_{3}$ [kPa]

$u$ [kPa]

Figure A.2: Results from Triaxial shear test at hole 2, depth 3.50 meter.
Triaxial test 2-2.1

Volume change and deformation during consolidation

NTNU-plot

Pore pressure and shear stress under shear phase

Figure A.3: Results from Triaxial shear test at hole 2, depth 4.30 meter.
Triaxial test 2-2.2

Depth: 4.68 m
Sampling date: 20.09
Opening date: 28.09
Testing date: 29.09
Triax apparatus: Triax1
Comment: rest-time 1h

\[ \sigma_0' = 54.7 \text{kPa} \]
\[ K_0' = 0.6 \]
\[ w = -0.4 \text{kN/m}^3 \]
\[ B-value = 0.95 \]

Volume change and deformation during consolidation

![NTNU-plot](image)

Pore pressure and shear stress under shear phase

![p'-q-plot](image)

Figure A.4: Results from Triaxial shear test at hole 2, depth 4.68 meter.
APPENDIX A. RESULTS TRIAXIAL SHEAR TESTS

Triaxial test 2-3.1

<table>
<thead>
<tr>
<th>Parameter</th>
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<td>09.10</td>
</tr>
<tr>
<td>Opening date</td>
<td>09.10</td>
</tr>
<tr>
<td>Testing date</td>
<td>09.10</td>
</tr>
<tr>
<td>Triax apparatus</td>
<td>Triax1</td>
</tr>
<tr>
<td>Comment</td>
<td>believed to be disturbed</td>
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<tr>
<td>$\sigma_{v0}'$</td>
<td>58.9 kPa</td>
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<td>$\omega$</td>
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<tr>
<td>$\rho$</td>
<td>1.6 kN/m$^3$</td>
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<tr>
<td>$B$-value</td>
<td>0.92</td>
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</table>

Volume change and deformation during consolidation

Pore pressure and shear stress under shear phase

Figure A.5: Results from Triaxial shear test at hole 2, depth 5.23 meter.
Triaxial test 2-3.2

Volume change and deformation during consolidation

Pore pressure and shear stress under shear phase

Figure A.6: Results from Triaxial shear test at hole 2, depth 5.35 meter.
APPENDIX A. RESULTS TRIAXIAL SHEAR TESTS

Triaxial test 2-4.1

- Depth: 6.25 m
- Sampling date: 09.10
- Opening date: 09.10
- Testing date: 10.10
- Triax apparatus: Triax1
- Comment: burette not working, ok shear

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<tr>
<td>$K_0'$</td>
<td>0.8</td>
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<tr>
<td>w</td>
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<tr>
<td>$\gamma$</td>
<td>18.1 kN/m$^3$</td>
</tr>
<tr>
<td>B-value</td>
<td>0.96</td>
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</tbody>
</table>

Volume change and deformation during consolidation

Pore pressure and shear stress under shear phase

Figure A.7: Results from Triaxial shear test at hole 2, depth 6.25 meter.
Triaxial test 2-4.2

Volume change and deformation during consolidation

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<td>09.10</td>
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<tr>
<td>Triax apparatus</td>
<td>Triax2</td>
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<tr>
<td>$v_0$</td>
<td>69.0 kPa</td>
</tr>
<tr>
<td>$K_0'$</td>
<td>0.6</td>
</tr>
<tr>
<td>$w$</td>
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</tr>
<tr>
<td>$\gamma$</td>
<td>18.6 kN/m$^3$</td>
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<tr>
<td>B-value</td>
<td>0.98</td>
</tr>
<tr>
<td>Comment</td>
<td>first step 40 min</td>
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Figure A.8: Results from Triaxial shear test at hole 2, depth 6.47 meter.
Figure A.9: Results from Triaxial shear test at hole 2, depth 7.23 meter.
Triaxial test 2-5.2

Depth: 7.50 m
Sampling date: 09.10
Opening date: 11.10
Testing date: 11.10
Triax apparatus: Triax2
Comment: V0 77.8 kPa
K0' 0.6
w: 38 %
y: 19.0 kN/m³
B-value 0.92

Volume change and deformation during consolidation

Pore pressure and shear stress under shear phase

Figure A.10: Results from Triaxial shear test at hole 2, depth 7.50 meter.
APPENDIX A. RESULTS TRIAXIAL SHEAR TESTS

Triaxial test 3-1.1

- Depth: 3.53 m
- Sampling date: 26.10
- Opening date: 26.10
- Testing date: 26.10
- Triax apparatus: Triax1
- Comment: rest time 1h

\[ \sigma_0 = 47.4 \text{ kPa} \]
\[ K_0 = 0.8 \]
\[ w = 50\% \]
\[ \lambda = 17.4 \text{ kN/m}^3 \]
\[ B = 0.95 \]

Volume change and deformation during consolidation

\[ \Delta V \] [cm³]

\[ \Delta \varepsilon \]

\[ t \] [min]

Pore pressure and shear stress under shear phase

\[ \frac{1}{3}(\sigma_1 + 2\sigma_3) \] [kPa]

\[ \varepsilon_{axial} \] [mm]

\[ \varepsilon_{axial} \] [%]

\[ \sigma_3 \] [kPa]

\[ u \] [kPa]

Figure A.11: Results from Triaxial shear test at hole 3, depth 3.53 meter.
**Triaxial test 3-1.2**

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</tr>
<tr>
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<tr>
<td>Comment</td>
<td>rest time 1h, something wrong with the pressure sensors.</td>
</tr>
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Volume change and deformation during consolidation

Pore pressure and shear stress under shear phase

Figure A.12: Results from Triaxial shear test at hole 3, depth 3.42 meter.
APPENDIX A. RESULTS TRIAXIAL SHEAR TESTS

Triaxial test 3-1.2X

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Volume change and deformation during consolidation

Pore pressure and shear stress under shear phase

Figure A.13: Results from extra Triaxial shear test at hole 3, depth 3.69 meter.
Triaxial test 3-2.1

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<td>Triax apparatus</td>
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<td>Comment</td>
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<td>( \sigma'_{v0} )</td>
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</tr>
<tr>
<td>Water content ( w )</td>
<td>51 %</td>
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<tr>
<td>( \gamma )</td>
<td>17.4 kN/m³</td>
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<td>( B )-value</td>
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</tbody>
</table>

Volume change and deformation during consolidation

\[ \Delta V \text{ [cm]} \]

\[ t \text{ [min]} \]

\[ \varepsilon_{\text{total}} \text{ [mm]} \]

\[ \tau(\sigma',+2\sigma_3') \text{ [kPa]} \]

Pore pressure and shear stress under shear phase

\[ \tau(\sigma',-\sigma_3) \text{ [kPa]} \]

\[ \sigma_3' \text{ [kPa]} \]

\[ \varepsilon_{\text{axial}} \text{ [%]} \]

\[ u \text{ [kPa]} \]

Figure A.14: Results from Triaxial shear test at hole 3, depth 4.22 meter.
APPENDIX A. RESULTS TRIAXIAL SHEAR TESTS

**Triaxial test 3-2.1X**

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<td>( \sigma'_v )</td>
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<td>( w )</td>
<td>48%</td>
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<td>( V_c )</td>
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<td>B-value</td>
<td>0.95</td>
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<tr>
<td>Volume change and deformation during consolidation</td>
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</tr>
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</table>

- **NTNU-plot**
  - \( \Delta V \) vs. \( t \)

- **\( p'\)-q-plot**
  - \( \frac{1}{3}(\sigma'_1+2\sigma'_3) \) vs. \( \varepsilon_{s\text{ual}} \)
  - \( \frac{1}{3}(\sigma'_1-\sigma'_3) \) vs. \( \sigma'_3 \)

Figure A.15: Results from extra Triaxial shear test at hole 3, depth 4.67 meter.
Triaxial test 3-2.2

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<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth:</td>
<td>4.42 m</td>
</tr>
<tr>
<td>Sampling date:</td>
<td>26.10</td>
</tr>
<tr>
<td>Opening date:</td>
<td>27.10</td>
</tr>
<tr>
<td>Testing date:</td>
<td>27.10</td>
</tr>
<tr>
<td>Triax apparatus:</td>
<td>Triax2</td>
</tr>
<tr>
<td>B-value</td>
<td>0.96</td>
</tr>
</tbody>
</table>

Volume change and deformation during consolidation

p'-q-plot

Pore pressure and shear stress under shear phase

Figure A.16: Results from Triaxial shear test at hole 3, depth 4.42 meter.
## Appendix A. Results Triaxial Shear Tests

### Triaxial test 3-3.1

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth</td>
<td>5.25 m</td>
</tr>
<tr>
<td>Sampling date</td>
<td>26.10</td>
</tr>
<tr>
<td>Opening date</td>
<td>29.10</td>
</tr>
<tr>
<td>Testing date</td>
<td>29.10</td>
</tr>
<tr>
<td>Triax apparatus: Triax1</td>
<td></td>
</tr>
<tr>
<td>B-value</td>
<td>0.99</td>
</tr>
<tr>
<td>γ₀</td>
<td>17.3 kN/m³</td>
</tr>
<tr>
<td>k₀</td>
<td>0.6</td>
</tr>
<tr>
<td>Preconsolidation pressure</td>
<td>66.2 kPa</td>
</tr>
<tr>
<td>W</td>
<td>51 %</td>
</tr>
<tr>
<td>Sample density</td>
<td>51.3 kN/m³</td>
</tr>
<tr>
<td>V0</td>
<td></td>
</tr>
<tr>
<td>B-value</td>
<td>0.99</td>
</tr>
</tbody>
</table>

### Volume change and deformation during consolidation

![Volume change and deformation during consolidation graph](image)

**NTNU-plot**

### Pore pressure and shear stress under shear phase

![Pore pressure and shear stress under shear phase graph](image)

**p'-q-plot**

Figure A.17: Results from Triaxial shear test at hole 3, depth 5.25 meter.
**Triaxial test 3-3.2**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth</td>
<td>5.56 m</td>
</tr>
<tr>
<td>Sampling date</td>
<td>26.10</td>
</tr>
<tr>
<td>Opening date</td>
<td>29.10</td>
</tr>
<tr>
<td>Testing date</td>
<td>29.10</td>
</tr>
<tr>
<td>Triax apparatus</td>
<td>Triax2</td>
</tr>
<tr>
<td>( \sigma_{v0} )</td>
<td>69.6 kPa</td>
</tr>
<tr>
<td>( K_0' )</td>
<td>0.8</td>
</tr>
<tr>
<td>( w )</td>
<td>52 %</td>
</tr>
<tr>
<td>( \gamma )</td>
<td>17.3 kN/m³</td>
</tr>
<tr>
<td>B-value</td>
<td>0.97</td>
</tr>
</tbody>
</table>

Volume change and deformation during consolidation

\[ \Delta V \text{[cm}^3\text{]} \]

\[ t \text{[min]} \]

**p'-q-plot**

Pore pressure and shear stress under shear phase

\[ \frac{\tau}{(\sigma'_1+2\sigma'_3)} \text{[kPa]} \]

\[ \epsilon_{axial} \text{[%]} \]

\[ \sigma'_3 \text{[kPa]} \]

\[ u \text{[kPa]} \]

Figure A.18: Results from Triaxial shear test at hole 3, depth 5.56 meter.
APPENDIX A. RESULTS TRIAXIAL SHEAR TESTS

Triaxial test 3-4.1

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth</td>
<td>6.22 m</td>
</tr>
<tr>
<td>Sampling date:</td>
<td>26.10</td>
</tr>
<tr>
<td>Opening date:</td>
<td>30.10</td>
</tr>
<tr>
<td>Testing date:</td>
<td>30.10</td>
</tr>
<tr>
<td>Triax apparatus:</td>
<td>Triax1</td>
</tr>
<tr>
<td>Comment:</td>
<td>power outage. Was exposed for suddenly high cell pressure</td>
</tr>
<tr>
<td>$\sigma_0'$</td>
<td>77.2 kPa</td>
</tr>
<tr>
<td>$K_0'$</td>
<td>0.6</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>17.8 kN/m$^3$</td>
</tr>
<tr>
<td>$w$</td>
<td>46 %</td>
</tr>
<tr>
<td>B-value</td>
<td>0.97</td>
</tr>
<tr>
<td>Depth:</td>
<td>6.22 m</td>
</tr>
<tr>
<td>Sampling date:</td>
<td>26.10</td>
</tr>
<tr>
<td>Opening date:</td>
<td>30.10</td>
</tr>
<tr>
<td>Testing date:</td>
<td>30.10</td>
</tr>
<tr>
<td>Triax apparatus:</td>
<td>Triax1</td>
</tr>
<tr>
<td>Comment:</td>
<td>power outage. Was exposed for suddenly high cell pressure</td>
</tr>
</tbody>
</table>

Volume change and deformation during consolidation

**p-q-plot**

Pore pressure and shear stress under shear phase

Figure A.19: Results from Triaxial shear test at hole 3, depth 6.22 meter.
Triaxial test 3-4.2

Depth: 6.52 m
Sampling date: 26.10
Opening date: 30.10
Testing date: 30.10
Triax apparatus: Triax2
Comment: power outage. Drained the cell for water and restarted.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{v0}$</td>
<td>80.6 kPa</td>
</tr>
<tr>
<td>$K_0'$</td>
<td>0.8</td>
</tr>
<tr>
<td>w</td>
<td>46 %</td>
</tr>
<tr>
<td>$\gamma_c$</td>
<td>17.8 kN/m³</td>
</tr>
<tr>
<td>B-value</td>
<td>0.83</td>
</tr>
</tbody>
</table>

Volume change and deformation during consolidation

p'-q-plot

Pore pressure and shear stress under shear phase

Figure A.20: Results from Triaxial shear test at hole 3, depth 6.52 meter.
Appendix B

Oedometer
Figure B.1: Summation plot of every oedometer test with adequate quality.
Oedometer 2-4.5

Sampling date: 20.09
Opening date: 28.09
Testing date: 25.10

\[ \rho_0' \quad \sigma_{v0}' \quad 56.6 \text{ kPa} \]

\[ \text{OCR} \quad 17.8 \text{kN/m}^3 \]

Figure B.2: Oedometer-results from hole 2, depth 4.50 meter.
Figure B.3: Oedometer-results from hole 2, depth 5.48 meter.
Oedometer 2-6.59

Sampling date: 09.10
Opening date: 10.10
Testing date: 24.10

\( \rho'_c = 210 \text{ kPa} \)
\( \sigma'_c = 79.7 \text{ kPa} \)
OCR 2.6
\( \gamma = 17.6 \text{ kN/m}^3 \)

Figure B.4: Oedometer-results from hole 2, depth 6.59 meter.
Figure B.5: Oedometer-results from hole 2, depth 7.42 meter.
Figure B.6: Oedometer-results from hole 3, depth 3.35 meter.
Figure B.7: Oedometer-results from hole 3, depth 4.35 meter.
Oedometer 3-5.16

Sampling date: 26.10
Opening date: 29.10
Testing date: 29.10

\( \rho' \)  240 kPa
\( \alpha_u' \)  63.7 kPa
OCR  3.8
\( \gamma \)  16.7 kN/m²

Figure B.8: Oedometer-results from hole 3, depth 5.16 meter.
Oedometer 3-5.37

Sampling date: 26.10
Opening date: 29.10
Testing date: 30.10

\( \rho_i \) - 66.0 kPa
\( \sigma_{oc} \) - 17.1 kN/m

\( \varepsilon \) [%]
\( u \) [kPa]
\( M \) [kPa]
\( c_v \) [m²/year]

\( \sigma' = \sigma - \rho g h \) [kPa]

Figure B.9: Oedometer-results from hole 3, depth 5.37 meter.
Oedometer 3-6.62

Sampling date: 26.10
Opening date: 30.10
Testing date: 30.10

\( p_c' \) 240 kPa
\( \sigma_{c}' \) 80.1 kPa
OCR 3.0
\( \gamma \) 17.2 kN/m³

Figure B.10: Oedometer-results from hole 3, depth 6.62 meter.