NTNU – Trondheim
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Shear Creep in Sensitive Clays

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Geotechnics and Geohazards
Submission date: June 2015
Supervisor: Arnfinn Emdal, BAT
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STUDY OF SHEAR CREEP IN SENSITIVE CLAY

Skjærkryp i sensitive leirer

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Abstract:

In standard triaxial test, pore water pressure in a sample is measured at the surface with external pore pressure sensor. The pore water pressure inside the clay sample is different from the pore pressure at the surface. This variation in the pore pressure can cause high mobilization of the shear strength during consolidation phase of K_0 triaxial test. This can cause failure of the sample far below the failure line during the undrained shear test. The pore water pressure in the sample can also cause the failure due to accumulation of pore pressure even if the axial load is held constant. Therefore, to confirm the role of the pore water pressure in the failure of the clay sample during K_0 triaxial test, undrained K_0 shear creep test was done for Tiller clay and Goteborg clay in the modified triaxial apparatus. The apparatus contained the pore pressure sensor installed in it that measures the pore pressure in the middle of the sample of height 50mm and diameter 54mm. The filter plates were not used during the test to reduce the end restraint effect, and drainage of the pore water was only in lateral direction. The consolidation of the samples were done with different loading steps. After that, they were sheared at different rates to different degree of shear mobilization and then creep phase was started. Pore pressure measurement in the middle and surface of the sample was done for consolidation, shearing and creep phases. The effective stress plot at consolidation phase showed that the sample was apparently mobilized to higher degree of mobilization or consolidated to higher K_0 value, although it was consolidated to their actual K_0 value. It showed that the consolidation along K_0 line should be done with the small loading steps and adequate time should be provided for each loading steps to release all the pore water pressure inside the sample otherwise pore water pressure will be accumulated in the consecutive loading steps. The pore water pressure variation in the middle and surface of the sample was about 4 to 6KPa. This variation was different for the different rate of strain. The variation was higher for the high strain rate than low strain rate.

During the creep phase, the strain rate decreased initially and remained constant and then increased leading to the failure of the sample. The sample that was sheared to higher degree of shear mobilisation failed earlier than that of sheared to lower degree of shear mobilization. The sample could not reach the allowed mobilisation degree when sheared at low strain rate.

Keywords: Pore water pressure, consolidation, degree of mobilization, shear creep

Signature

NTNU
Faculty of Engineering Science and Technology
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GEOTECHNICAL MASTER DEGREE THESIS

Spring 2015

for

Suresh Shrestha

STUDY OF SHEAR CREEP IN SENSITIVE CLAY

Skjærkryp i sensitive leirer

BACKGROUND

Stability of natural clay slopes subjected to environmental changes is today under close investigation in Norway. This is stimulated by the NIFS project (Natural Hazards, Infrastructure, Flow and Slides), a cooperation research project between Norwegian Public Road Administration, Norwegian Energy Directorate and Norwegian National Rail Administration, involving most research institution and consultancy companies in Norway.

This project is a continuation of a MSc thesis from 2014 by Guro Torpe. The challenge in the testing of soft sensitive quick clays has been the consolidation phase, where the samples failed at K_0'-levels far below expected failure. The pore pressure condition inside the sample is suspected to be the reason for this, and therefore a project including instrumentation of the central pore pressure is now started. The intention is to run the consolidation procedure under full control of the effective stress level, apply shear and finally under constant shear measure the behaviour in creep.

TASK

The study of the shear creep in sensitive clay will be done by carrying out the undrained triaxial test in Goteborg clay and Tiller quick clay. An internal pore pressure sensor will be used to measure the pore pressure at the middle of the sample in addition to external pore pressure measurements at the surface of the sample. The setup will be on samples with height/diameter approximately ~1, with non-draining end caps and therefore only radial consolidation. The specimens will be consolidated to their insitu stress by simulating the slope condition and they will be sheared undrained to different degrees of mobilization. Thereafter, the shear stress level is kept constant and the sample is allowed to fail in creep.

General about content, work and presentation

The report shall include:

- Title page with abstract and keywords
- Preface, summary and acknowledgement
- Literature study
- Test setup and documentation of material to be tested
- Results; measurements and presentation
- Discussion of results
- Conclusion
- Proposals for further work

Submission

The thesis shall be delivered by 25. June 2015.

Additional supervisors

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June 2015

Arnfinn Emdal

Amanuensis

NTNU Geoteknikk

Preface

This thesis is submitted in partial fulfilment of the requirements for the Degree of Master of Science in Geotechnics and Geohazards.

The study was supervised by Asst. Professor Arnfinn Emdal. The work is presented as a literature review of creep in clay and pore pressure variation in clay sample during triaxial test followed by test procedure and presentation of the soil characteristics, test results, discussions and conclusion.

This thesis was carried out at the Geotechnical Division, Department of Civil and Transportation Engineering, Norwegian University of Science and Technology (NTNU) from January 14 to June 25, 2015.

Trondheim/ June 25, 2015

Suresh Shrestha

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I would like to thank Assistant Professor Arnfinn Emdal for giving me the opportunity to work with him and for this invaluable advice and extensive support throughout the course of this thesis. I would like to thank NIFS project (Natural Hazards, Infrastructure, Flow and Slides), along with Norwegian Public Road Administration, Norwegian Energy Directorate, Norwegian National Rail Administration, and other research institution and consultancy companies in Norway that are in cooperation with this project, for providing an opportunity to write thesis in this project and supporting my thesis work.

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Lastly, I would like to thank my family and friends for all the support and encouragement they have given me.

Trondheim/ June 25, 2015

Suresh Shrestha

Summary

Stability of natural clay slopes subjected to environmental changes is today under close investigation in Norway. This is stimulated by the NIFS project (Natural Hazards, Infrastructure, Flow and Slides), a cooperation research project between Norwegian Public Road Administration, Norwegian Energy Directorate and Norwegian National Rail Administration, involving most research institution and consultancy companies in Norway.

This project is a continuation of a MSc thesis from 2014 by Guro Torpe. The challenge in the testing of soft sensitive quick clays has been the consolidation phase, where the samples failed at K_0'-levels far below expected failure. The pore pressure condition inside the sample is suspected to be the reason for this, and therefore a project including instrumentation of the central pore pressure is now started. The intention is to run the consolidation procedure under full control of the effective stress level, apply shear and finally under constant shear measure the behaviour in creep.

In standard triaxial test, pore water pressure in a sample is measured at the surface with external pore pressure sensor. The pore water pressure inside the clay sample is different from the pore pressure at the surface. This variation in the pore pressure can cause high mobilisation of the shear strength during consolidation phase of K_0 triaxial test. This can cause failure of the sample far below the failure line during the undrained shear test. The pore water pressure in the sample can also cause the failure due to accumulation of pore pressure even if the axial load is held constant. Therefore, to confirm the role of the pore water pressure in the failure of the clay sample during K_0 triaxial test, undrained K_0 shear creep test was done for Tiller clay and Goteborg clay in the modified triaxial apparatus. The apparatus contained the pore pressure sensor installed in it that measures the pore pressure in the middle of the sample of height 50mm and diameter 54mm. The filter plates were not used during the test to reduce the end restraint effect, and drainage of the pore water was only in lateral direction. The consolidation of the samples were done with different loading steps. After that, they were sheared at different rates to different degree of shear mobilisation and then creep phase was started. Pore pressure measurement in the middle and surface of the sample was done for consolidation, shearing and creep phases. The effective stress plot at consolidation phase showed that the sample was apparently mobilised to higher degree of mobilisation or consolidated to higher K_0 value, although it was consolidated to their actual K_0 value. It showed that the consolidation along K_0

line should be done with the small loading steps and adequate time should be provided for each loading steps to release all the pore water pressure inside the sample otherwise pore water pressure will be accumulated in the consecutive loading steps. The pore water pressure variation in the middle and surface of the sample was about 4 to 6KPa. This variation was different for the different rate of strain. The variation was higher for the high strain rate than low strain rate. During the creep phase, the strain rate decreased initially and remained constant and then increased leading to the failure of the sample. The sample that was sheared to higher degree of shear mobilisation failed earlier than that of sheared to lower degree of shear mobilization. The sample could not reach the allowed mobilisation degree when sheared at low strain rate.

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Chapter 1

1.0 Introduction

1.1 Background

This project is in co-operation with NIFS (SVV, JBV, and NVE) in co-operation with EU-CREEP and Geofuture. Natural hazard Infrastructure for Flood and Slide (NIFS) is collaboration between NPRA, Railway and Norwegian Water Resources and Energy Directorate (NVE). This project consists of seven subprojects:

1. Natural Disaster Strategy
2. Preparedness and Crisis Management
3. Mapping, Data Coordination and RAV analysis
4. Monitoring and Notification
5. Management of Flood and Water Astray
6. Quick Clay
7. Landslide and Flood Protection.

This thesis is related to subproject 6, “Quick Clay”. The main objective of this project is to clarify and facilitate the development of current regulations and procedures so that it will provide the basis for better and more uniform application in areas of quick clay either in terms of mapping and delineation of areas, interpretation of soil surveys, calculations and studies, based on the most similar safety philosophy, regardless of where and by whom is developed ***(NIFS 2013).***

Clay possesses specific structure and properties. Important aspects of clay behaviour include time/rate dependency, anisotropy in strength and stiffness and structure/destructuration. These provide greater challenges for stability. Hence appropriate characterization of natural clay for proper understanding of their behaviour is needed to treat it properly to avoid landslide ***(Grimstad, Degago et al. 2010)***. From sampling to interpretation and calculation, it requires high accuracy. Nowadays block samples are used instead of the 54mm piston sampler ***(Karlsrud, Otter et al. 2013)*** because comparatively, block samples provide high test quality with greater accuracy in test results. It provides high undrained shear strength (su) value and 15% reduction is recommended to prevent the brittle failure. It is also correlated with CPT and CPTU measurements before using in the stability analysis ***(Torpe 2014)***.

Time dependent response of clay is also important to be considered in the analysis while choosing the su-value from the block sample.

As shown in figure 1.1, natural slopes i.e. the slope that has not been influence by human activities, the soil is in the state of high degree of shear mobilization than the soils in the flat terrain. Mobilization degree for the natural soil is about f=0.83 to 0.90. Moreover, it is difficult and time consuming to extract the sample from slopes. Therefore, the sample that is extracted from the flat terrain, need to be simulated to the stress level corresponding to natural slope, as the samples from flat terrain has lower degree of shear mobilization ***(Torpe 2014)***.

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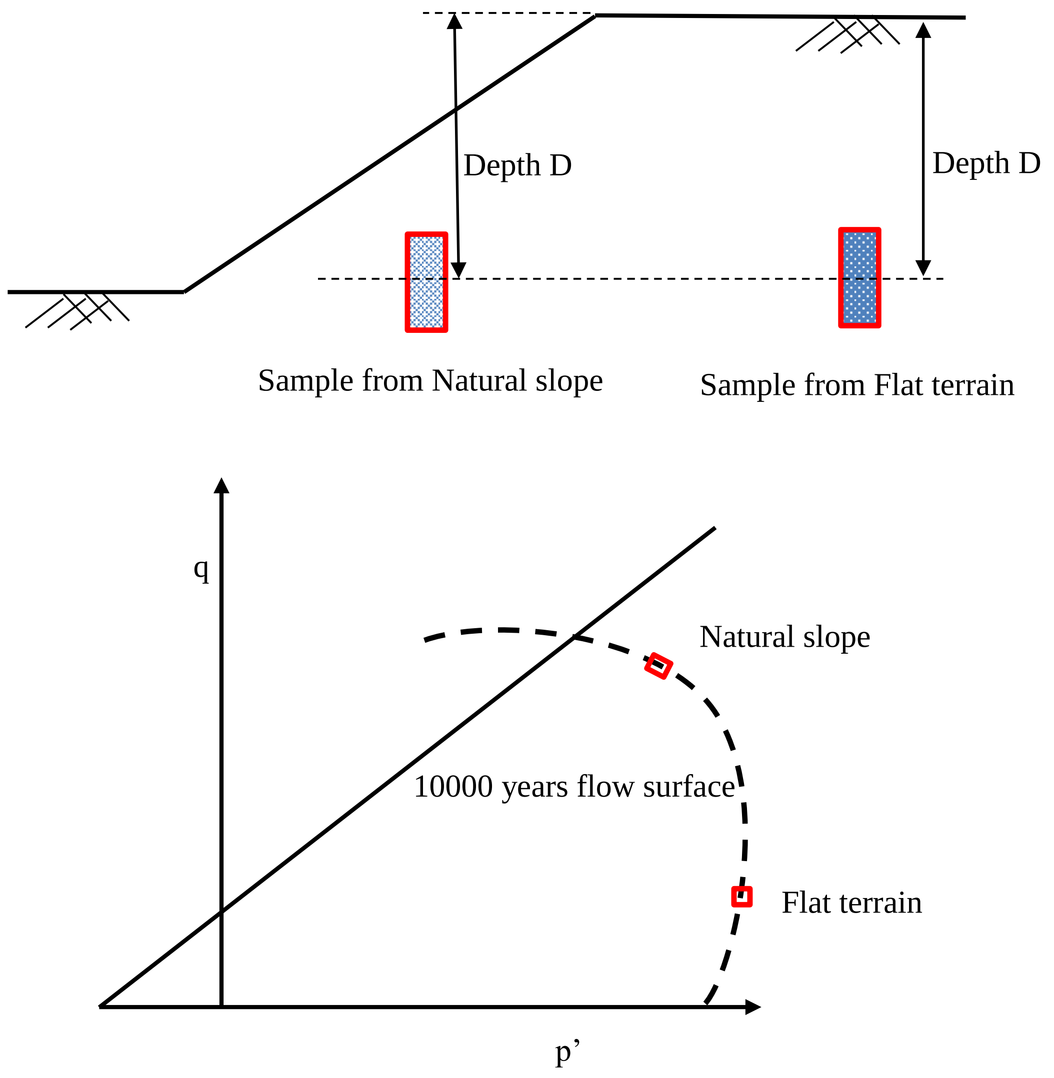


Figure 1.1 Stress condition in soil in natural slope and flat terrain

It has been previously studied on the undrained shear strength on the clay slopes and influence in stress condition ***(Lofroth 2008)*** and effect of creep on shear modulus of slope ***(Athanasopoulos and Richart 1983)***. ***(Lefebvre 1981)*** made research on the Canadian clay slopes to study the long term stability of the slopes and confirms that post peak strength permits a reasonable evaluation of stability of natural slopes. Drained triaxial test carried on the Champlain clay indicated that there is existence of stability threshold above which, even small increase in stress accelerates deformation, increase in pore water pressure and rapidly bring clay to failure. The stability threshold coincides fairly well with the post-peak strength defined at 8% of deformation in consolidated drained triaxial test. The peak strength is due to the bonding or cementation between particles and is not in equilibrium with the void ratio and with effective stress in the samples. But post peak strength represents the maximum resistance that can be mobilised after the breakdown of the time sensitive bonds, for a given void ratio and effective stress condition. The peak strength in slope stability analysis grossly overestimates the factor of safety ***(Lefebvre 1981)***. The research made on the creep behaviour of the intact, overconsolidated Saint-Alban clay ***(Tavenas, Leroueil et al. 1978)*** provides the knowledge of creep phenomenon and the detail study on the creep deformation, both volumetric and shear deformation, using drained and undrained triaxial test along with the oedometer test.

There was huge quick clay slide at ESP at Byneset in Trondheim in the morning on 01.01.2012 with an estimated volume of 300,000-350,000 m^3. The slide was believed to be caused by erosion. The material was supposed to be examined and hence used for the shear creep experiment in the thesis by ***(Torpe 2014)*** and determine the flow surface for the material. During test, the sample failed at K_0 level below the failure line, and the pore water pressure inside the clay was suspected to be the cause for the failure. Therefore, my research is focused on the study of pore water pressure inside the clay sample during the undrained K_0 triaxial creep test.

1.2 Objectives:

The main objective of this thesis was to focus on the study of pore pressure condition inside the soil sample during the consolidation phase. In the triaxial shear creep test done by ***(Torpe 2014)***, failure occurred below the failure line when the sample was tried to consolidate along the K_0 line. Therefore, it was recommended for the further work to study the condition of pore water pressure inside the sample during test with the K_0 value greater than 0.5. This has been done by installing the pore pressure sensor on the base of the triaxial cell that measures the pore pressure at the centre of 54mm diameter and 50mm height of sample. This study will help to understand how well natural slopes can take change in stress condition because it includes shearing of the samples to different degree of mobilization at constant rate of strain. Thereafter, the load was held constant so as to study the behaviour of clay under constant load with time.

The objectives of the thesis are listed below:

i. Study on the triaxial apparatus that required carrying out the shear creep test which measures the pore water pressure inside the sample and allows the lateral drainage of pore water only.
ii. Simulate the natural slope condition in the test by consolidating the sample along K_0 line.
iii. Study of pore pressure condition and stress condition in middle and surface of the sample during consolidation along K_0 line.
iv. Carry out shear creep test and study the pore pressure variation in middle and surface of the sample during the shearing and creep phase of the clay sample.
v. Study and understand stress and strain condition in undrained test while the clay sample is creeping after certain degree of mobilisation.

1.3 Thesis composition

This thesis is composed in the following manner below. Chapter 2 comprises of all the relevant theories related to the study of creep and pore pressure variation inside the triaxial specimen. Chapter 3 describes the procedure adopted to do the test along with the information about test apparatus and soil in details. Chapter 4 consist of the results obtained from the test with the illustration of the results. Chapter 5 discusses about the results obtained from the test with the possible reasons and the brief conclusion and finally, chapter 6 consist of recommendation for the further work for the continuation of this study.

Chapter 2

2.0 Literature study of Shear Creep in Sensitive Clay

2.1 Sensitive Clay

Sensitive soil consist of the significant amount of the water, because of this, the foundation lying on the sensitive soil has the low bearing capacity. While dealing with the sensitive soil, deformation in the soil is the main challenge. A clay whose shear strength is reduced to the very small fraction of its former value when remoulded at the constant moisture content is referred as the sensitive clay ***(Skemption and Northey 1952)***.

2.2 Creep

The settlement in the sensitive soil is divided into two categories; Primary consolidation and Secondary Consolidation as shown in figure 2.1. Primary consolidation is due to the change in the vertical effective stress. The change in the vertical effective stress is due to variation in pore water pressure or change in the regulatory framework to the slope. One of such changes may be erosion, change in ground water level or varying water level as the counterweight at the bottom of the slope.

Secondary Consolidation is the time dependent deformation and also known as creep. It occurs at the constant effective stress. It effects significantly on the future stability and serviceability built on the sensitive clay. It is generally assumed that secondary consolidation is small percentage of the primary consolidation but in case of the sensitive soil with the large primary deformation, the secondary deformation or the creep cannot be neglected ***(Alexandre, Martins et al. 2013)***

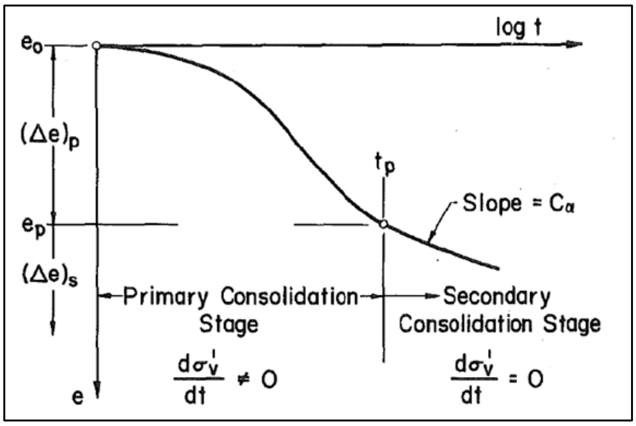


Figure 2.1. Primary and Secondary Compression (Gray 1936)

As indicated by *(Ladd 1977)* and *(Jamiolkowski 1985),* two hypothesis A and B were used as a basis for the extrapolation of creep from the short term observation in the laboratory test to the long term predictions in the field as shown in figure 2.2.

Hypothesis A: - Although creep starts during the primary consolidation but the end of the primary consolidation is unique and all the equations are based on this assumption. In hypothesis A, settlement is divided into two parts: Primary consolidation followed by secondary consolidation.

Hypothesis B: - Considering the fact that the primary consolidation consist of the creep, end of the primary consolidation void ratio cannot be constant and the equations are the result of this assumption. In hypothesis B, soil settlement is estimated using elasto-viscoplastic constitutive model simulating the soil creep and the consolidation settlement simultaneously.

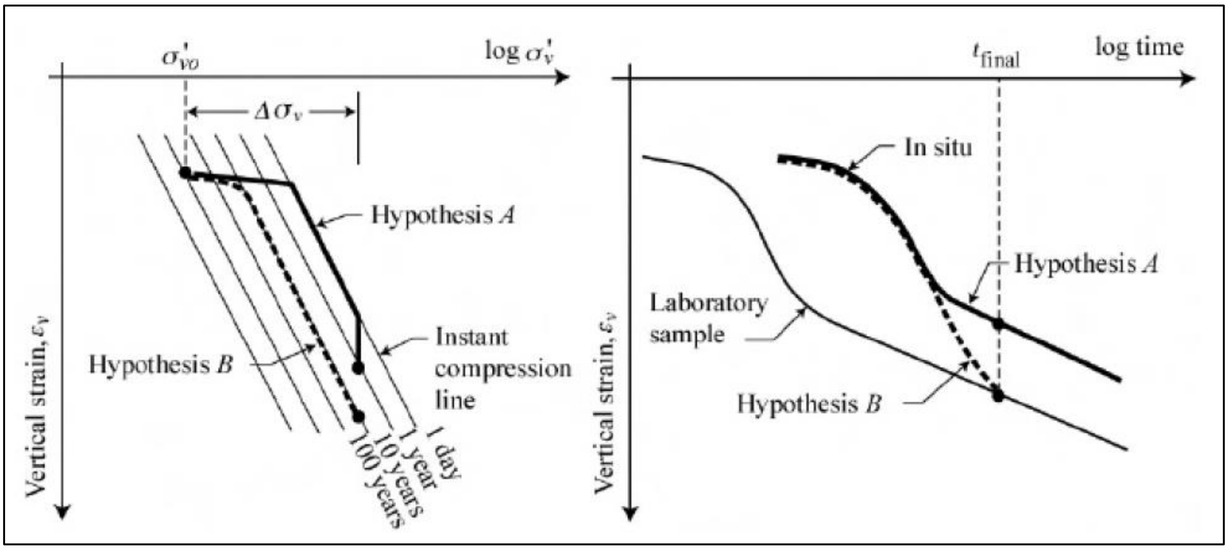


Figure 2.2 Hypothesis A and B after (Ladd 1977), from(Leroueil 2006)

Figure 2.2 shows that there is the opposing views between the hypothesis A ***(Ladd 1973), (Mesri and Rokhsar 1974)*** and hypothesis B ***(Taylor 1942) (Brinch Hasen 1969), (Barden 1969)*** and ***(Degago 2011)***. Hypothesis A shows the unique end of the primary (EOP) strain or void ratio that is valid for both the laboratory and the insitu primary condition. This means that for given effective stress increment with same initial effective stress, EOP strain is independent of the primary consolidation duration (t_{final}). Thus hypothesis A predicts the same preconsolidation stress for the laboratory tests and insitu condition. Hypothesis B shows different insitu EOP strain than the laboratory sample and also EOP increase with the increase in the sample thickness. Due to higher time required for the primary consolidation. Hypothesis A predicts lower EOP than Hypothesis B but after reaching EOP both hypotheses predicts similar creep deformation.

2.3 Mechanism of creep

“If the applied stress in a creep test is not greater than the maximum friction resistance available, the cohesion, which is fully mobilized for very small strains at the beginning of the test, would eventually be entirely transferred to friction with creep deformation coming to an end. On the other hand, if the applied stress is greater than the available friction, the transference of cohesion to friction will continue until the available friction is mobilized and the difference between applied stress and friction will be carried by the cohesion. As the cohesion is assumed to be strain dependent, the strain rate will decrease until all the available friction is mobilised and remain constant thereafter. Hence creep process involves the transfer of the effective cohesion to effective friction”. ***(Bjerrum 1974)***,

According to Martin’s hypothesis “the shear strength of saturated, normally consolidated clay has two components; the frictional resistance and the viscous resistance. The frictional resistance would develop between Terzaghi’s “solid bonds” and it would be the function of shear strain. The viscous resistance would develop between Terzaghi “film bonds” and it would be the function of the strain rate.” ***(Martins 1992).*** The equation for shear strength would be then.

$$\tau = \sigma' * \tan\varphi' + \eta(e) * \dot{\varepsilon} \tag{1}$$

Where

σ’ is the normal effective stress

φ’ is the mobilized effective angle of internal friction.

$\eta(e)$ is the coefficient of viscosity of the adsorbed water layer surrounding the clay particle (a function of void ratio for a normally consolidated clay)

$\dot{\varepsilon}$ is the strain rate.

Alexandre (2006) modified the Martin's model by replacing equation 1 by equation 2 *(Alexandre, Martins et al. 2013)*

$$\sigma_d = \sigma_{df}(\varepsilon) + K(e) * \dot{\varepsilon}^n \quad (2)$$

Where

σ_d is the deviatoric stress of a creep test

σ_{df} is the deviatoric frictional resistance (considered as a function of the shear strain for normally consolidated soils)

K and n are constants (K is the function of the consolidation pressure, σ'_c)

$\dot{\varepsilon}$ is the strain rate.

The mechanism of the creep can be understood with the aid of equation 2 and figure 2.3.

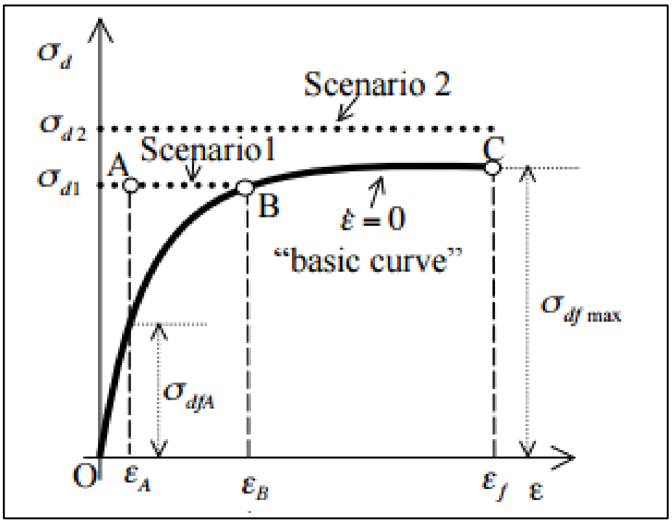


Figure 2.3 Deviatoric stress vs strain curve for two creep tests (Alexandre, Martins et al. 2013)

Figure 2.3 is the deviatoric stress strain curve showing two different scenario of the test. Scenario 1 is the condition when the applied stress is less than the maximum frictional deviatoric stress, $\sigma_{df\ max}$ and scenario 2 is the condition where the applied stress is greater than the maximum frictional deviatoric stress.

For scenario 1, at the beginning of the test, at t=0, the deviatoric stress, $\sigma_{d1} < \sigma_{dfmax}$ is applied. Considering equation 2 and figure 2.3, the vertical distance between the curve of test 1 and the basic curve is viscous resistance. Hence at this point the viscous resistance is identical to the applied deviatoric stress. After sometime, at point A, the frictional resistance will be σ_{dfA}, relative to shear strain ε_A. At point A because of equation 2, the viscous resistance will be smaller than before. As the process continues, the frictional resistance is mobilized and the viscous resistance is demobilized and strain rate decreases continuously. As $\sigma_{df1}<\sigma_{dfmax}$, viscous resistance will be transferred to frictional resistance continuously until the frictional resistance is equal to the applied deviatoric stress, at point B. At this point the viscous resistance as well as strain rate is equal to zero and the shear strain is ε_B.

The same process occurs in scenario 2, but, the applied deviatoric stress is now greater than the maximum frictional resistance, so, there will not be enough frictional resistance to be mobilized. At point C, where the frictional resistance is maximum and the viscous resistance is minimum and equal to (σ_{d2}-σ_{dfmax}) and strain rate will be $\dot{\varepsilon} = \left[\left(\sigma_{d2} - \sigma_{dfmax}\right)/K\right]^{1/n}$ .From this point on, the soil will continue to creep at constant strain rate indefinitely.

2.4 Creep process

Creep process is characterised by the decreasing strain rate followed by constant strain rate and finally increasing strain rate leading creep rupture. The period at which the strain rate is decreasing is called primary creep; secondary creep represents the constant strain rate and, the increasing creep rate represented by the tertiary creep as shown in figure 2.4.

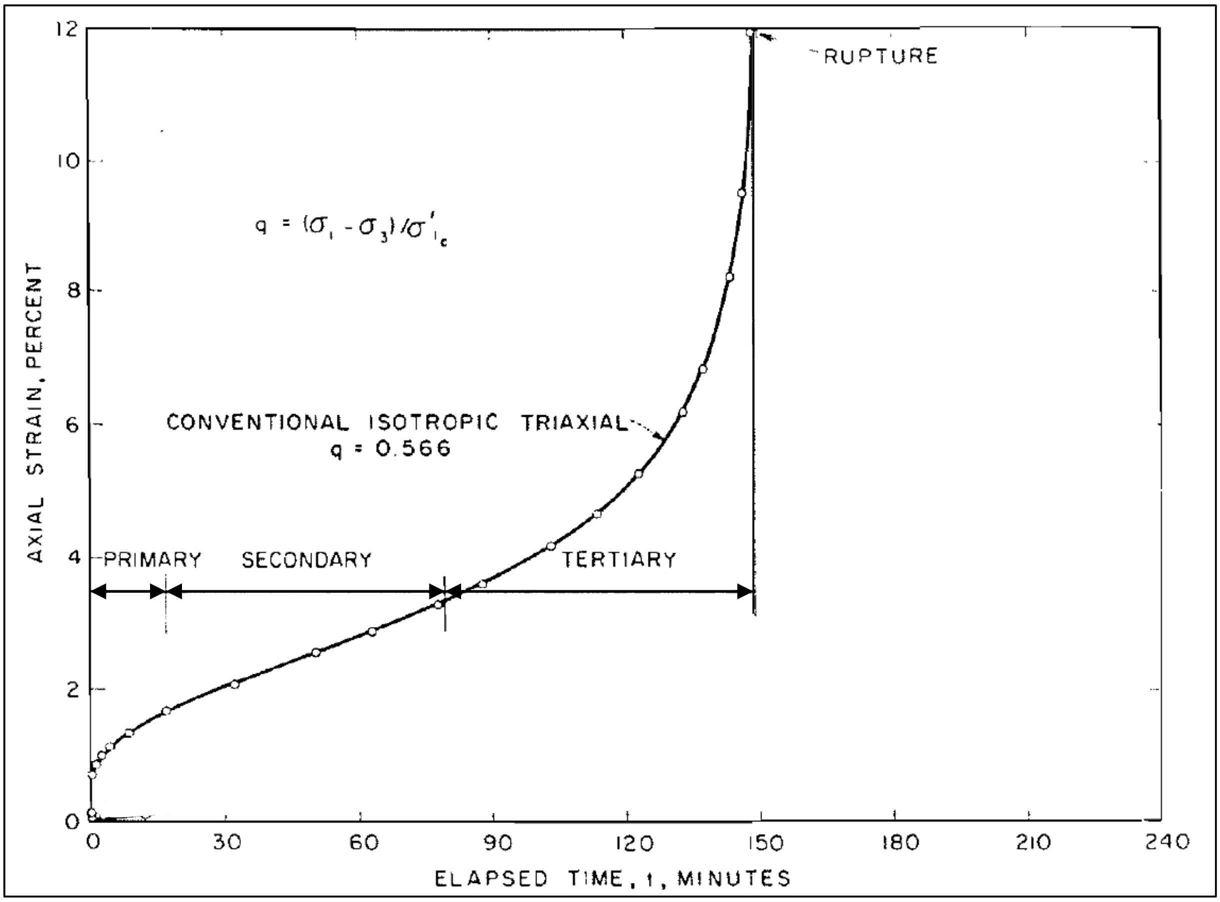


Figure 2.4 Different stages during creep process (Campanella and Vaid 1974)

2.5 Creep rupture life

It is the component part of the overall creep process. It is defined as the log-log plot of the total rupture life against the minimum creep rate as shown in figure 2.5. The total rupture life is the time elapsed from the initiation of the creep to the instant of rupture. Figure 2.5 shows that there is linear relationship between the creep rupture life and minimum strain rate in log-log plot of the total creep rupture life and corresponding minimum strain rate.

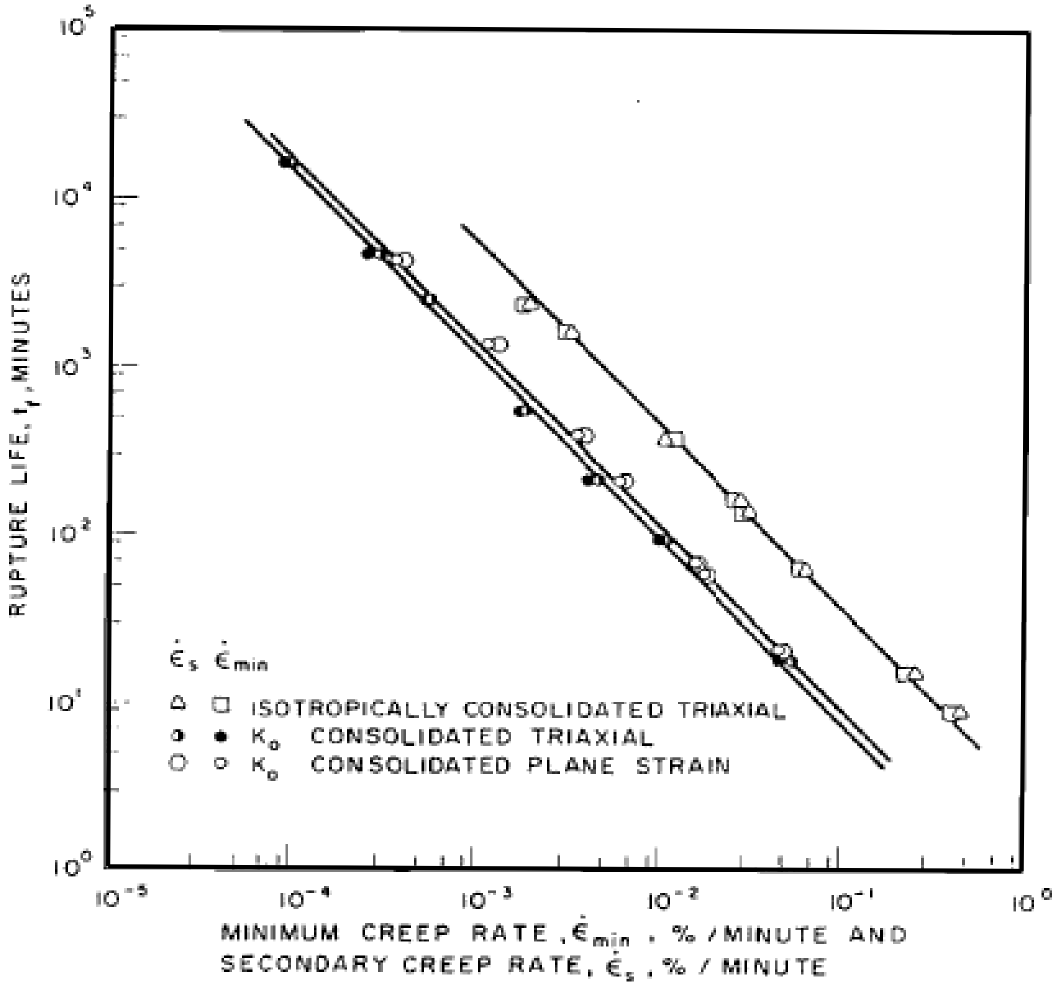


Figure 2.5 Relationship between rupture life and minimum creep rate –normally consolidated undisturbed Haney clay, (Campanella and Vaid 1974)

Minimum creep rate or minimum strain rate ($\dot{\varepsilon}_{min}$) is the strain rate during the secondary creep stage. It also shows the resemblance between the result of the K0- consolidated triaxial test and the plain strain test and the large deviation of both test from the conventional creep test ***(Campanella and Vaid 1974).***

2.6 Remaining creep rupture life

It is difficult to find out the start of the creep in the creep problems of the earth structure. Hence, total creep rupture life is less used to make the estimates in the creep problems. The accelerating strain rate indicates the impending failure and remaining life of the sample can be determined but for the decreasing strain rate, the remaining creep rupture life cannot be determined because the sample may not fail at all. ***(Campanella and Vaid 1974)***, in Haney clay, investigated and found that it hold the linear relationship between the creep strain and logarithmic remaining rupture life as shown in figure 2.6 . The graph shows that the slope is independent of the stress level for the given type of test.***(Campanella and Vaid 1974)***

i: e

$$\frac{\partial \varepsilon_1}{\partial\left(\log\left(t_{rf}\right)\right)} = -k \tag{3}$$

$$\boldsymbol{\dot{\varepsilon}_1 * t_{rf} = k/2.303} \tag{4}$$

Here t_{rf} is the remaining creep rupture life in minute

k is the constant and $\dot{\varepsilon}_1$ is the creep strain rate in % per minute.

k=1.54 for K0 consolidated triaxial test

k=4.54 for isotropic consolidated triaxial test

k=1.10. For K0 consolidated plane strain test

Figure 2.6 q represents the magnitude of the creep stress and expressed as the principle stress difference normalised with the effective vertical stress during consolidation, i.e $q = (\sigma_1 - \sigma_3)/\sigma'_{1c}$ where subscript ‘c’ represents that effective vertical stress is for consolidation phase. Eqn (4) shows that the strain rate and the remaining rupture life holds the inverse relation.

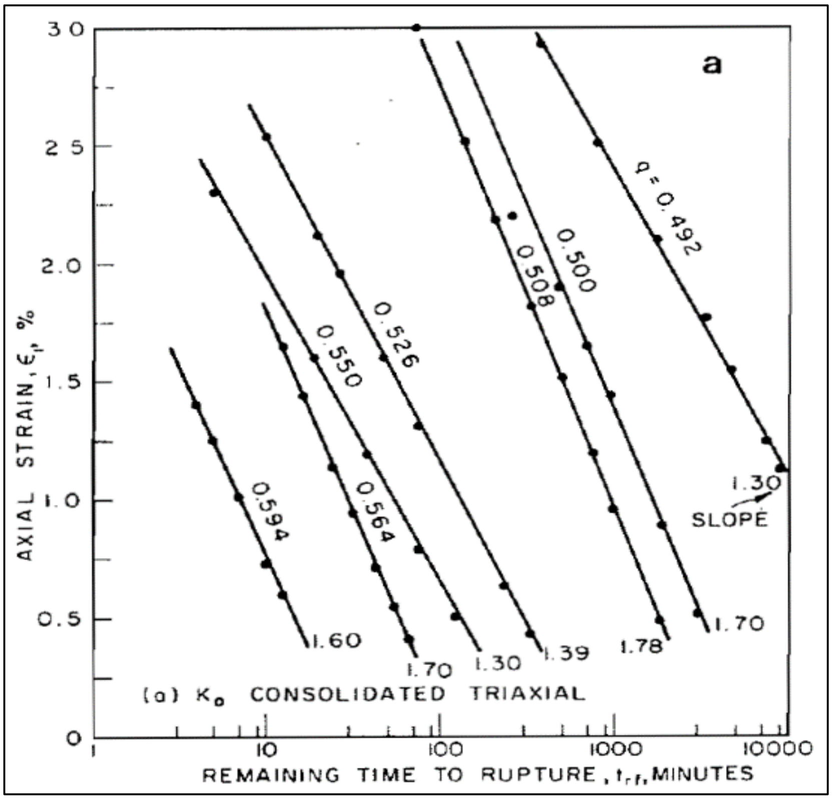


Figure 2.6 Relation between axial strain and remaining time to rupture during tertiary creep, normally consolidated Haney clay (Campanella and Vaid 1974)

2.7 Calculation of creep deformation

Janbu's method

This method is mostly used in Norway. It uses the time resistance R to determine to study the secondary deformation process and the long-term creep deformation. Time resistance R can be defined as the inverse of the strain rate and it is determined from the oedometer test ***(Nordal 2013)***.

$$time\ resistance\ (R) = \frac{dt}{d\varepsilon} \tag{5}$$

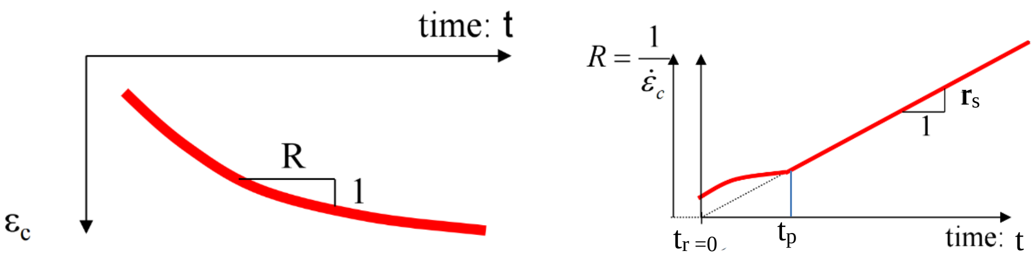


Figure 2.7 Time resistance (R) and resistance number (r_s) (Nordal 2013)

$$Secondary\ strain\ (\varepsilon_s) = \frac{1}{r_s} * \ln\left(\frac{t - t_r}{t_p - t_r}\right) \tag{6}$$

$$\text{creep deformation } (\delta_s) = \int_0^H \varepsilon_s \, dH \tag{7}$$

Here

r_s *is the resistance number(dimensionless)*

t_r *is reference time*

t_p *is the time for end of primary consolidation*

2.8 Time Dependent Phenomenon in clay

During the 1930s it was recognised that the volumetric deformation of cohesive soils, under constant effective stress increments in the oedometer, does not cease after the dissipation of excess pore water pressure but it continues for very long periods of time. ***(Athanasopoulos and Richart 1983)***. ***(Taylor 1942)*** developed a conceptual description of time effects on the deformation and the strength behaviour of clay that is still valid. He showed that any clay would present not a unique relationship between its void ratio e and the applied effective stress but rather a specific relationship for possible durations of load application. From this simple and classical assumption, secular settlement would develop according to the function of the form

$$\Delta e = C_\alpha * \log((t_i + t)/t_i) \tag{8}$$

where the coefficient of secondary consolidation $C\alpha$ would be constant and t_i is the reference time and t is the duration of loading. Taylor showed that e-σ_v'-t relationship would take the form of a set of parallel lines in e-logσ_v' space.

The important consequence of this phenomenon is that with the time, there is the development of bonds ***(Taylor 1942)*** or reserve resistance ***(Bjerrum 1967)*** in the aged clay which is evidenced as a preconsolidation pressure (σ_p) during the further loading of clay consolidated under σ_v'. When loading an overconsolidated aged clay, the actual preconsolidation pressure will be the function of the age of the clay first and then, of the duration of loading, decreasing linearly with the duration of loading or with strain rate. Hence the entire limit surface of the natural clay is age and rate dependent ***(Tavenas, Leroueil et al. 1978).***

2.9 Insitu stress condition in clay slopes

(Rankka 1994) investigated on insitu stress condition in clay slopes and how it varies during changes such as seasonal variation in pore water pressure or excavation or filling of earth material. She investigated on the 7 test sites, 6 in Gothenburg and 1-235 km north of Gothenburg. From the investigation, she found that the total horizontal stress is higher in passive zone than in active zone and the differences varies from 2KPa and 30KPa. The change in the total stress with the depth was lower in active zone than in passive zone. She also plotted the effective horizontal stress against height above the mean sea level and found that effective horizontal stress at the given height above mean sea level is same irrespective of the location on the slope. She also studied on the change in the horizontal stress due to the man made changes in the slope and measurement showed that the man made changes have the effect in the horizontal stress and pore pressure only at the area near the earthwork.

2.10 Undrained shear strength

Shear resistance is the shear stress that is mobilized as the displacements is applied to the soil. The shear strength refers to the peak or failure shear resistance only ***(Graham, Crooks et al. 1983)***. The undrained shear strength is taken as one half of the principle stress difference at failure, in undrained triaxial tests as shown by equation 9

$$su = \frac{1}{2}(\sigma_1 - \sigma_3)_f \tag{9}$$

2.11 Strain rate effect on undrained shear strength

The rate of loading or strain rate affects the shear strength of the clay. Strain rate effects have two components; one is due to the partial drainage and another is viscous effect. Partial drainage is not the significant source if error in triaxial test since drainage can be controlled in the laboratory. Viscous effects can lead to the significant difference in the strengths from triaxial tests and the strength that can be mobilized in the field ***(David A. Varathungarajan 2008/2009).***

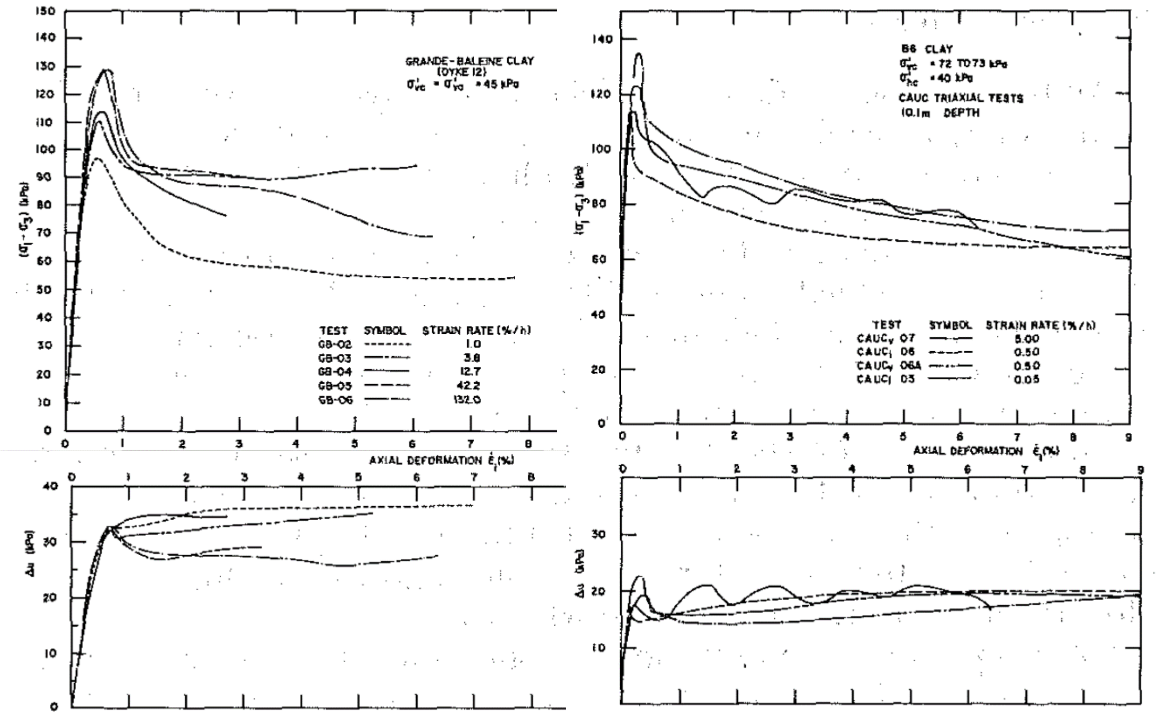


Figure 2.8 Stress – strain and pore pressure strain curves for structured clay (left) and normally consolidated clay (right) (Lefebvre and Leboeuf 1987)

A series of monotonic and cyclic triaxial tests were carried out to study the influence of the rate of strain and load cycles on the undrained shear strength of three undisturbed sensitive clay from eastern Canada. It revealed that, for structured clays (naturally overconsolidated), the peak strength envelope is lowered as the strain rate is decreased but the pore water pressure generated at a given deviatoric stress is essentially independent of the strain rate as shown in figure 2.8. But, for destructured clay (normally consolidated), lower strain rate increase the pore pressure. During shearing because of the tendency of the clay skeleton to creep but the peak strength envelope remains same. However from the quantitative point of view, the shear strength is increased with increase in the strain rate both structured and normally consolidated clays in similar manner ***(Lefebvre and Leboeuf 1987).***

2.12 Coefficient of earth pressure at rest (K_0)

The ratio of horizontal to vertical effective stresses in soil is known as the coefficient of earth pressure at rest, K0:

$$K_0 = \frac{\sigma_h{}'}{\sigma_v{}'} \tag{10}$$

Coefficient of earth pressure at rest for normally consolidated Soils

It has been established empirically that the value of K_0 during one dimensional normal compression known as $K_{0,NC}$, is constant for a given soil. Some of the most widely used relationship for estimating $K_{0, NC}$ is provided below

(Jaky 1944)

$$K_{0,NC} = (1 + \frac{2}{3} * \sin \varphi'_{crit}) * \left(\frac{1 - \sin \varphi'_{crit}}{1 + sin\varphi'_{crit}} \right) \tag{11}$$

This can be approximated by the equation

$$K_{0,NC} = 0.95 - \sin \varphi'_{crit} \tag{12}$$

(Brooker and Ireland 1965)

$$K_{0,NC} = \left(\frac{1 - \sin(\varphi'_{crit} - 11.5^0)}{1 + \sin(\varphi'_{crit} - 11.5^0)} \right) \tag{13}$$

(Bolton 1991)

$$K_{0,NC} = \left(\frac{\sqrt{2} - \sin \varphi'_{crit}}{\sqrt{2} + sin\varphi'_{crit}} \right) \tag{14}$$

Coefficient of earth pressure at rest for over consolidated soils

For overconsolidated soils K_0 can be calculated from values of $K_{0,NC}$ and OCR. Widely accepted formulas are as follows

(Wroth 1978)

$$K_0 = OCR * K_{0,NC} - \frac{\vartheta}{1-\vartheta}(OCR - 1) \text{ where } \vartheta \; is \; poisson's \; ratio. \tag{15}$$

(Schmidt 1966) (K_0 for clays on unloading)

$$K_0 = K_{0,NC}(OCR)^{\alpha}, where \; \alpha = \sin(1.2 * \varphi'_{crit}) \tag{16}$$

(Prŭska 1973)

$$K_0 = \frac{\sqrt{K_a} * OCR}{1 - K_a * (1 - OCR)}, \tag{17}$$

$where \; K_a \; is \; the \; Rankine \; earth \; pressure \; coefficient$

$$K_a = \frac{1 - sin\varphi'}{1 + sin\varphi'}$$

where φ is the internal frictional angle.

2.13 Preconsolidation stress

It is the maximum vertical overburden stress that a particular soil sample has sustained in the past. Preconsolidation stress in a soil is due to change in the total stress, change in pore water pressure, change in soil structure due to aging (secondary compression), environmental change and chemical weathering ***(Holtz and Kovacs 1981)***. Preconsolidation stress is one of the main parameter for estimating deformation of clay material. It marks the transition between the material of over consolidated (OC) area and normally consolidated (NC) area. It is the parameter used in oedometer test and indicates the point from where the material begins to flow i:e where the material suffers the large plastic strains. This stress point is called yield point or yielding point ***(Wood 1990)***. Preconsolidation stress can be determined from the oedometer test either by Constant rate of strain (CRS) or incremental load (IL). Preconsolidation stress is the function of age of the clay, time of loading or strain rate and temperature.

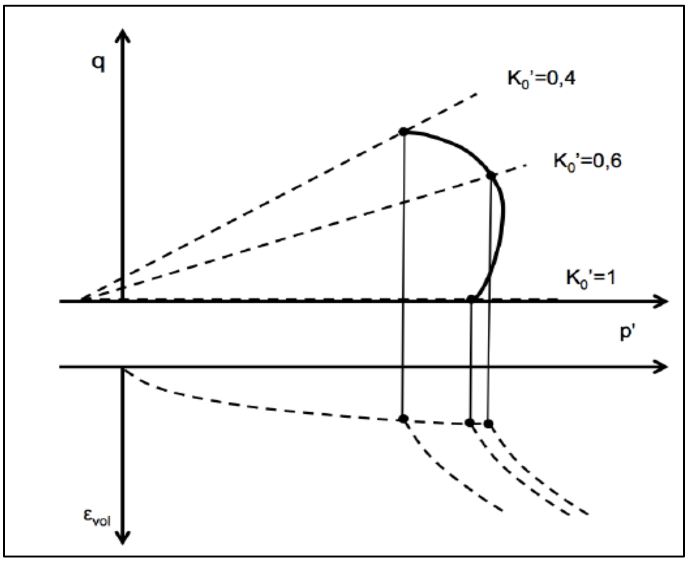
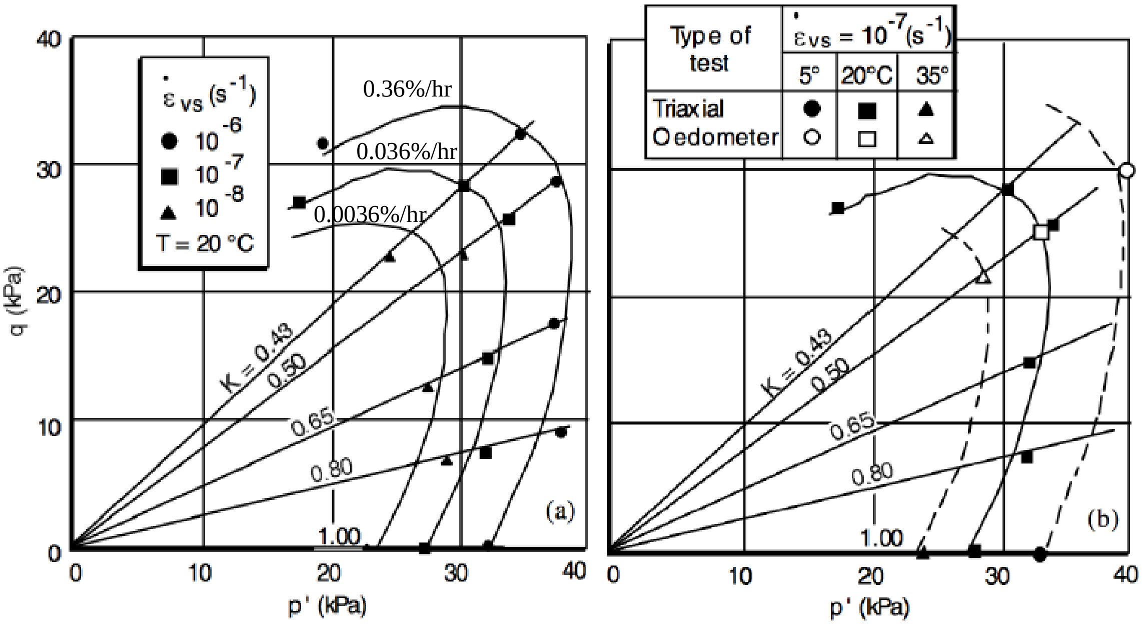


Figure 2.9 shows Different yield points for different K=σ3'/σ1' –ratio tests indicate that a Preconsolidation surface exists in the p'-q space (Nordal 2013)

Figure 2.9 is the principle test results from different drained test at a constant horizontal stress ratio K= σ_3'/σ_1'. The stress-strain graph shows the preconsolidation stress as transition point from the stiffer elastic response to the softer elastoplastic response.



Flow surface at different strain rate   Flow surface at different temperature

Figure 2.10 Preconsolidation stress at different temperature and strain rate (Boudali 1998)

Figure 2.10 shows the flow surface or the preconsolidation stress at different horizontal stress ratio when there is variation in the strain ratio and temperature. It shows that the preconsolidation stress increase with the increasing strain rate and decreasing temperature ***(Boudali 1998)***.

Crawford (1963) studied on pore pressure variation inside the triaxial specimen and found that the pore water pressure inside the triaxial sample is temperature dependent. He plotted the variation of the pore water pressure with the temperature as shown in figure 2.11. The change in pore water pressure is directly proportional to the change in laboratory temperature. It shows that for every 4-degree rise/fall in temperature, the pore pressure will increase/decrease by 1.5psi.

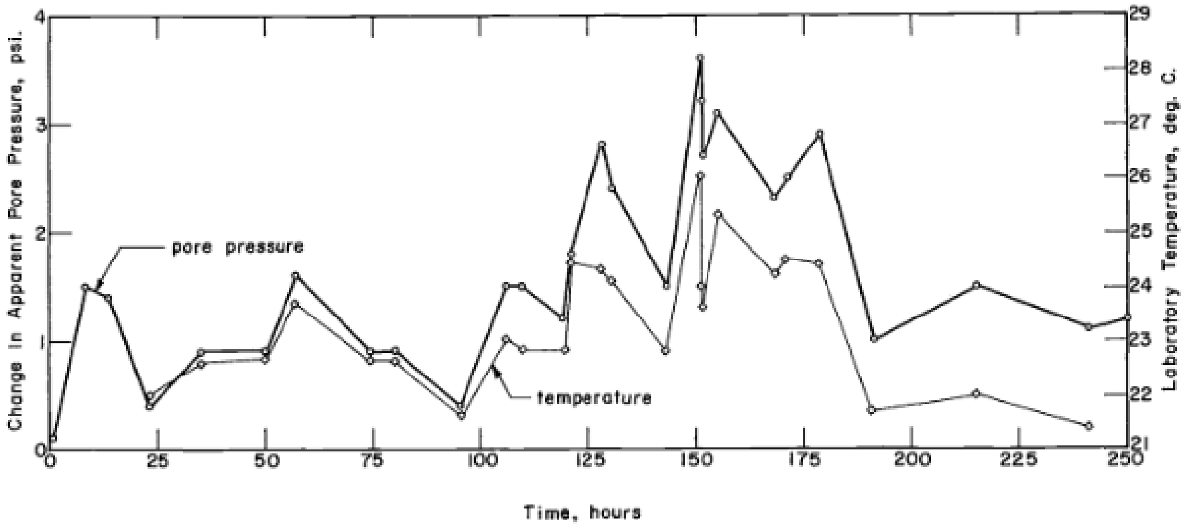


Figure 2.11 Influence of temperature on apparent pore water pressure in illite specimen (Crawford 1963)

2.14 K0-consolidated triaxial test

There are different techniques to deal with the creep but the data required for these techniques cannot be obtained from the conventional triaxial test and unconfined compression test because the consolidation stress in these test does not corresponds to the insitu situation. In the nature, clay is normally consolidated under the one-dimensional strain condition i: e K0- consolidated. Many earth structure problems are closely approximate to plain strain and are rarely represented by the axially symmetrical triaxial test. Thus, plain strain creep test on initially K0 consolidated samples would be the most appropriate laboratory test for the study of the creep rupture.

In normally consolidated clay with fully or partial drainage condition, the volume of the clay decrease and strain increase consequently with time. But in the undrained creep the drainage condition is under control which results in creep rupture. Also, in case of the deep thick beds of the normally consolidated clay, due to the low permeability and the long drainage path, it does not allow the drainage of the pore water and thus can be simulated by the undrained creep rupture ***(Campanella and Vaid 1974)***.

Also,***(Campanella and Vaid 1974)*** carried out the conventional triaxial test and the K0 consolidated triaxial and plane strain test in Haney clay (undisturbed clay) and compared results with one another. He found that the results from the k0 consolidated triaxial test and the plane strain test resemble with one another but both are different from the result of the conventional triaxial test as shown in the figure 2.12

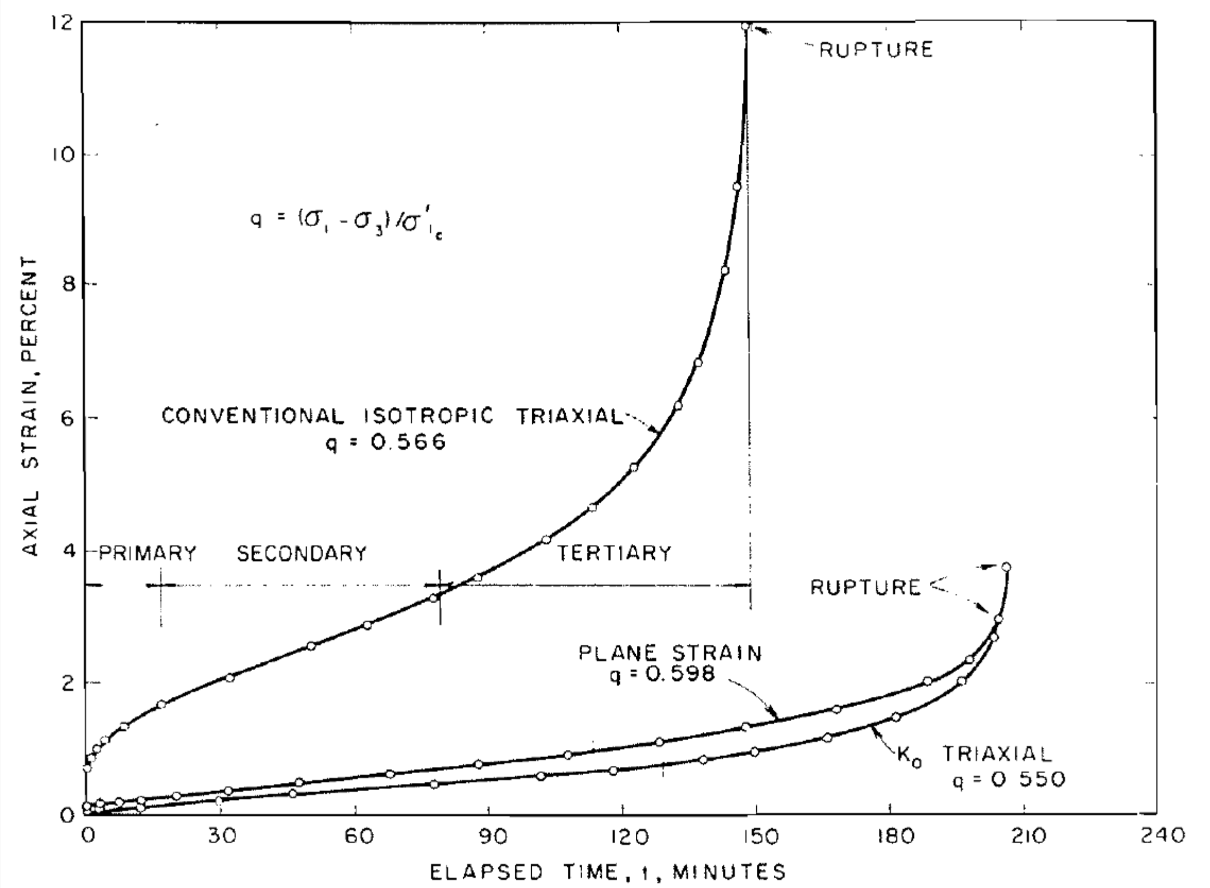


Figure 2.12 Typical creep curves for isotropically and K0 consolidated triaxial and K0 consolidated plane strain samples-normally consolidated undisturbed Haney clay (Campanella and Vaid 1974).

2.15 Drained and undrained test for shear creep

Intact, over consolidated Saint-Alban clay was investigated for the creep behaviour by the means of many drained, undrained test and oedometer test. Both in drained and undrained test, clay samples were consolidated to the same initial effective stress level. The results from the both test were plotted and compared as shown in figure 2.13 and 2.14. Figure 2.13 and 2.14shows the change in axial strain rate with time in logarithmic scale for the different total vertical or axial stress. Both drained and undrained test results show the linear decrease in the logarithmic values of the strain rate and time and the slope of the line is constant for the different axial stress level. The value is of the order 0.78 which is close to the value obtained from drained test *(Tavenas, Leroueil et al. 1978)*.

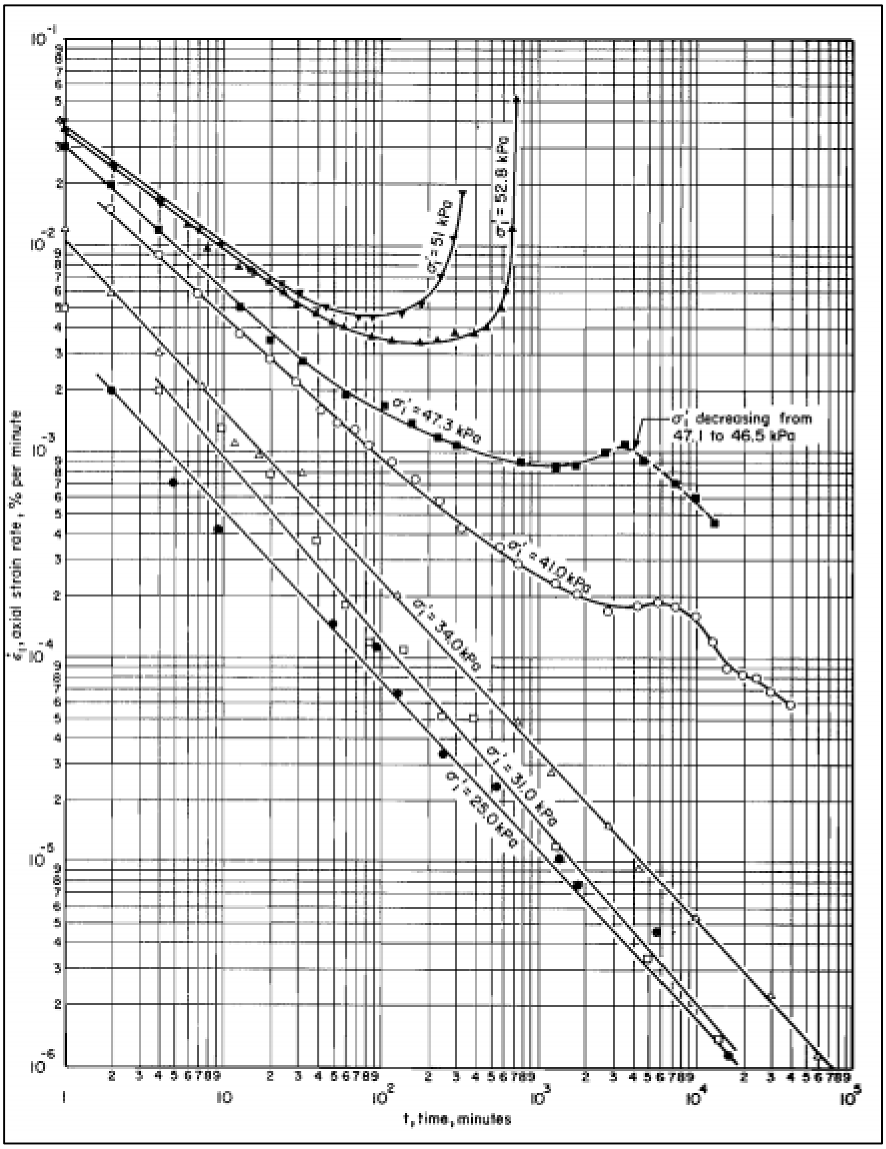


Figure 2.13 Axial strain rate – time relationship for drained tests at $\sigma_3' = 16.5$KPa (Tavenas, Leroueil et al. 1978)

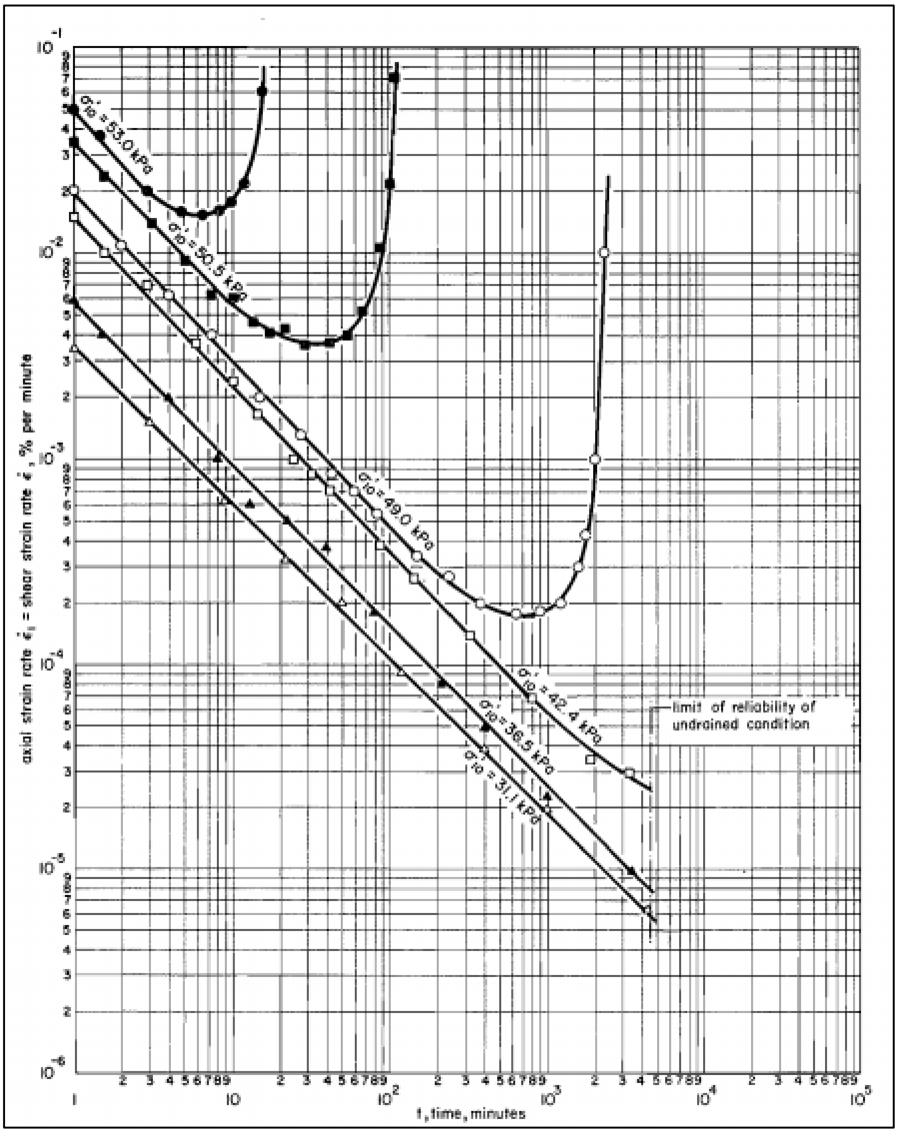


Figure 2.14 Shear strain rate – time relationship for undrained tests at σ3'=16.5KPa (Tavenas, Leroueil et al. 1978)

Relation for obtaining shear strain indirectly from axial strain ***(Tavenas, Leroueil et al. 1978)***

$$shear\ strain\ (\varepsilon) = \varepsilon_1 - \frac{1}{3} * v \quad (18)$$

$$shear\ strain\ rate\ (\dot{\varepsilon}) = \dot{\varepsilon}_1 - \frac{1}{3} * \dot{v} \quad (19)$$

Here

ε_1 *is axial strain*

$\dot{\varepsilon}_1$ *is axial strain rate*

v *is volumetric strain*

$\dot{v}$ *is volumetric strain rate*

But figure 2.15 shows the variation of axial strain rate with shear stress at different time. Here, the axial strain rate is considered to be the shear strain rate initially. Dash line represents the drained test and solid lines represent the undrained test. AB line is for axial strain at time 100 min for drained test and PQ is for axial strain at 100min for undrained test. There is a linear relation between the logarithmic axial strain rate with the shear stress for both drained and undrained test. However, the slope for the drained test seems to be little greater than for the undrained test (when AB and PQ line is compared). Moreover, for the given stress level at a specific time, the axial strain rates for drained test are greater than those in undrained test.

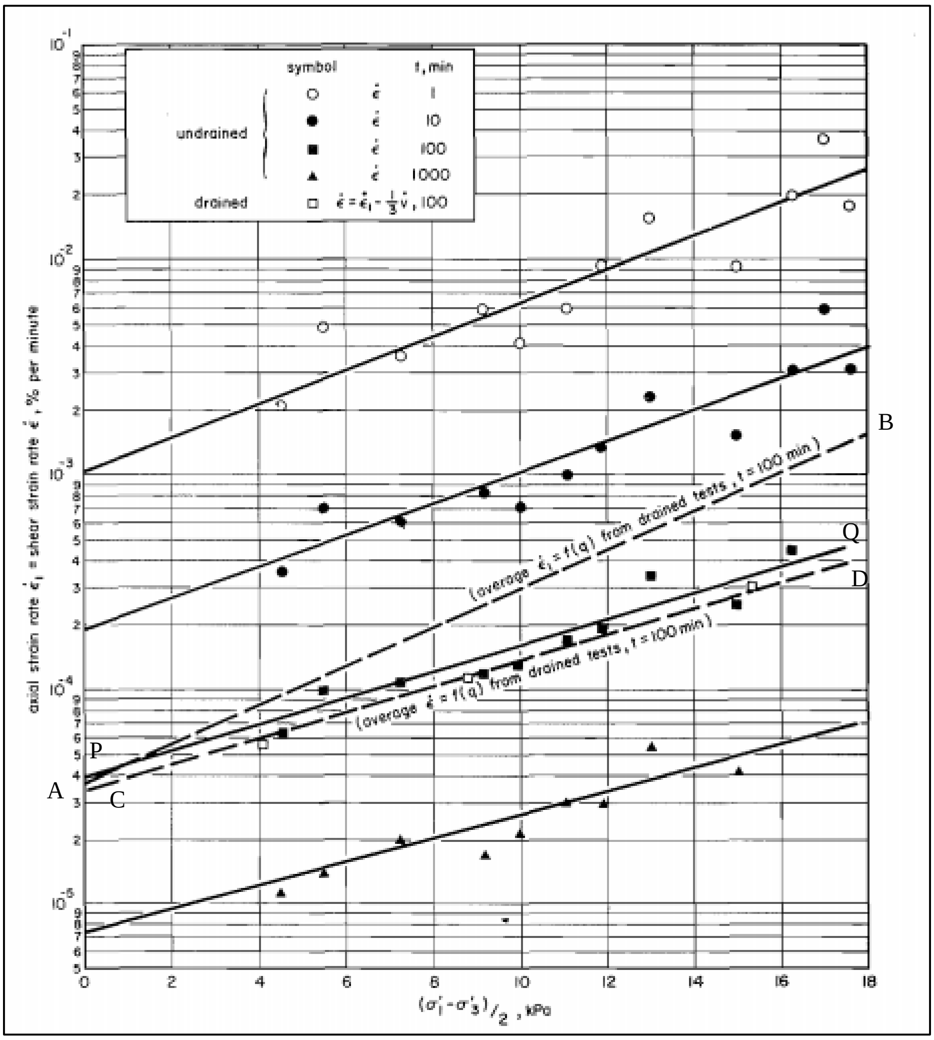


Figure 2.15 Shear strain rate vs shear stress for undrained and drained tests at σ3' = 16.5 KPa (Tavenas, Leroueil et al. 1978)

Hence this difference in drained and undrained test results is due to the contribution of the volumetric strain in the drained test, which is best explained by equation 18 and 19. It has also shown that when the volumetric strain effect is reduced from the drained test results then the modifications are in best agreement the results of undrained test (volumetric effect is reduced from line AB. Then AB line reduces to be CD line which fits close to line PQ). These results has also given validity to the separation of total strain into volumetric and shear strain. The consideration that is made before is valid for the undrained test only but for drained test axial strain is the combine effect of shear strain and volumetric strain. Therefore for the shear creep both drained and the undrained test can be performed. In undrained test, the volumetric strain is zero and we encounter shear strain only but for the drained test both volumetric and shear strain is encountered but it has been observed that the undrained tests do not remain as such after 3-4 days. Therefore, both drained and undrained test can be done for the shear creep test ***(Tavenas, Leroueil et al. 1978)***.

2.16 Pore pressure inside Soil

Pore water pressure inside the soil is due to the mechanical and physiochemical effects. If there is no flow of pore water inside the soil mass i.e. equilibrium condition, total pore water pressure remains same at any point inside the soil. The pore water pressure has different components that is related to stress between particles and that combine to give total pressure. Pore water pressure can be divided into four components

1. Pressure due to gravity
2. Hydrostatic pressure
3. Pressure due to osmotic or ionic concentration difference effects
4. Pressure due to adsorptive pore field.

Each components of total pressure cannot be measured directly except in special system. The change in one component causes change in other component in other point to give the same total pore pressure at different point ***(Mitchell 2013).*** From the effective stress principle, the strength of the soil does not depend upon the total stress applied on the soil but depends upon the difference between the total stresses applied and pore water pressure inside the soil as given by eqn (20) as per the principle of effective stress stated by Terzaghi in 1936 ***(Skempton 1960)***

$$\sigma' = \sigma - u \tag{20}$$

Here, σ *is the total stress applied to soil*

u *is the pore water pressure in the soil.*

"Analysis of the forces acting within a soil mass across a surface which approximates a plane, but passes through the pore space and points of interparticle contact, indicates that the average intergranular force per unit area of the horizontal projection of the plane is given by" ***(Mitchell 2013),***

$$\sigma'_i = \sigma - (1 - a_c)u \tag{21}$$

Here, a_c *is the contact area of the soil particles per unit area of the plane.*

Bishop has demonstrated that the change in volume of the granular soil is only due to $(\sigma - u)$ but not the contact area. He also said that the shear strength of soil depends only upon $(\sigma - u)$ but not upon contact area but this has been remain for conjecture only. However the contact area is so small that eqn (20) and (21) gives almost same value. The forces that come into play in addition to the forces due to the applied load and hydrostatic water pressure is due to the activity of the clay particle at surface. However, principle of effective stress obtained from the pore pressure measurement correlates well with the observed behaviour of the fine grained soil despite of these additional forces.

But the effective stress as stated by eqn (20) is not sufficient for the partially saturated soil. The behaviour of the partially saturated soil can be better explained by eqn (22) ***(Bishop 1960), (Aitchison 1985) and (Croney and Coleman 1953, Jennings 1961)***

$$\sigma' = \sigma - u_{air} + x(u_{air} - u_{water}) \tag{22}$$

Eqn (22) accounts for the pore air pressure different from 1 atm. It is the condition that may easily arise in practise. x is the parameter ranging from 0 to 1 and for dry soil the value of x is 0 and for fully saturated soil the value of x is 1.

The physical significance of the pore water pressure for the system with active physiochemical forces is different from the system that is free from particle surface forces. The pore water in the clay is different from the free water; hence pore water pressure measured should reflect the influence of these force fields along with the stresses induced by the mechanical strain of the system ***(Mitchell 2013).***

2.17 Physical significance of pore pressure measured in the tests

Pore water pressure in clay in strength and compression test is usually measured by the sensing element that is either a porous tip inserted inside the sample or the porous stone at the base of the sample that is connected to the measuring device as shown in figure 2.17.

Here the connection is made between the sample and null point A as shown in figure 2.17. The back pressure required to maintain the level at point A is the pore water pressure inside the sample. If we consider point A and C (figure 2.17), the pressure measured at point A is the total pressure at point C, at equilibrium condition. However the pore pressure component may vary from point to point within the sample. The pore pressure measured includes the hydrostatic pressure, osmotic pressure and the absorptive pressure. During the consolidation and shearing phase of the soil, pore pressure measured should reflect the change in the hydrostatic component of the pore water pressure because it is the hydrostatic pressure that is experienced by the particles in response to the change in thrust and that influence the static equilibrium of the structure.

If the free water is used in the connection and the clay contains the salt then there will be the difference in the concentration of the ions. Since the porous stone can easily pass the ions, there will be the flow of ions from sample to the connection and it takes long time to come into

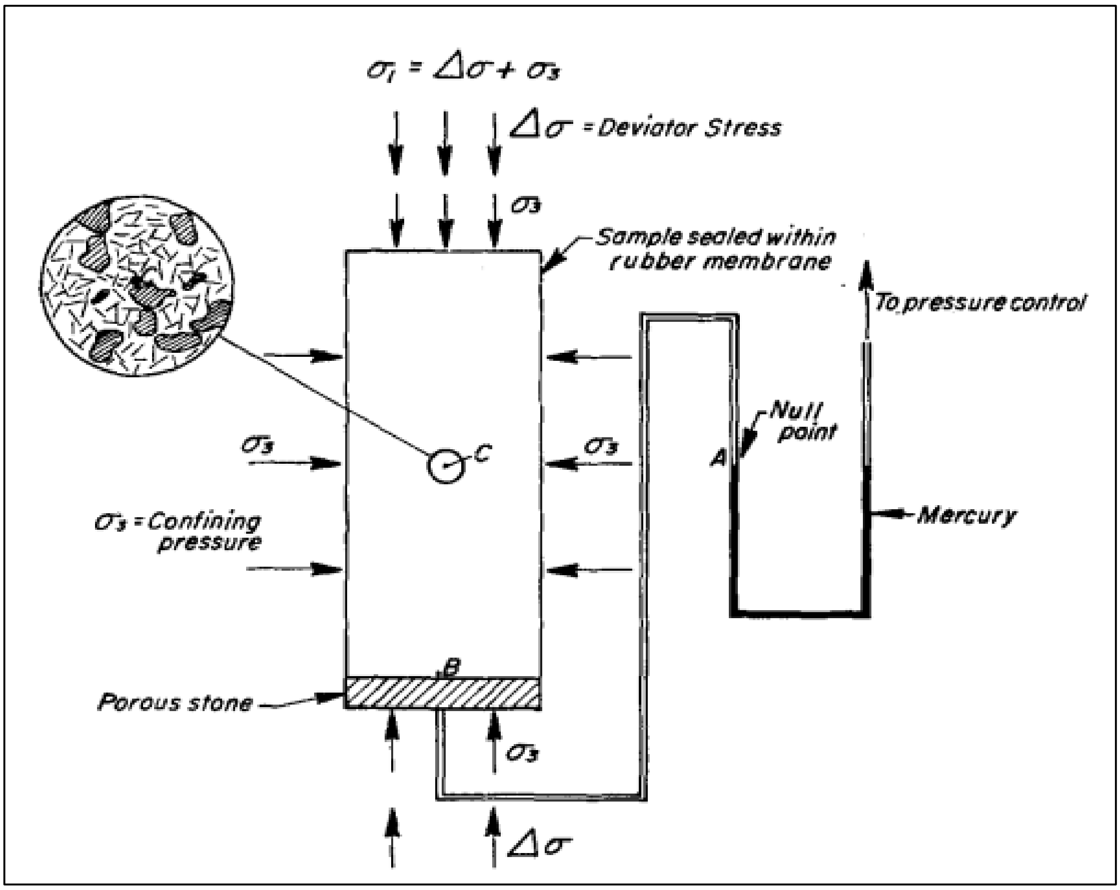


Figure 2.17 Pore pressure measurement in triaxial test (Mitchell 2013)

equilibrium condition. If we considered the point B and C (figure 2.17) then the pore pressure measured will be relative and also the water in both points are same and have same concentration of ions and equilibrium condition is met easily ***(Mitchell 2013).***

2.18 Negative pore water pressure in the soil

Negative pore water pressure exists inside the soil and the major cause of this is the osmotic effect, adsorptive effect and the surface tension of the water. Mechanical effects like the tendency of the bent soil particles to straighten when the load is released or the tendency of the soil structure to dilate when it is sheared, also causes negative pore water pressure inside the soil. All these effects are related to the absorptive force between the soil particles and pore water. Negative pore water pressure can be in saturated soil or partially saturated soil.

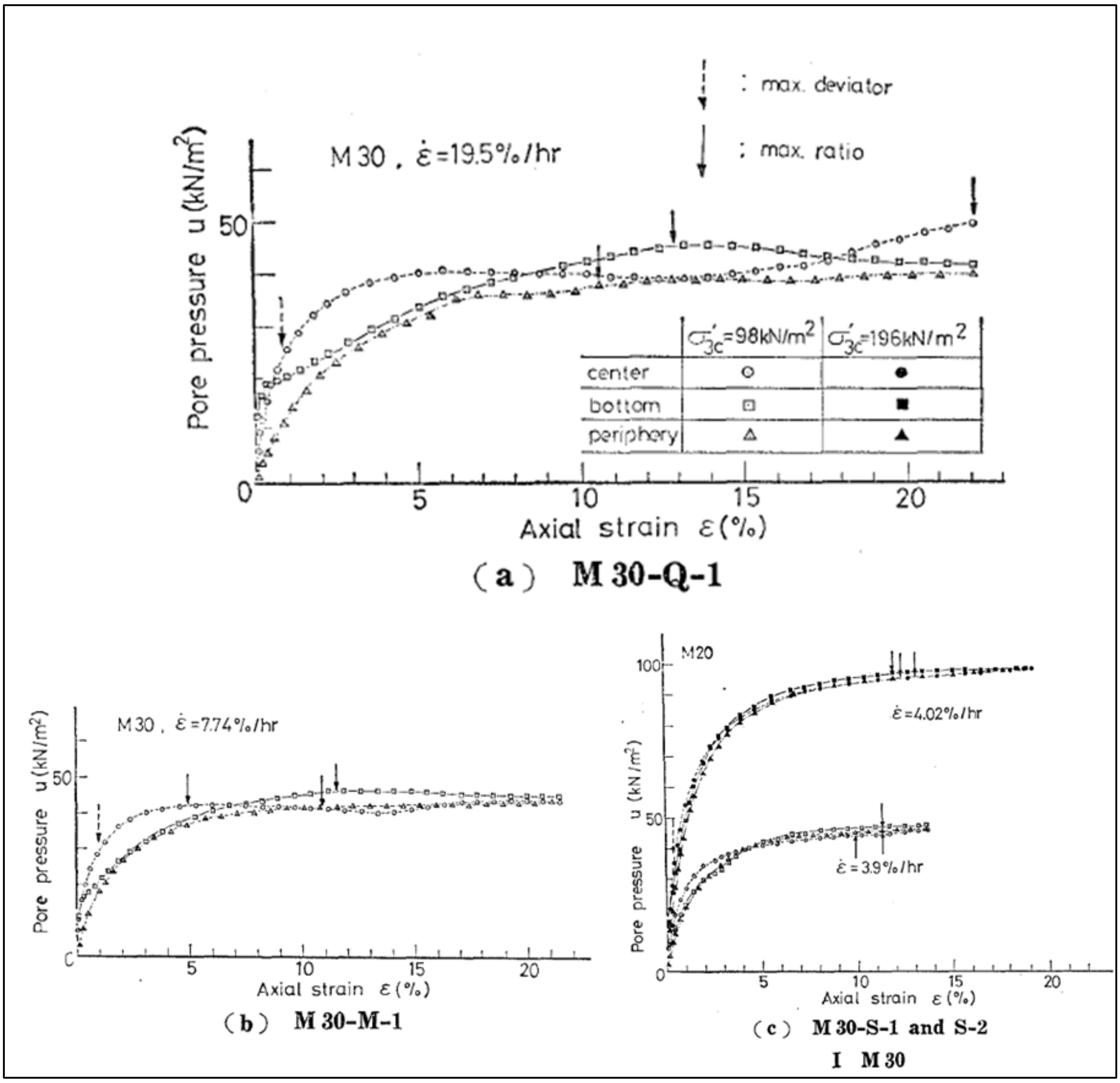
In case of saturated soil, the main cause of negative pore pressure is osmotic effect and hydrostatic effects resulting from the load release in bent or distorted soil particles or dilating tendency on shearing of soil structure.

In case of partially saturated soil, surface tension of pore water in connection with the absorptive effect at the particle surface is the major cause of the negative pore water pressure ***(Mitchell 2013).***

2.19 Pore Pressure inside the triaxial specimen

The pore water pressure inside clay in triaxial apparatus is supposed to be identical theoretically as discussed in section 2.15. But, researchers have found that the pore water pressure inside the clay in triaxial apparatus varies at every point and the measurements made by the pore pressure sensor have proved it. Therefore, the pore water pressure measured at any point may not be the representative of all the condition in shear zone ***(Crawford 1963).***The research made by Division of Building Research of National Research Council, Ottawa, Canada, to assess reliability of normal pore water pressure observation during triaxial loading of undisturbed specimen of clay found that the pore water pressure at the bottom of the specimen is higher than pore water pressure at the centre of specimen. The sample used in the research was block samples of Leda clay with water content of 58%, clay content 51%, liquid limit 53%, plastic limit 25% and sensitivity of 50 and salt concentration of 2 gm/litre. The sample was extracted from the depth of 10.065m. Six identical tests were done with single membrane and no side drains and pore pressure sensors, one installed at middle and another installed 2 cm above the bottom at the centre. The tests showed different magnitude of pore pressure for different strain rate at failure in undrained test ***(Crawford 1963)***.

(Kimura and Saitoh 1983) from the soil mechanics group of Tokyo university of Science and Technology had done some experiments to establish the strain rate for the undrained test on the normally consolidated cohesive soils based on the pore pressure measured at the centre, bottom and periphery of the sample. The triaxial sample was 150mm in height and 75mm in diameter. The clay sample had specific gravity of 2.69, liquid limit 55.3%, plastic limit 25.9%, plasticity index 29.4% and clay content of 22.3%. The clay was normally consolidated. The surface of top and bottom platens was smeared with the silicon grease to reduce the effect of the end restraint. The pore pressure at the centre, bottom and periphery of the sample was measured with the pressure transducer. The pore water pressure profile obtained at the different strain rates are shown in figure 2.18



	σ'_{3c}=98kN/m²	σ'_{3c}=196kN/m²
center	○	●
bottom	□	■
periphery	△	▲

Figure 2.18 Pore water variation in triaxial sample at different strain and strain rates (Kimura and Saitoh 1983)

The cell pressure applied to the sample is either 98Kpa or 196Kpa and it had been consolidated along their K0 value that is approximately 0.4. The strain rates used here seems to be very high. The maximum strain rate is 19.5%/hr which is 29.25mm deformation in 1 hr and minimum strain rate is 3.9%/hr which is 5.85mm deformation in 1 hr. at the strain rate of 19.5%/hr , maximum difference in the pore water pressure at the periphery and centre of the sample is 10-15 Kpa at 2% strain. For the low strain rate of 3.9%/hr, this difference is just 4-6Kpa at 2% strain. The pore pressure difference is higher at the low strains than at higher strains. As seen in test result shown in figure 2.18, in quick test pore pressure variation is more significant than in slow test. The pore pressure at the middle of the sample is always greater than the pore water pressure at the periphery and bottom. To maintain the uniform pore water pressure inside the soil sample, triaxial test should be done at low strain rates. The strain rates that are used in normal triaxial test is generally 3%/hr or 1.5%/hr. So, at these strain rates, the difference in the pore water pressure within the triaxial sample is small in value. The corresponding effective stress path for the clay sample is shown in figure 2.19 ***(Kimura and Saitoh 1983)***

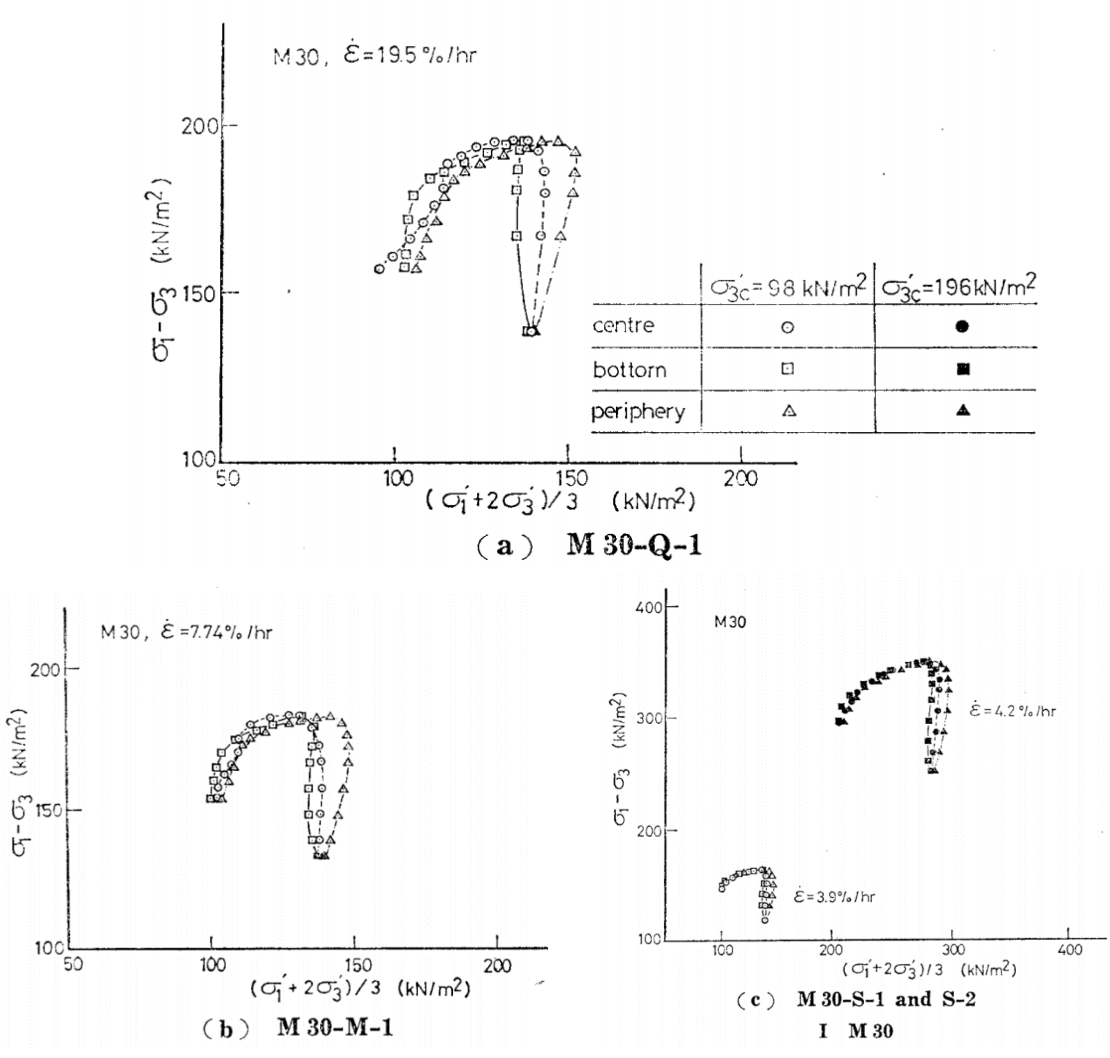


Figure 2.19 Effective stress path (Kimura and Saitoh 1983)

Figure 2.19 shows that the surface of the sample can take higher effective stress than at the bottom and the centre of the sample. The frictional angle at the surface is lower but the difference is not significant.

Filon investigated the effect of the end restraints due to the friction created by the rough platens at the end of the elastic cylindrical material, on the stress inside the material and the results is presented in figure 2.20. Here Q is the applied stress and A is the cross-section area of the sample and F is the frictional force due to the end platens. It shows if the friction is presents at the ends of the sample then it will rotate the major principle axis and soil encounters high resultant stress than applied stress at the ends. If the friction due to end platens is removed, the resultant force at the ends of the sample will be equal to the applied stress and the major principle stress will be in the direction of the applied stress. It shows that the frictional force also causes the variation in the lateral force in the sample *(Crawford 1963)*.

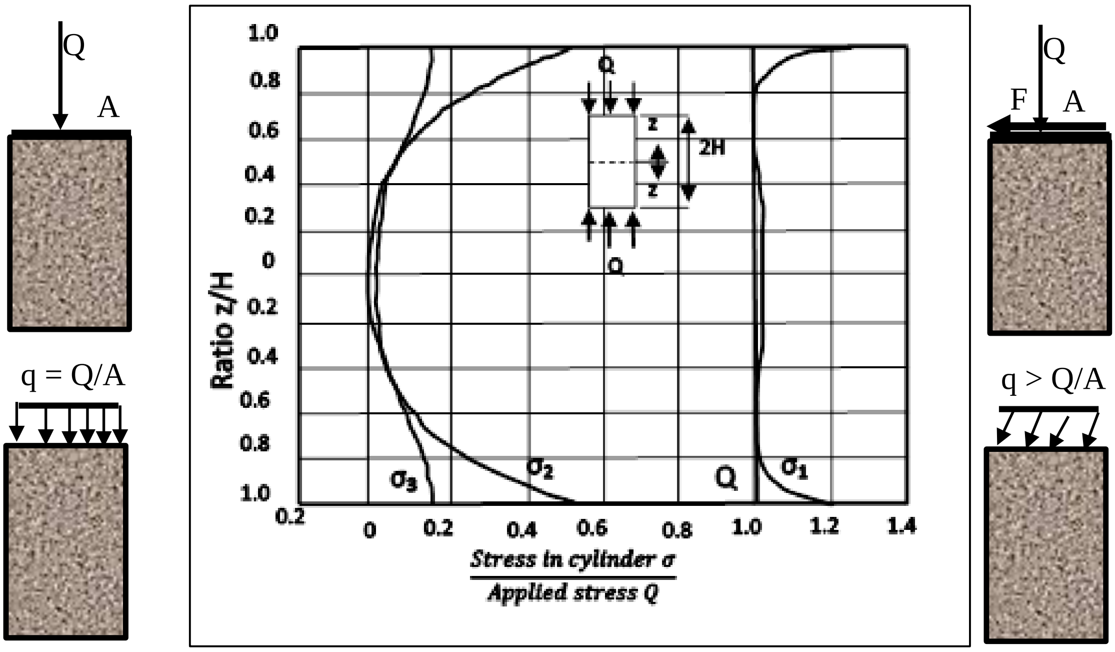


Figure 2.20 Stress in a Cylinder of elastic material axially loaded between rough rigid platens ((Crawford 1963)).

This end friction also causes variation in the pore pressure in the specimen as shown in figure 2.21. The variation is very high for the conventional end platens than the lubricated end platens. It should be reduced to decrease the variation in the pore pressure. Friction can be reduced by the use of the lubricants to smoothen the ends. Acrylic plastics can be used at the top and bottom of the specimen instead of the porous but the use of smooth end increase the difficulty in the experimental technique *(Crawford 1963)*.

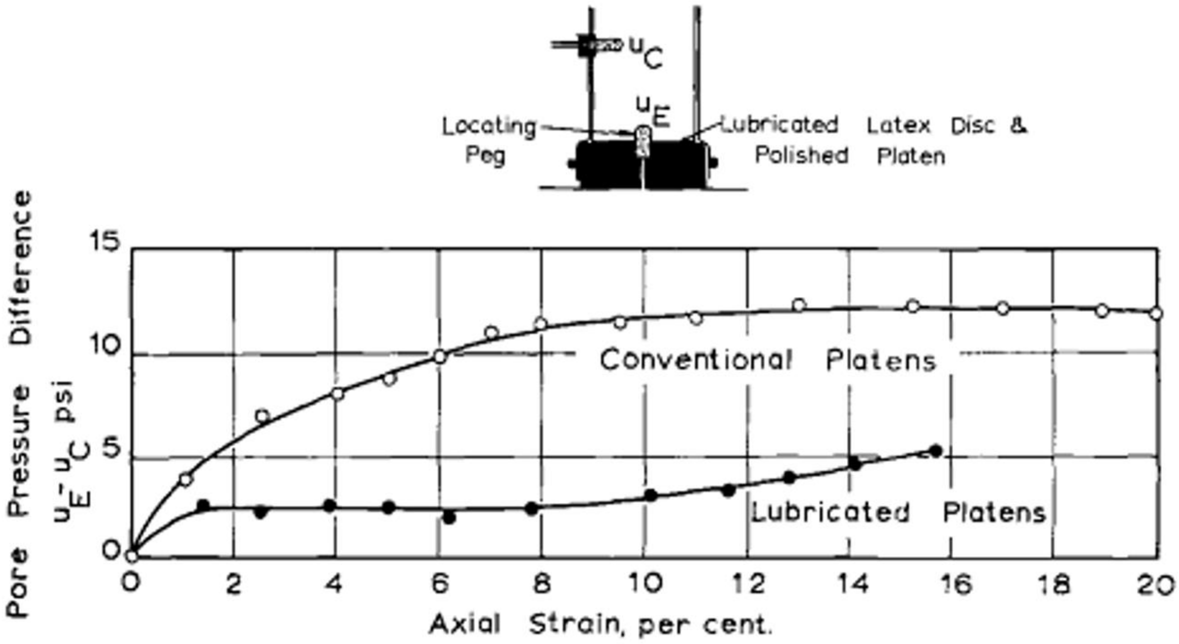


Figure 2.21 Comparison between the maximum pore pressures obtained in triaxial test using conventional end platens and the lubricated end platens (Crawford 1963)

There is a variation in the stress at each point within the specimen is and the pore water pressure inside the soil is dependent upon the change in the stress as shown by equation 23

$$\Delta u = \frac{\Delta\sigma_2 + \Delta\sigma_3}{2} + A * \frac{2 * \Delta\sigma_1 - (\Delta\sigma_2 + \Delta\sigma_3)}{2} \tag{23}$$

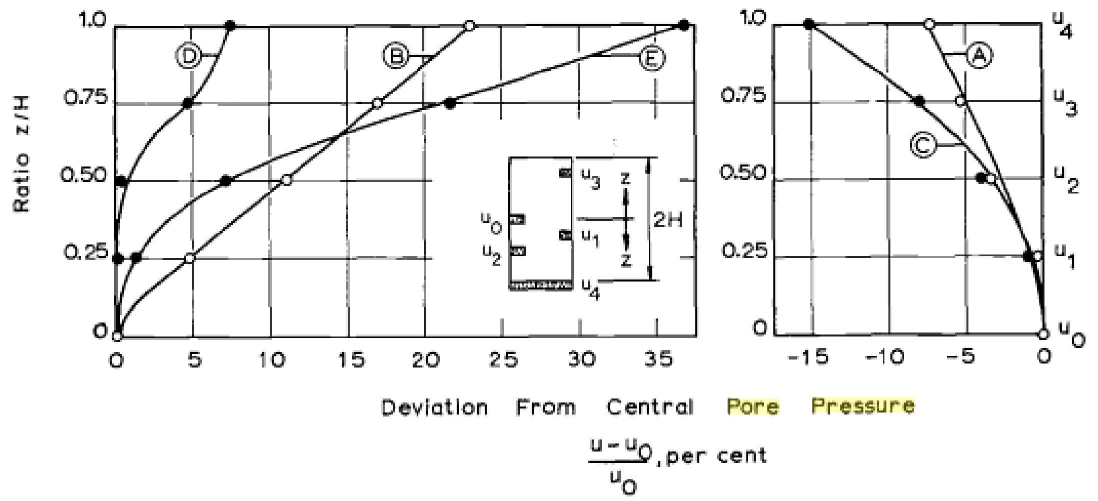
Here, A is the pore pressure coefficient that seems to be dependent on the stress ratio. Hence the pore pressure parameter will also vary with the height of the triaxial sample. The pore pressure profile can be drawn using the equations 24 and 25.

$$A_E = \alpha * A_c \tag{24}$$

$$A = A_c(1 + (\alpha - 1) * (z/H)^2 \tag{25}$$

Here subscript E and C denotes the end and centre of the specimen.

H is half the height of the specimen and z is measured from the centre of the specimen



A. Loading, axial strain 5 per cent.
B. Loading, axial strain 14 per cent.
C. Unloading, axial strain 18.4 per cent.
D. Reloading, axial strain 17.8 per cent.
E. Reloading, axial strain 18.6 per cent.
The degree of equalization of pore pressure at 14 per cent axial strain was 40 per cent.

Figure 2.22 Experimental Pore pressure profiles Measured During Undrained Loading of a Specimen of Saturated Clay (Crawford 1963)

Figure 2.22 shows experiment pore pressure variation at different height of the specimen with respect to the pore pressure at the centre for the different condition of loading and at different strains. The variation is high at the bottom than at others height. The pore pressure at centre is high at low strains than at other height (z), so the variation is negative as shown by curve A but at high strain the pore pressure at centre is low compared to other section as shown by curve B. the pore pressure at the centre is again greater than the pore pressure at the end upon unloading as shown by curve C and get reversed when reloaded as shown by curve D and E. it shows that the pore pressure inside the specimen is strain dependent and the difference in pore pressure from the pore pressure at centre increases with the increase in the strain ***(Crawford 1963)***.

Chapter 3

3.0 Procedure for Triaxial Creep test

3.1 Site location

The soil sample was brought from the Goteborg and Tiller for the test. The samples from Goteborg were the block samples and the samples from tiller were piston sample.

3.2 Soil properties

The soil from Goteborg was soft overconsolidated clay from the depth of 6.35m. The water table was at the depth of 1m from the ground. It was non-homogeneous in colour and content, as shown in figure 3.1. Shell fragments were found inside the soil. Soil had small to large voids inside as shown in figure 3.1. Soil contained 74.64% of water by weight. It was very sensitive in nature. The index properties of the soil are given in table 1. The sample from tiller was homogenous, saturated and quick clay with 40% water content and 40% clay content. It was from depth of 11.4m with water table at 0.5m depth.

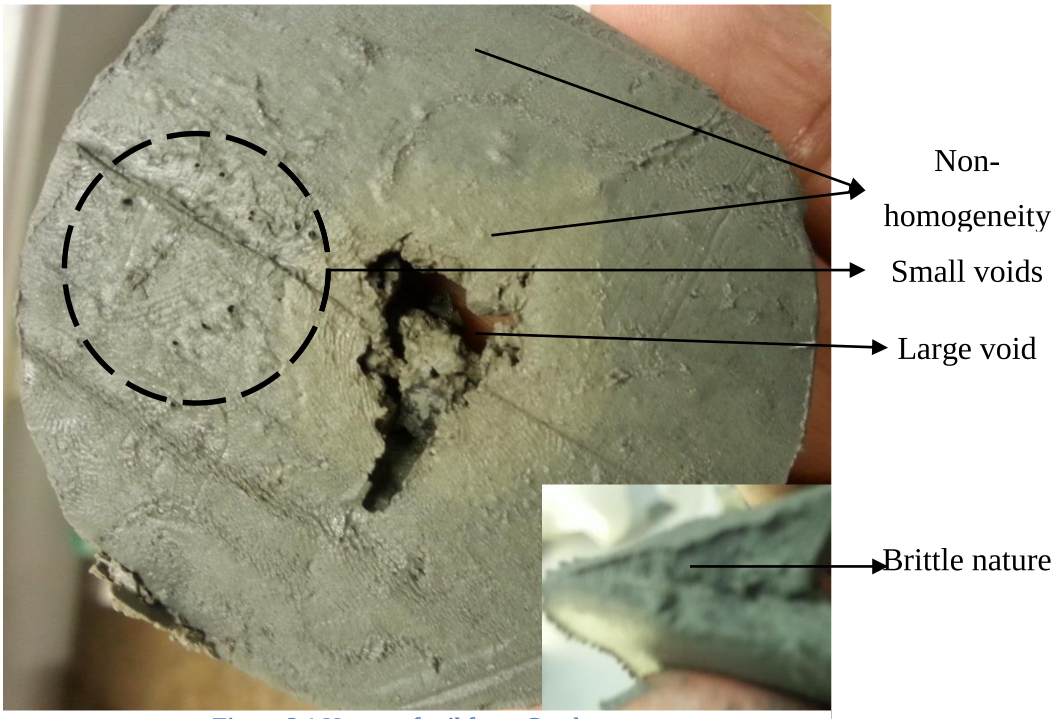


Figure 3.1 Nature of soil from Goteborg

Table 1 Properties of Goteborg and Tiller clay soil

Soil Parameter	Value for Goteborg clay	Value for Tiller clay
Depth	6.35m	11.4m
Unit weight	16 KN/m^3	18.25 KN/m^3
Water content	74.64%	40%
Liquid limit	59.29%	25%
Plastic limit|	24.12%	19%
Plasticity index	35.17%	6.5%
Liquidity Index	1.436	4
Undrained shear strength	24 KPa	42KPa
Remoulded shear strength	1.1 KPa	0.1KPa
Sensitivity	21.81	420

3.3 Overview of laboratories used

All the tests were carried out in the Geotechnical Department of NTNU. Triaxial creep test was carried out in the laboratory located in the basement but the oedometer test, standard triaxial test and index properties test was carried out in the laboratory located on the first floor of Civil and Transportation Department. The laboratory in the basement was temperature controlled. The temperature of the room is maintained between 10^{0}C to 12^{0}C. It was also shield from noise and traffic to prevent influence of the external environment but the laboratory in the first floor was at the temperature of about 23^{0}C and influenced by the noise and traffic. It was done in order to simulate the insitu condition because the change in the temperature affects the preconsolidation stress and other deformation characteristics of the soil. Moreover, in the triaxial apparatus in the laboratory on the first floor, the load could not be held constant. Hence it was not possible to perform the creep test.

3.4 Test Parameters:

Unit weight: The diameter of the sample was 54mm and the height of the sample was 50mm. The weight of the soil sample was measured in weighing machine and unit weight of soil was calculated as follow

$$\text{Unit weight}(\gamma_w) = \frac{weight\ of\ the\ sample(w_s)}{Volume\ of\ the\ sample\ (V_s)} = \frac{183.92gm}{114.511cm^3} = 1.606\ {}^{gm}/_{cc} \tag{26}$$

$$\gamma_w = 16.06\,{}^{KN}/_{m^3}$$

The volume of the sample was calculated as

$$Volume\ of\ sample\ (V_s) = \frac{\pi}{4} * D^2 * H = \frac{\pi}{4} * 54^2 * 50 = 114511.05mm^3 \quad (27)$$

Here D is the diameter of the sample

H is the height of the sample.

Preconsolidation stress: Preconsolidation stress of Goteborg and Tiller clay was determined by oedometer test using the CRS method and strain rate of 0.30mm/hr as shown in figure 3.2. Preconsolidation stress of the Goteborg clay and Tiller clay was found to be 85Kpa and 160Kpa respectively.

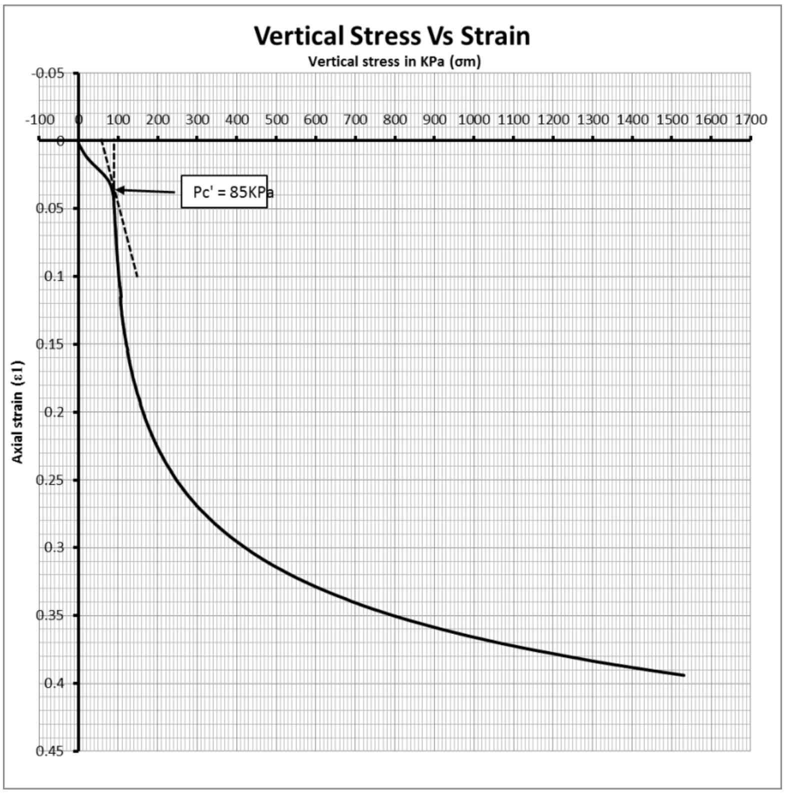


Figure 3.2 Oedometer test in Goteborg Clay (performed by Helene Amundsen)

Soil Strength Parameter: Soil strength parameters, internal frictional angle, undrained shear strength and adhesion/cohesion were determined using the standard undrained triaxial anisotropic compression test. The undrained shear strength was determined to be 31.68Kpa and 42Kpa and internal frictional angle was 37.83^0 and 29^0 for Goteborg clay and tiller clay respectively.

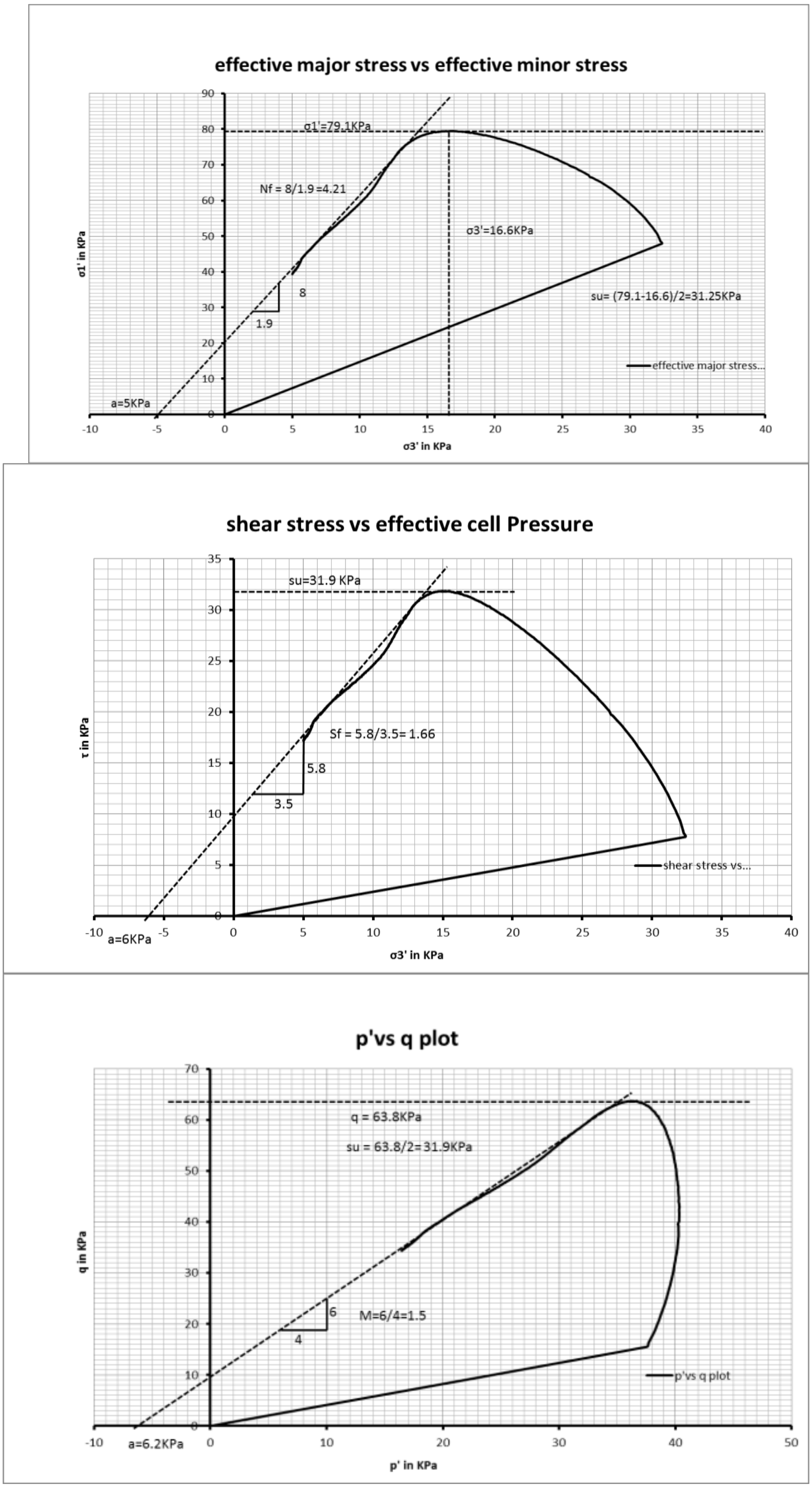


Figure 3.3 Consolidated Anisotropic Undrained Triaxial test in Goteborg clay (performed by Helena Amundsen)

3.5 Test Procedure

Conditions for the test

The aim of the test was to determine the effect of the pore water pressure inside the soil during the test and to study the failure of the soil under creep when the soil reaches certain degree of mobilisation. Hence the test was carried out under the following conditions.

- The pore water inside the soil was allowed to drain only in the lateral direction i.e. axial drainage was not allowed in the test.
- The ends of the soil sample was kept smooth to maintain the vertical stress as the major principle stress i.e. shear stress at the ends of the sample was not allowed. Therefore filters were not used in the test.
- As the filters were not used, back pressure was nearly zero throughout the test.
- Pore pressure sensor was installed on the base to measure the pore water pressure in the middle of the soil sample as shown in figure 3.5.
- The height of the sample was taken only 54 mm. Sample heights can be reduced as axial drainage was not allowed.

Procedure.

Initially, sample of height 50mm and diameter 54mm was trimmed from the block sample. The triaxial apparatus for the test consist of the base with the pore pressure sensor and top cap with the groove around the circumference which was connected to the burette with two holes from the top as shown in figure 3.5. The filters were not use in the test to keep the end of the sample smooth so that the vertical stress can be major principle stress. The test apparatus was as shown in the figure 3.4

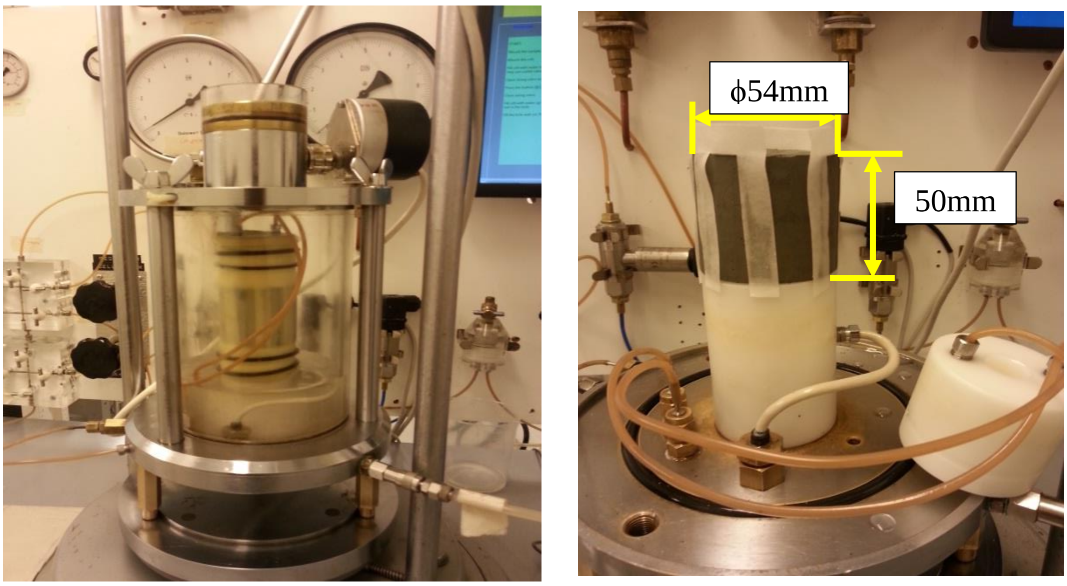


Figure 3.4 Triaxial apparatus and sample for the creep test

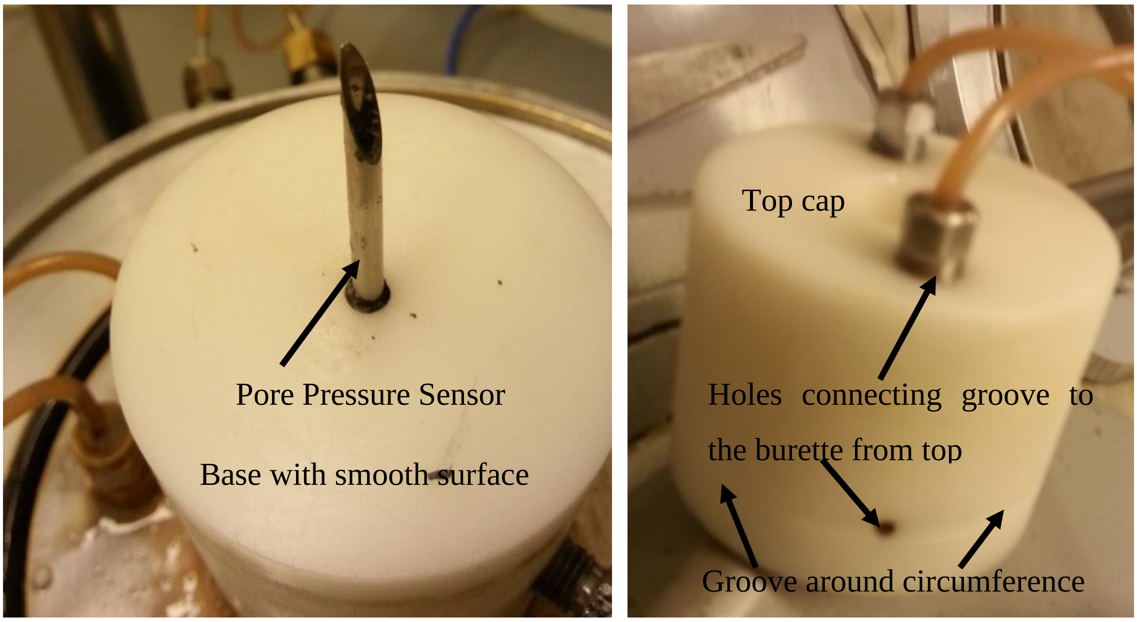


Figure 3.5 Base and the top cap used in the triaxial creep test.

The soil sample was mounted in the apparatus as shown in figure 3.4. Then the cylinder was fully filled with water and the rod was mounted in it. After the apparatus was ready for the test, the consolidation phase was started. The soil was consolidated along the K_0 line and cell pressure and the additional load applied at different loading steps for Goteborg clay were as shown in table 2.

As shown in table 2., the soil from Goteborg was slowly consolidated to the insitu stress (σ_1=48.1KPa; σ_3=26.4KPa) along the K_0 line and it was consolidated to the preconsolidation stress (σ_1=85KPa; σ_3=46.6KPa) and again brought back to the insitu stress (σ_1=48.1KPa; σ_3=26.4KPa). This was done inorder to simulate the stress condition of the soil in the slope. The cell pressure and piston load were increased in a very slow manner with small loading steps inorder to avoid disturbance in the sample or daamge in the structure of the soil sample due to the fluctuation of the pore water pressure inside the soil. Moreover, the K_0 value of the soil was very low ($k_{0,OCR}$ = 0.548), that means significant amount of the shear strength of the soil get mobilised during the consolidation phase (nearly 40% shear strength was mobilized for k_0 =0.548). The clay was also very sensitive. Hence, taking all these factor into consideration, the cell pressure and additional piston load was applied in small loading steps. The cell pressure and piston load were not applied at equal interval of time. At each loading step, the sample was consolidated to at least 90% consolidation or until the volume of the water in the burette remained constant.

In consolidation phase, pore water was drained laterally to the filter papers and collected at the groove in the top cap and then finally to the burette through pipe.

Table 2. Loading details for the consolidation phase of Goteborg clay.

Consolidation Phase with $k_{0,OCR}$ =0.548				
S.N	Cell pressure, $\sigma_{3(KPa)}$	Total axial stress, $\sigma_{1(KPa)}$	Additional axial stress, $\Delta\sigma_{1(KPa)}$	Remarks
1	8.2	15	6.8	
2	11.0	20	9.0	
3	13.7	25	11.3	
4	16.4	30	13.6	
5	19.2	35	15.8	
6	21.9	40	18.1	
7	24.7	45	20.3	
8	**26.4**	**48.1**	**21.7**	**Insitu stress**
9	27.4	50	22.6	
10	30.1	55	24.9	
11	32.9	60	27.1	
12	35.6	65	29.4	
13	38.4	70	31.6	
14	41.1	75	33.9	
15	43.8	80	36.2	
16	**46.6**	**85**	**38.4**	**Preconsolidation stress**
17	43.8	80	36.2	
18	41.1	75	33.9	
19	38.4	70	31.6	
20	35.6	65	29.4	
21	32.9	60	27.1	
22	30.1	55	24.9	
23	27.4	50	22.6	
24	**26.4**	**48.1**	**21.7**	**Insitu stress**

The test for tiller clay was also carried out in similar way as done for Goteborg clay. But, for Tiller clay, pore water pressure in middle of sample was considered more than volume of water in burette. Due to this, the consolidation time for each loading steps was higher for Tiller clay. The K_0 value for tiller were 0.618 and 0.725. Details of the loading steps for Tiller clay and sample calculations for tiller clay and goteborg clay are attached in the appendix.

After the soil sample was brought back to the insitu condition, shearing phase was started. The shearing phase was undrained since the volume of the soil should remain constant i.e. volumetric strain was zero and only shear strain was allowed inorder to study shear creep. The soil was initially sheared to the certain degree of mobilization at the continuous rate of strain. The rate of strain was different for each tests to study the effects of the strain rate in strength and pore pressure condition in the soil. The soil in the slope is at very high degree of mobilization, so the degree of mobilization was choosen f=0.9 for the first test after discussion with supervisor. After the soil reached the mobilization of f=0.9, the load was hold constant. At f=0.9 degree of mobilzation, total axial stress was 83.4KPa and addition piston load was 57.0KPa. For other tests, samples were sheared at different degrees of mobilization. Now, at this state, shear creep of the soil was started. During the creep phase, the soil might or might not fail. If the rate of axial strain is increased, then the sample will obviously fail but until axial strain rate is decreasing the sample will not fail and may not fail at all. In this first test, since the degree of mobilization was high,the sample failed. As the sample failed, piston load also decreased because the motor could not hold the load constant due to decrease in strength of soil. Figure 3.6 shows the outline for carrying out the triaxial test in p'-q space. It shows that the sample is consolidated along k0 line up to point A, which was preconsolidation stress of clay and then it was reconsolidated back to insitu stress, point B. Then, shearing phase starts and BC represents the loading path. The shearing phase was undrained. Point C represents the point of 90% shear mobilization. At point C, load was hold constant for the creep test.

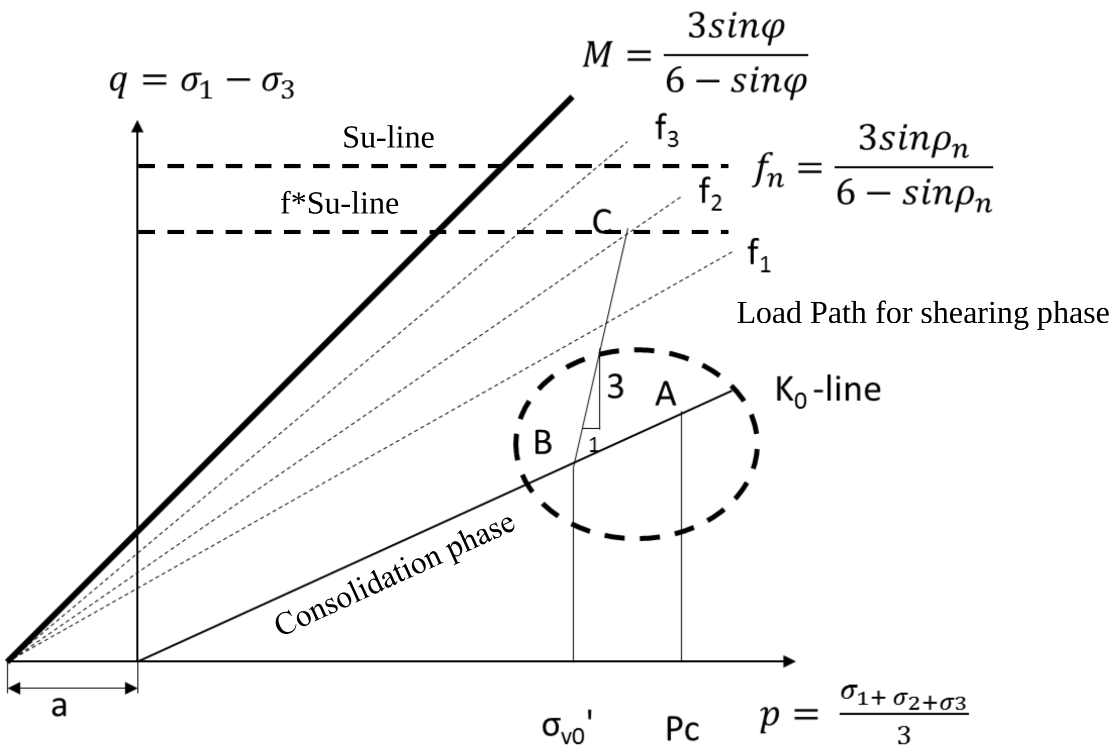


Figure 3.6 shows the procedure for the triaxial creep test

Procedure for the pore pressure check

Total pore pressure at any point inside the soil sample should be same. Hence, the sample was consolidated at a very small cell pressure of 15KPa. The low value of cell pressure was selected not to damage or disturb the sample. Then the valve to the burette was closed to make it undrained i:e undrained consolidation. Then the pore pressure at the surface of the sample and middle of the sample was recorded. Theoritically the pressure measured at the surface and middle of the sample should be equal.

The response of pore pressure sensor at middle was initially checked without mounting sample on it. The cylinder was just filled with water and undrained condition was maintained. The increase or decrease in cell pressure should be equal to the response of pore pressure sensor. It helped to find the presence of air bubbles inside the pore pressure sensor too.

Chapter 4

4.0 Results

4.1 Goteborg clay

Two tests were done on clay sample from Goteborg. Test 1 was quick test with the strain rate of 3.25mm/hr and Test 2 is slow test with the strain rate of 0.15mm/hr. In both tests, samples had been consolidated to the same stress level along the same K0 line. The results of both tests are presented below

Consolidation phase

Depth = 6.35m

Ground water table = at 1m depth from GL

Bulk density = 16KN/m^3

Frictional angle (φ) = 37.83^0

Adhesion (a) = 5.7KPa

Poisson ratio (ν) = 0.15

OCR = 1.8

$K_{0, OCR}$ = 0.548

Undrained shear strength (su) = 31.68KPa

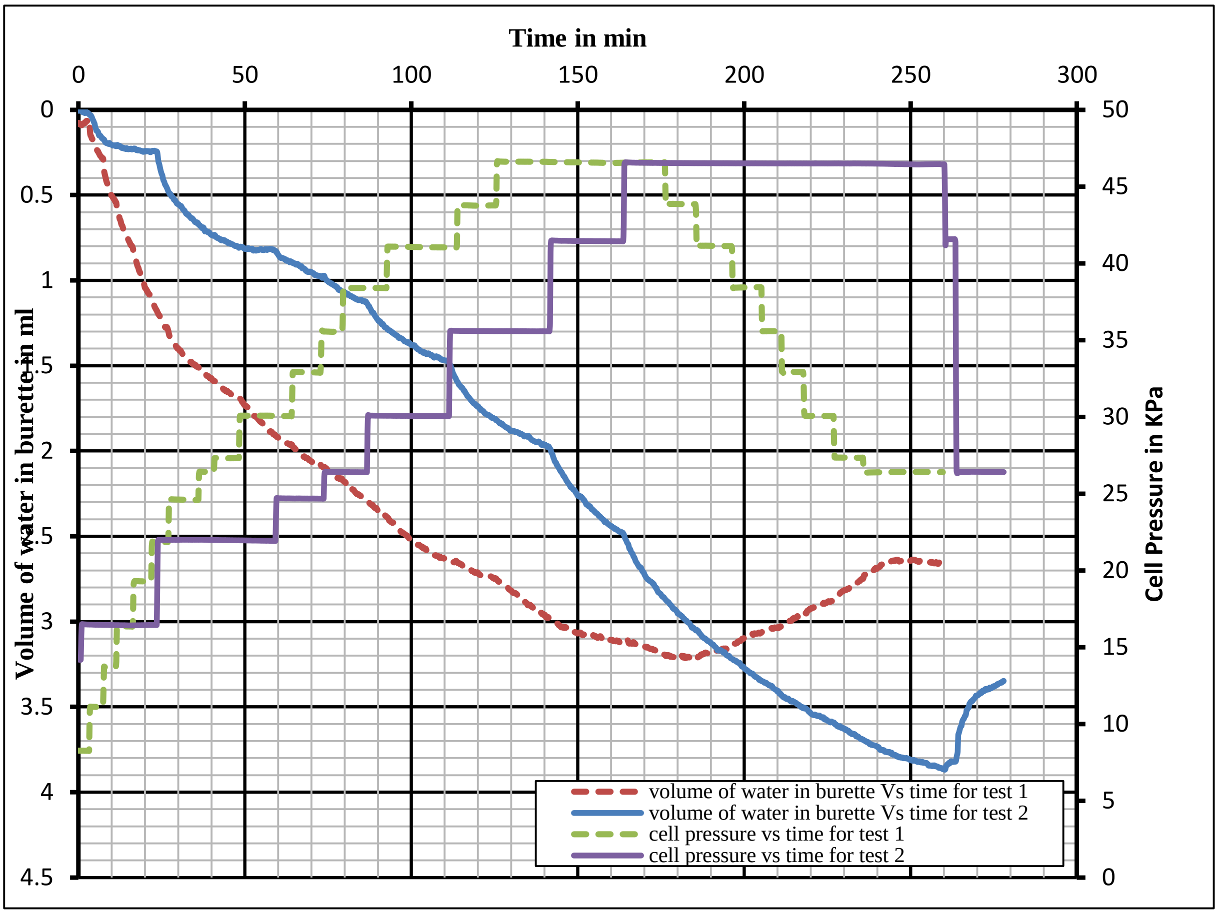


Figure 4.1 shows the volume of water consolidated from the sample during consolidation along K0 line

Figure 4.1 shows that different amount of water is consolidated out from the sample in test 1 and test 2 during consolidation phase. In test 2, about 0.7 ml of more water is consolidated out from sample than in test 1 due to more consolidation time in test 2. When the sample is consolidated back from the preconsolidation state to the insitu state, the sample sucked the water inside. In both test, the amount of water sucked inside by the sample is same although the sample is brought to insitu state rapidly within 2 steps in test 2 and slowly with the number of small steps in test 1.

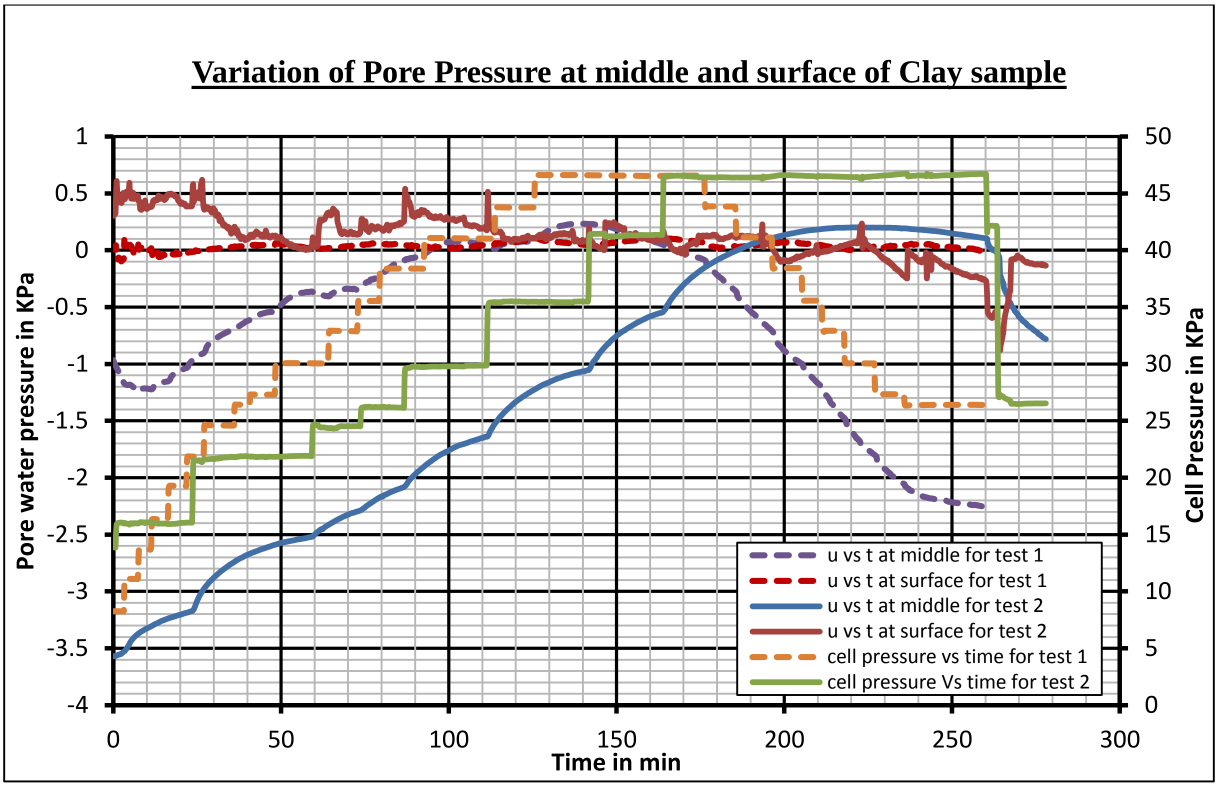


Figure 4.2 Variation of pore water pressure at the middle and surface of clay sample for test 1 and 2

Figure 4.2 shows that pore water pressure at the surface of the clay sample during consolidation. During consolidation, the surface of the sample is connected to the drainage system, so the pore water pressure should be zero at surface but the graph shows the small values which is the calibration error. The fluctuation in the nature of the curve is due to the fluctuation of the temperature during the test.

Figure 4.2 shows that there is the pore water pressure in the middle of the sample which increases with increase in the cell pressure. The pore pressure at the middle tries to be in equilibrium condition with the pore pressure at the surface, so the pressure at middle increases gradually from negative value to the pore pressure value at surface. It shows the difference of about 2.5KPa pore pressure at middle of sample in betweeen test 1 and test 2. The difference is because of the calibration error and the presence of the air in the needle of pore pressure sensor. There is smaller increment of the cell pressure in each step for test 1 than in test 2. Also test 1 starts at the cell pressure of 8KPa but test 2 starts at 14KPa. Consolidation time at each steps is higher in test 2 compared to test 1. In both the test, the rate of change in pore pressure at the middle of the sample is very slow compared to the cell pressure increment and also the pore pressure values are very low, almost negligible. This slow pore pressure response is due to the presence of air in the sensor needle which was later removed by dessication and further tests were done in tiller clay.

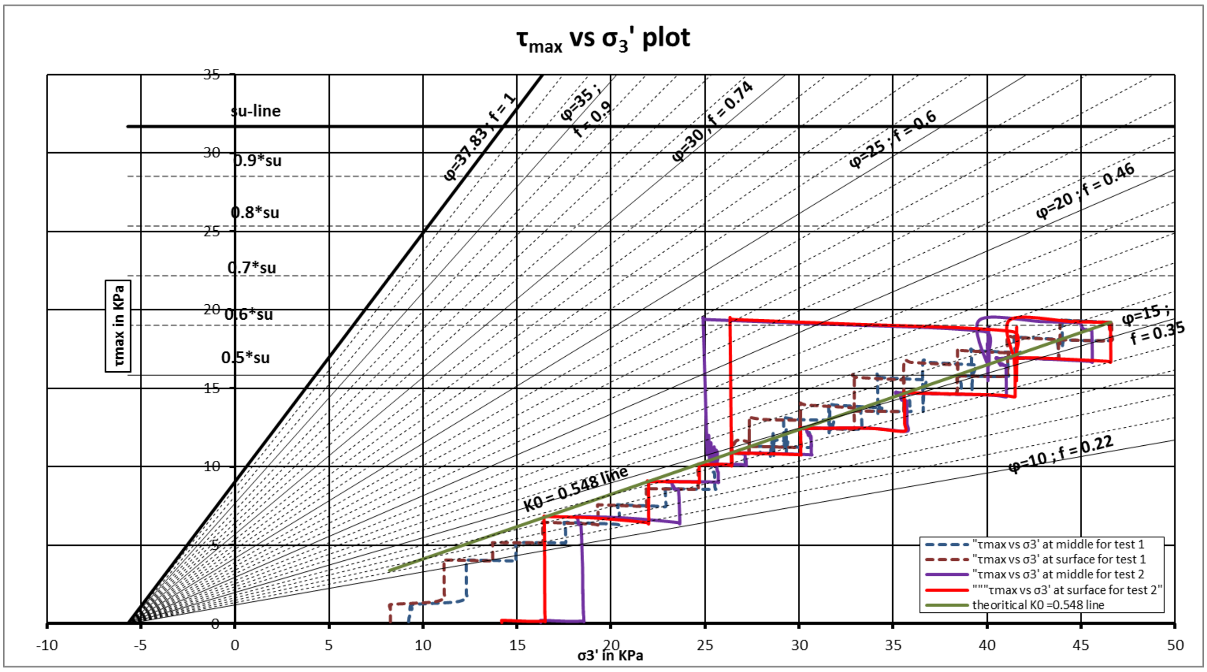


Figure 4.3 Effective stress path for test 1 and test 2 during consolidation phase

Figure 4.3 shows the effective stress path when the clay sample is consolidated along the K0= 0.548 line up to the preconsolidation stress level and back to the insitu stress level. There is no large variation in the pore pressure at the middle and surface of the sample during consolidation, so the effective stress path is along the theoritical K_0 line with small deviations of about $\pm$ 2.5 KPa. During reconsolidation of the sample back to insitu stress, it should be done in steps with small reduction in the stress level. Otherwise, the shear stress will be highly mobilized approaching near the failure line as shown for test 2 in which the shear strength is mobilized by 55% instead of 35%, i.e. 20% more shear strength is mobilized. Hence while doing anisotropic consolidation, the pressure should be changed slowly in small steps to control the higher degree of shear mobilization.

Shearing phase

Drainage condition = Undrained	Degree of shear mobilization (f) = 0.9
Depth = 6.35m	Poisson ratio (ν) = 0.15
Ground water table = at 1m depth from GL	OCR = 1.8
Bulk density = 16KN/m^3	$K_{0,\,OCR}$ = 0.548
Frictional angle (φ) = 37.83^0	Undrained shear strength (su) = 31.68KPa
Adhesion (a) = 5.7KPa	

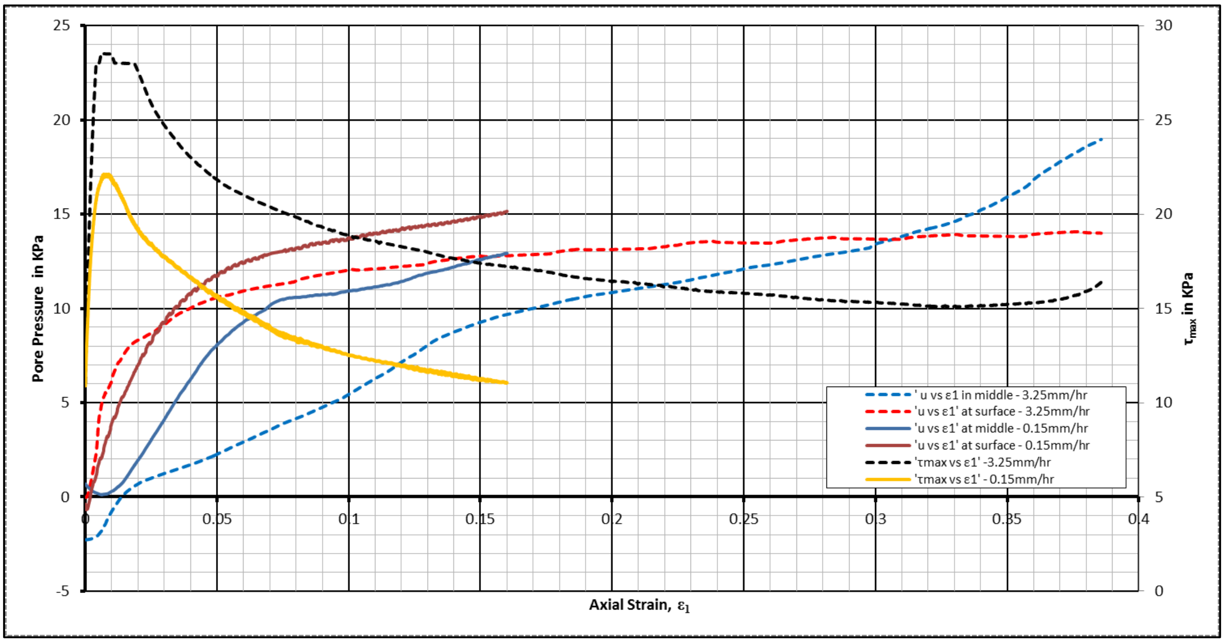


Figure 4.4 Pore pressure variation during shearing phase for test 1 and test 2

Here test 1 is done at strain rate of 3.25mm/hr and test 2 at strain rate of 0.15mm/hr. Figure 4.4 shows that the variation in the pore pressure at the surface and middle of sample for test 1 is very high which is 9.5KPa at 5 % strain and 6.5KPa at 8% strain. In test 2, the pore pressure variation at middle and surface of the sample is 4KPa at 5% strain and 2.5KPa at 8% strain. In test 1, the pore pressure variation has been increased after failure and again decreased when soil reached a residual strength. In test 2, pore pressure variation is almost uniform after failure.

Figure 4.4 also shows stress-strain curve for the clay sample. It shows that shear strength for quick test is higher than for slow test. In the quick test the shear strength become constant when it reachs around 28KPa and then fails but in slow test the peak shear strength is 22KPa. When the strain rate is decreased from 3.25mm/hr to 0.15mm/hr i.e. 95% reduction in the strain rate, the shear strength reduced from 31.68KPa to 22KPa i.e. 30.5% reduction in shear strength.

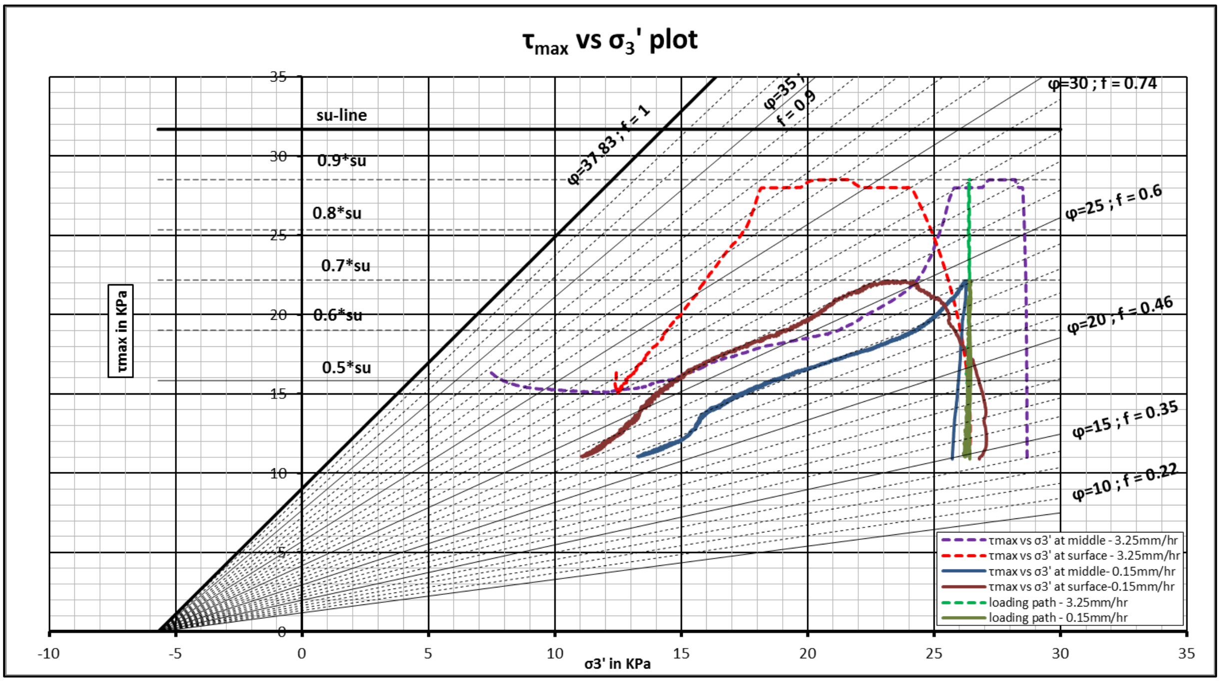


Figure 4.5 Effective stress path for CAUC of clay sample for test 1 and test 2

Figure 4.5 shows that the failure surface obtained from the standard triaxial test and from the creep test does not coincide with each other. The strength parameters obtained from the standard triaxial test is very high i.e. 37.83^0 frictional angle but in shear creep test it is 32^0. In test 1, the soil is mobilized to 90% shear mobilization and then load is kept constant. But the pore water pressure in the soil increases the shear mobilization and drives the soil towards failure. In test 2, the soil cannot reach the 90% shear mobilization state because of the decrease in the strength caused by low strain rate and fails earlier.

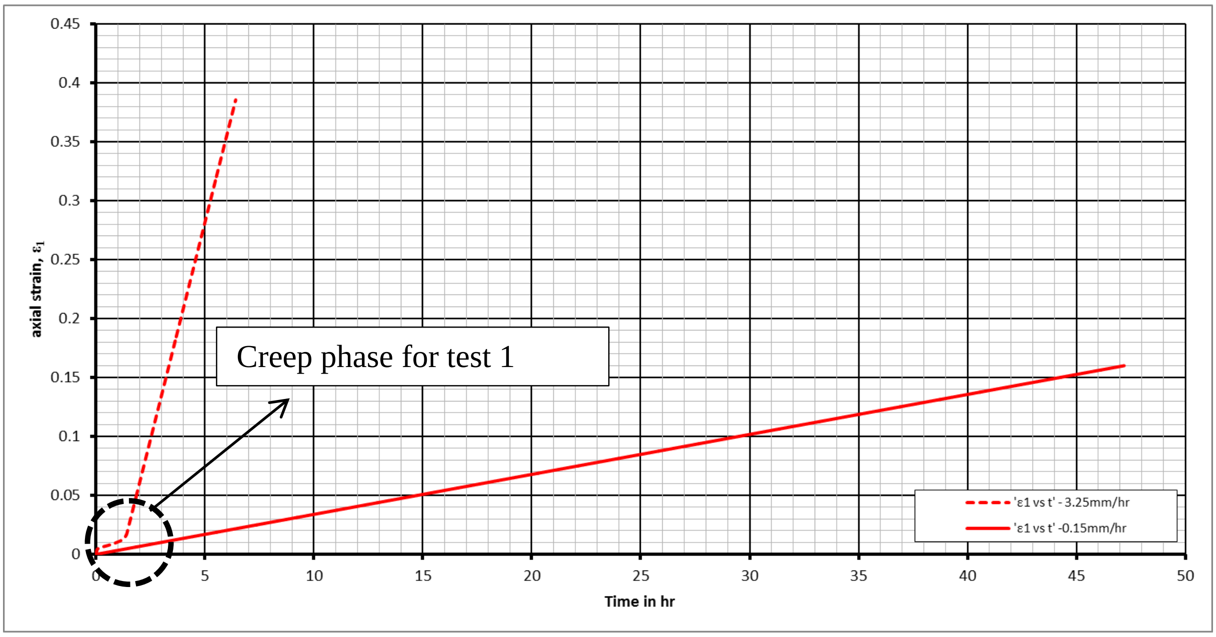


Figure 4.6 Axial strain – time curve for Goteborg soil

Creep phase exist in test 1 only so strain rate during the creep for test 1 is magnified in figure 4.7. Strain rate is constant for test 2 as it fails before creeping.

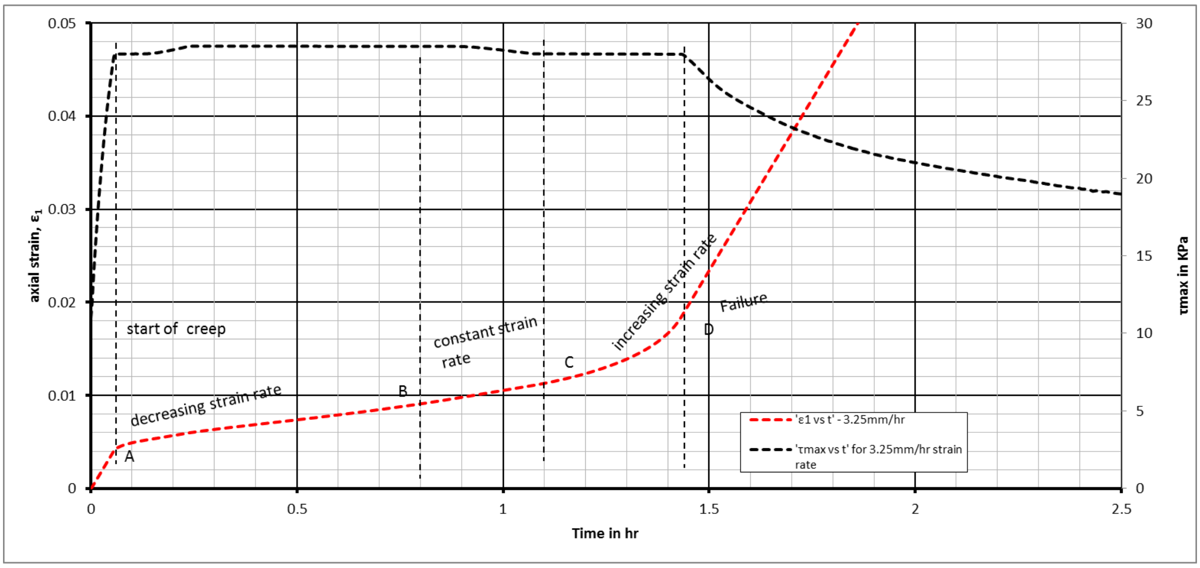


Figure 4.7 Axial strain-time curve for test 1 of Goteborg Sample

In test 1, the sample is sheared at constant strain rate of 3.25mm/hr as shown by line OA and the creep starts at point A, from where the strain rate is decreased initially to B. Then strain rate becomes constant along BC. After point C, the strain rate starts increases indicating that the sample will fail after certain time. Finally at point D it fails and strain rate increase rapidly, as shown in figure 4.7.

4.2 Tiller clay

Two tests were done in tiller clay. Test 1 is done at strain rate of 0.4mm/hr and test 2 is done at strain rate of 0.8mm/hr. In test 1, the sample is consolidated along the K0 = 0.62 line but in test 2, the sample is consolidated along K0 = 0.725 line. The different K0 line for two tests is due to change in the Poisson's ratio which is 0.3 and 0.15 respectively. The results of both the tests are presented below.

Consolidation phase

Depth = 11.4m

Ground water table = at 0.5m depth from GL

Bulk density = 18.25KN/m^3

Frictional angle (φ) = 29^0

Adhesion (a) = 10.824KPa

Poisson ratio (v) = 0.30 and 0.15

OCR = 1.6

$K_{0, OCR}$ = 0.618 and 0.725 respectively

Undrained shear strength (su) = 42KPa

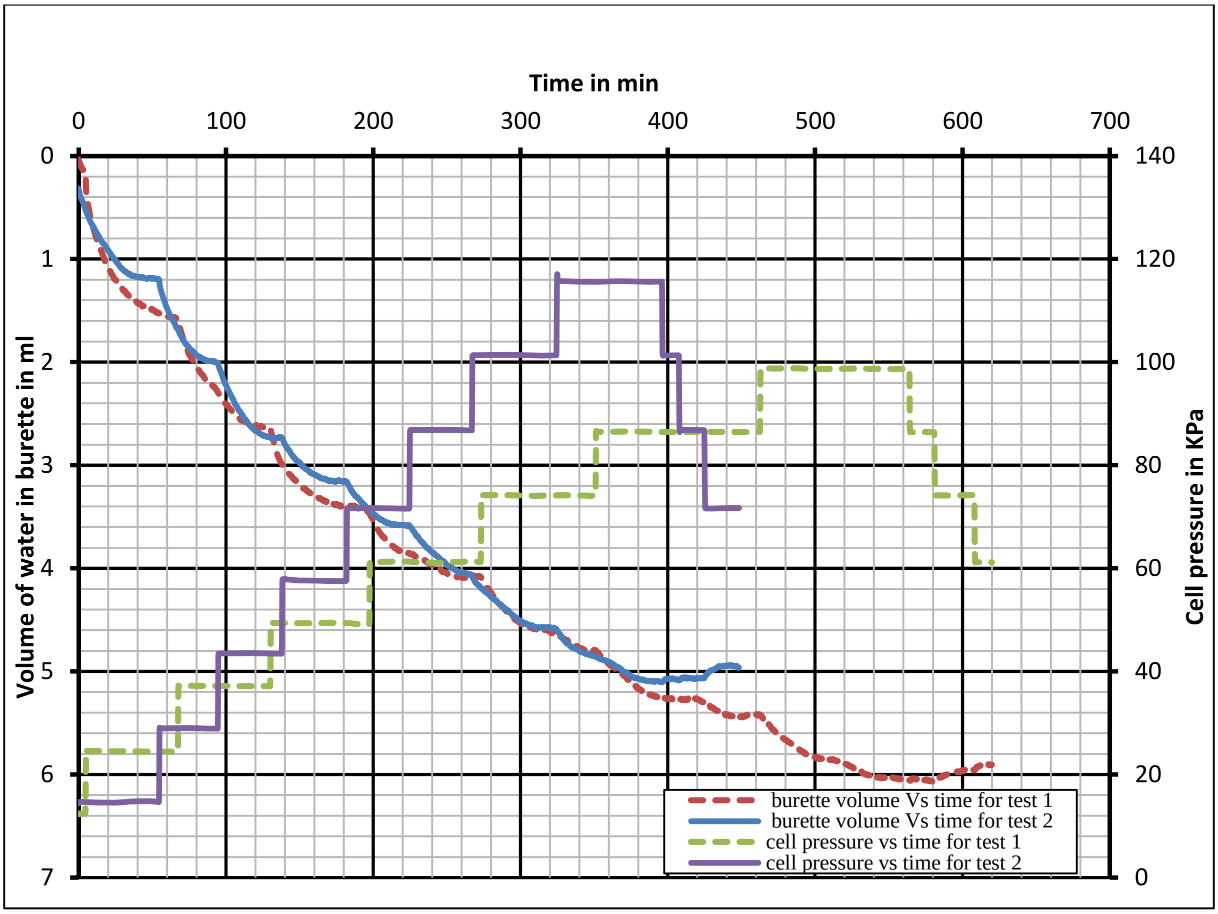


Figure 4.8 volume of water in burette- time curve for tiller clay

As shown in figure 4.8, the volume of water consolidated in test 1 is higher than in test 2 by about 1ml. The consolidation time required for test 1 is higher than for test 2. Although the sample is consolidated to the same stress level but the path of consolidation is different. It may be the reason for the different consolidation time and consolidated volume. The total stress change at each step is constant for both test.

The pore pressure variation inside and at surface of the sample during the consolidation is shown in figure 4.9. The curve shows that the pore pressure sensor gives quick response to the change in pressure. When the stress level is increased at each step, the pore pressure inside the sample increases abruptly to the maximum value and then decreases slowly. The decrease in the pore pressure seems to be asymptodic curve which shows that the pore pressure inside the sample is never equal to zero and there is always small amount of pore water pressure which is trapped inside the soil. the maximum amount of pore pressure obtained in both test is same around 25KPa. It shows that at certain stress level, the peak pore pressure inside the soil is maximum and it decreases in the other stress level. In both the test, the maximum peak pore pressure is obtained when the sample is subjected to total vertical stress of 40 KPa. Although

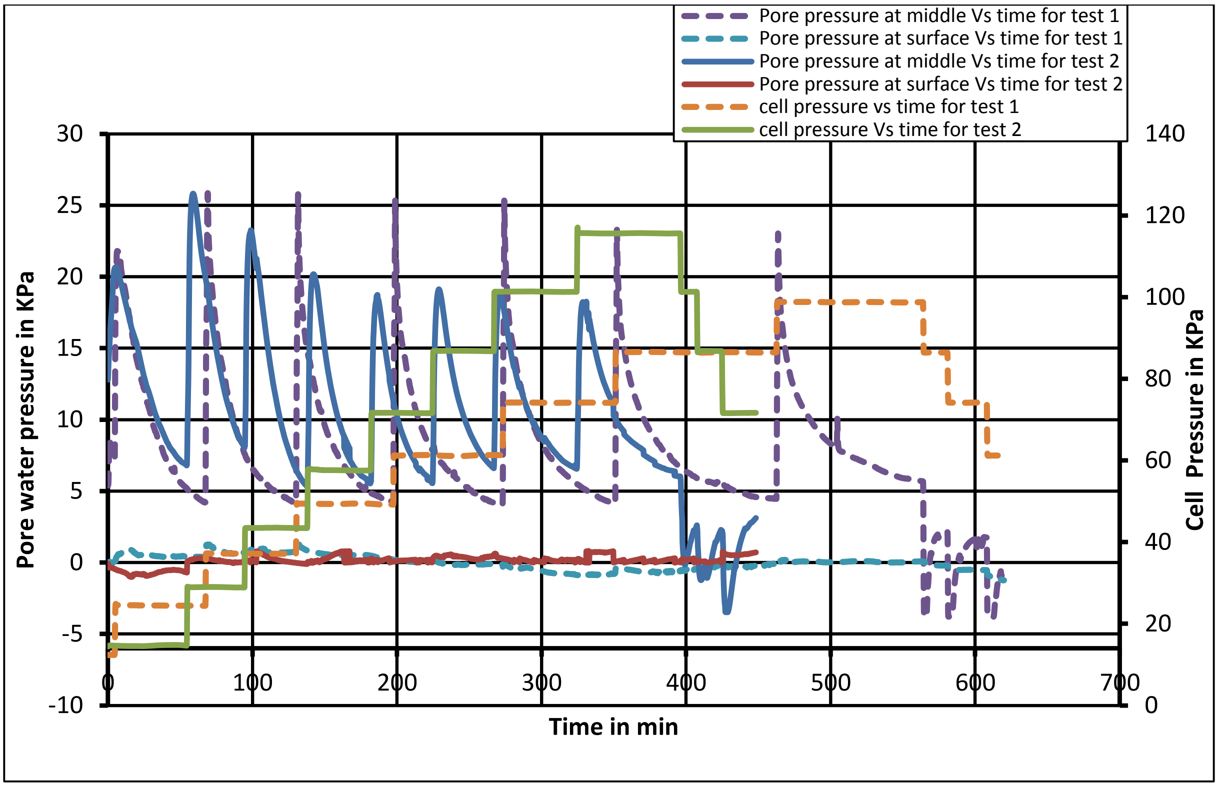


Figure 4.9 Pore Pressure variation – time curve for tiller clay

the nature of pore pressure variation is same for both test, test 2 shows comparatively small values. This may be because the sensor has little slow response due to the clay stucked in the sensor needle while changing the sample for doing test 2. It may be due to the different consolidation path. When the sample is consolidated from preconsolidation stress to the insitu stress, the pore water pressure rapidly decrease to the negative value and the sample sucks water inside from burette. It shows that for 10KPa decrease in cell pressure or 20KPa decrease in total stress, the drop in pore pressure is also nearly10KPa for test 1 but further 10KPa decrease in cell pressure does not give same reduction in pore pressure, rather it decreases to the same pore pressure level i.e. -4 KPa, for every consecutive decrement thereafter. It is because the minimum pore pressure the sensor can measure is -4KPa. The pore pressure at the surface during the consolidation is almost zero in both test.

Figure 4.10 shows the effective stress path during the consolidation of the sample. The theoritical K_0 line along which the sample is to be consolidated is also shown in figure 4.10 for test 1 and test 2. The consolidation path that is obtained from the test is also shown, when the pore pressure at the middle of the sample is considered. In test 1 the sample is loaded along the K0=0.618 line where the sample will be mobilised to 38% shear mobilization but actually the soil is mobilised to 66% shear mobilisation. It is about 73% higher than expected.

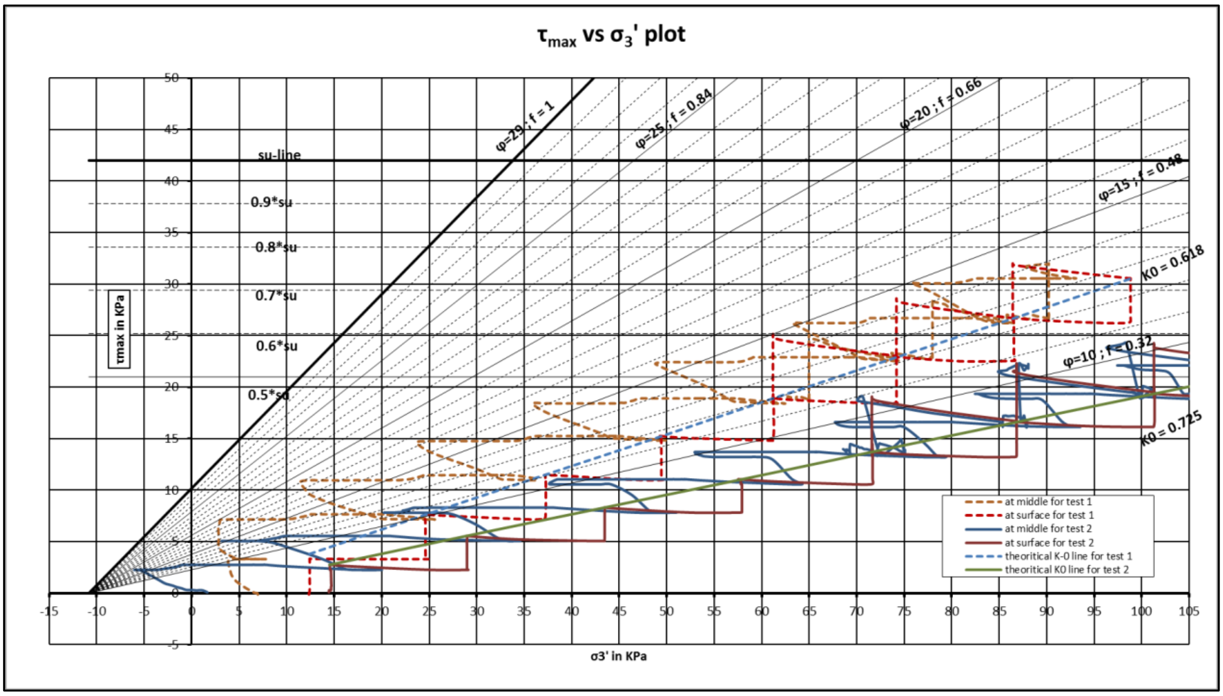


Figure 4.10 Effective stress path for the consolidation phase of tiller clay

In test 2, the soil is mobilised to about 84% shear mobilisation, which is almost near to failure, but the expected shear mobilisationis only 32%. It seems that soil is subjected to high mobilization at the low stress level than high stress level. As shown in figure 4.10. in both test maximum degree of shear mobilization i.e. 66% and 86% repectively, is obtained at the starting of consolidation phase.

Shearing phase

In the shearing phase, the sample was sheared at 0.4mm/hr strain rate for test 1 and 0.8mm/hr strain rate for test 2. The shear strength of the soil was fully mobilised for test 1 but for test 2 only 80% of the shear strength was mobilised, then the load was constant and sample was allowed to creep.

Drainage condition = undrained test	Mobilization degree (f) = 1.0 and 0.8
Depth = 11.4m	Poisson ratio (ν) = 0.30 and 0.15
Ground water table = at 0.5m depth from GL	OCR = 1.6
Bulk density = 18.25KN/m^3	$K_{0, OCR}$ = 0.618 and 0.725 respectively
Frictional angle (φ) = 29^0	Undrained shear strength (su) = 42KPa
Adhesion (a) = 10.824KPa	

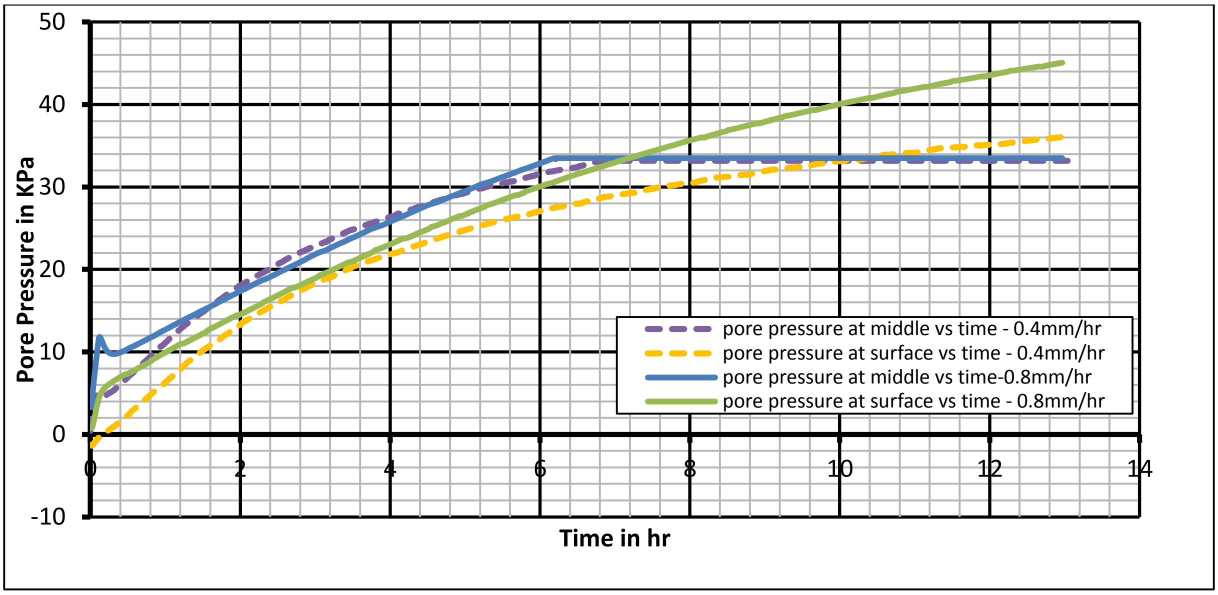


Figure 4.11 Pore Pressure variation – time for shearing phase of tiller clay

Figure 4.11 shows the pore pressure variation during the shearing phase for test 1 and test 2. It shows that the pore pressure variation in the middle and surface of the sample during undrained shearing is almost same which is about 5KPa, independent of the rate of shearing. The pore pressure difference for both the test is also small which is nearly 3KPa. It shows that for the overconsolidated clay, the change in the strain rate does not cause much variation in the pore pressure. In both tests the maximum pore pressure at the middle of the sample is 34 KPa. It is the maximum pressure that the sensor can measure. Hence the pore pressure at middle is supposed to be higher than the value shown in figure 4.11. The increase in pore pressure is higher initially at the middle of sample but as the peak shear strength is reached, the rate of change in pore pressure is decreased, as shown in figure 4.12.The pore pressure variation occured in very small strain for test 2 as the sample has not failed yet.

Figure 4.13 shows effective stress path for the tiller clay for test 1 and test 2. The soil is not failed when the shear strength is mobilised to 80% only. But the pore water pressure drags the soil to failure under the constant load also. The pore water pressure at the middle of the soil is higher which shows that the soil at the middle cannot take higher effective stress compared to soil at periphery. From this aspect, the soil at the middle fails early than at pheriphery. It also shows that when the pore water pressure at the surface is considered then soil fails at frictional angle of 26^0 and when pore pressure at middle is considered, failure occurs at 29^0. Figure shows that for test 1, there is vertical decrease in shear strength at certain effective stress at middle of sample. It is due to the threshold of capacity of the pore pressure sensor. However the difference in the effective stress capacity at the centre and periphery of the soil is very small about 5 KPa. The test 2 results shows that the pore pressure inside the soil leads to the failure even at the constant stress at low degree of shear mobilization.

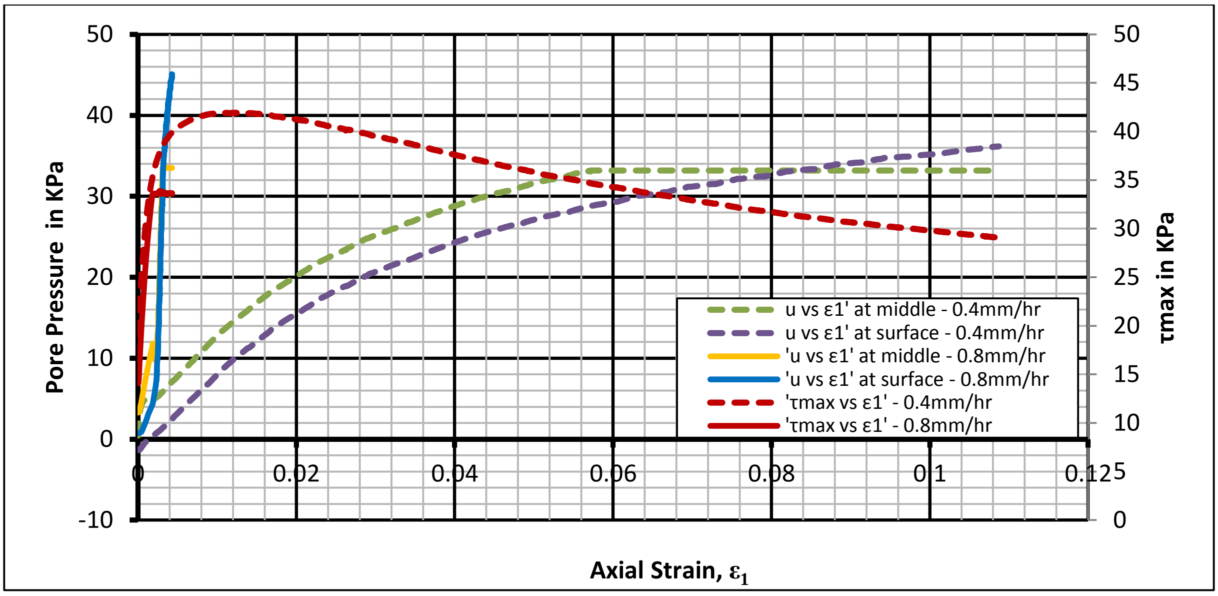


Figure 4.12. Pore pressure variation with strain for tiller clay

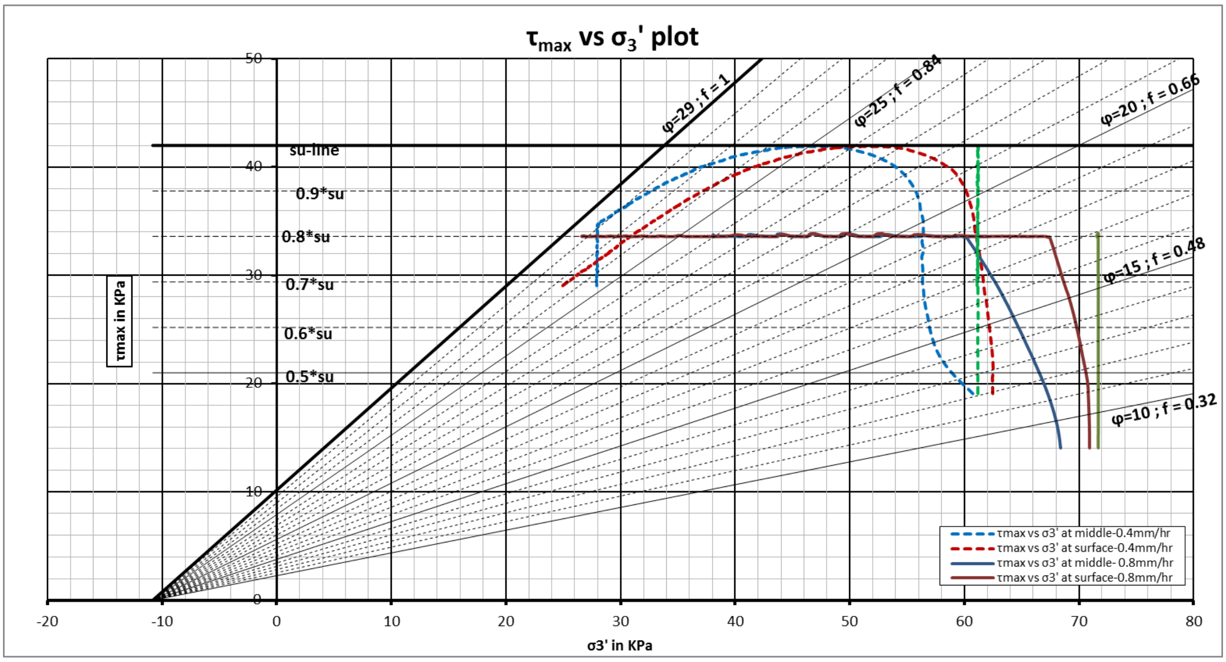


Figure 4.13 Effective stress path for the shearing phase of tiller clay.

Figure 4.14 shows the axial strain in the sample during test 1 and test 2 with time. For test 1, the axial strain increases linearly at the rate of 0.4mm/hr and reaches the maximum strain but the value shown in figure 4.14 is the cut off value. The total axial strain occurred in the sample during the test is shown in figure 4.15. For test 2, the axial strain increases linearly at the rate of 0.8mm/hr till it reaches 80% of the maximum shear strength of the soil, then starts creeping at constant load. During the creep phase, axial strain increases at the decreasing rate initially and then, increase in the axial strain at increasing rate was not observed as the sample has not been failed yet as shown in figure 4.16.

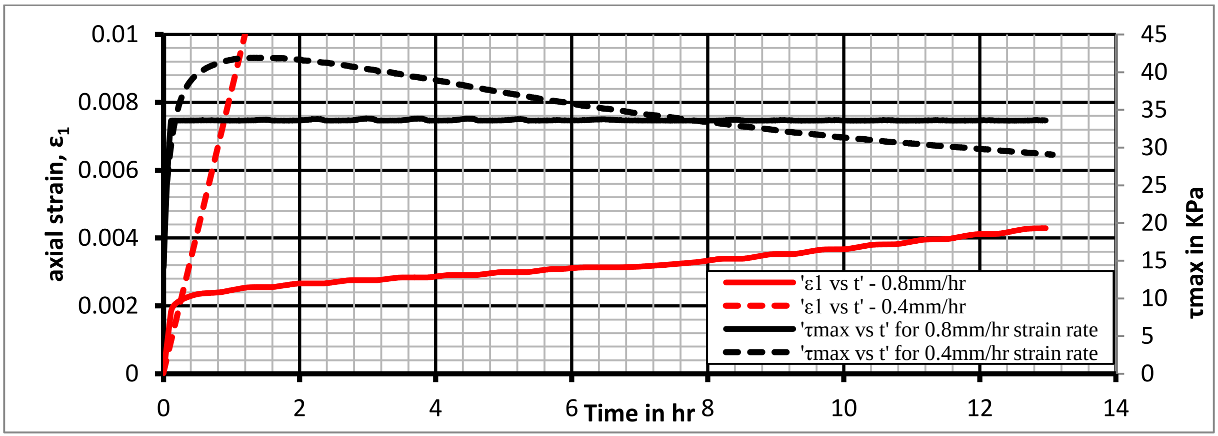


Figure 4.14. Axial strain-time curve for tiller clay.

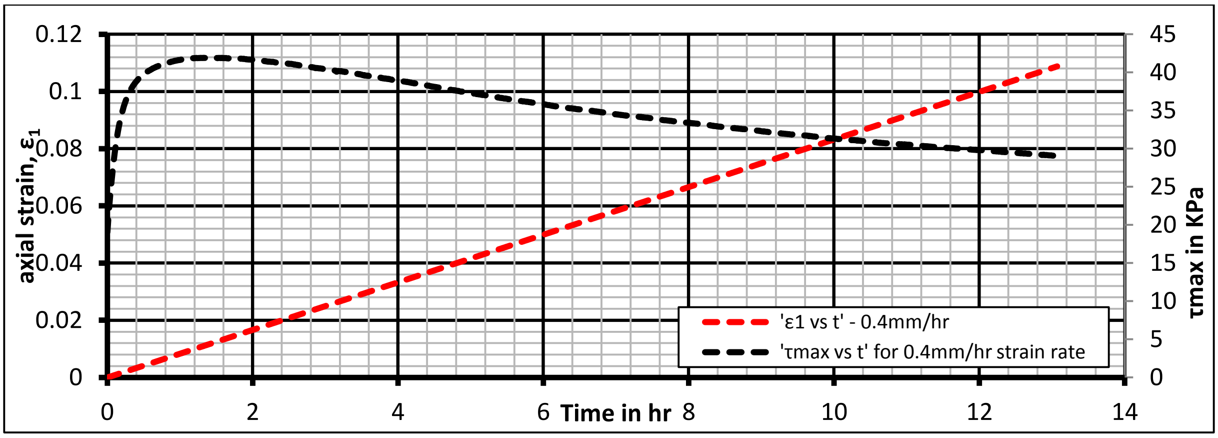


Figure 4.15 Axial strain vs time plot for test 1 of tiller clay

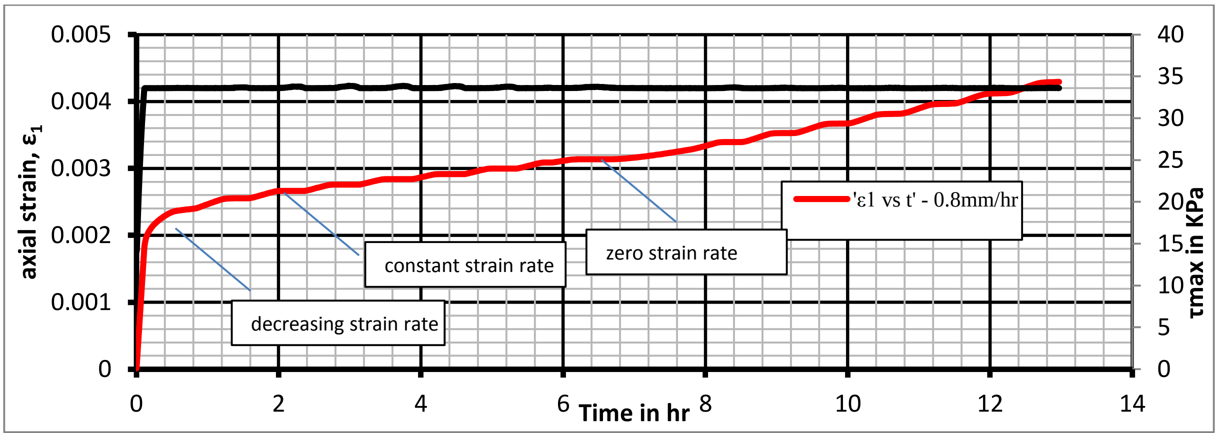


Figure 4.16 Axial strain vs time plot for test 2 of tiller clay

During the creep phase in figure 4.16, strain rate decreases initially at the start of the creep, then it becomes constant and becomes zero but again the axial strain increase at constant rate of 0.275mm/hr and becomes constant and cycle repeats again as shown in figure 4.16. The increase in strain rate was not observed during the creep so the test was stopped manaully and the maximum axial strain observed is 0.43 %, nearly 0.2mm deformation as shown in figure 4.17. The shear strength is 33.6KPa.which is above the residual strength or post peak strength of the soil i.e. 29KPa as shown in figure 4.12.

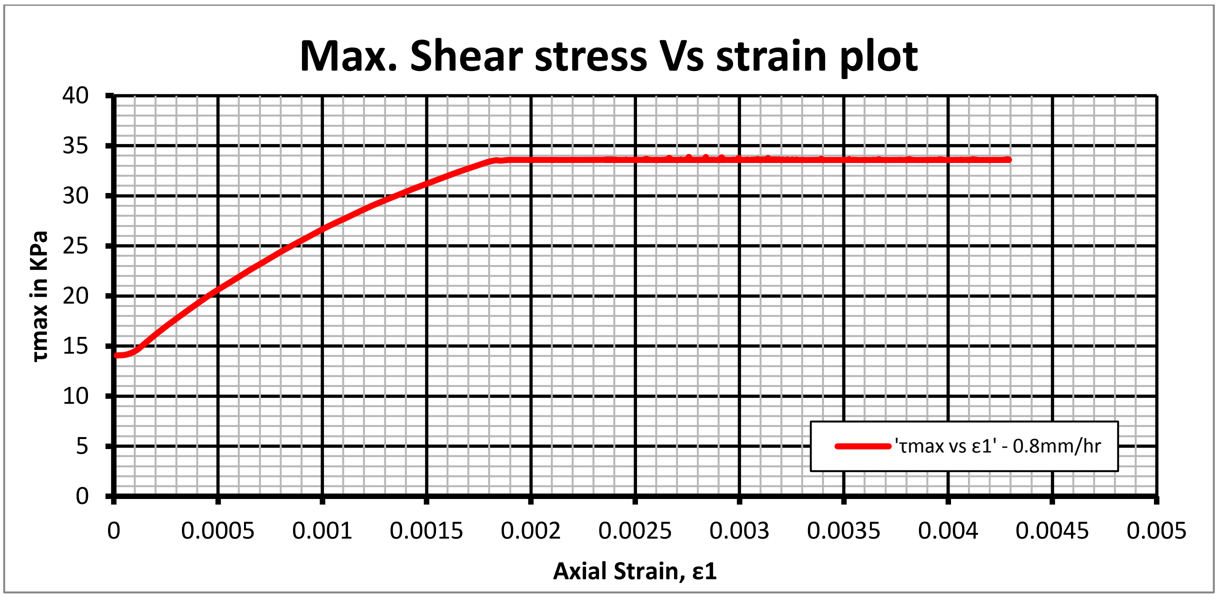


Figure 4.17 stress strain plot for test 2 for tiller clay

Chapter 5

5.0 Conclusion and Discussion

5.1 Sample quality

The clay sample from the Goteborg contained the organic shells inside the soil. It has also the voids from small to large size. One of the sample contained large void at the middle of the sample of height 80mm as shown in figure 3.1 and was useless for the test. The voids were filled with air which may have affected the sensor to response quickly to the pore pressure change. Back pressure was not used during the test, so the presence of the air voids inside the sample has affected the test result for Goteborg sample.

The clay samples from the tiller were the piston samples. The soil was quick clay and it was fully saturated. There were not air voids and any organic matter inside the sample that affects the test result.

5.2 Test apparatus

When the drainage in the sample in triaxial test is restricted, then the technique for the test becomes complex. The modification made in the triaxial apparatus created little difficulty to carry out the test. The modification has restricted the use of the back pressure. Due to the lack of back pressure, it was impossible to trap out the air present inside the sample. So the test apparatus cannot be used for the unsturated soils.

The pore pressure sensor inserted in the sample to measure a pore pressure in middle of the sample was very sensitive to the pressure change. If the air gets trapped inside a needle with sensor then it shows delayed respose to the pore pressure change. Many trial tests were done to check the response of the pore pressure sensor. When the test was completed and the sample was removed from the apparatus, the clay gott stucked at the top of the needle and it seems to delay the respose of sensor. If the stucked clay was removed before starting another test then air might be trapped inside the sensor when mounting next sample. So, before starting each test, it would be better to check the presence of air inside the sensor or new sand filters can be filled inside the sensor and then remove the air inside the needle keeping it in a dessicating machine.

5.3 Test parameters

Preconsolidation stress of Tiller clay and Goteborg clay was obtained by doing oedometer test by using CRS method. Strength paramters like undrained shear strength, cohesion and frictional angle of the soil was determined from standard triaxial test. Only one standard triaxial test was done which is not sufficient to confirm the parameters obtained. It may be the reason for the difference in the failure line obtained from the standard triaxial test and shear creep test for the Goteborg clay. The poisson's ratio for the soil was determined in discussion with the supervisor and the K_0 value for the clay was determined by using eqn 10 and 11.

5.4 Test results

The results obtained from the Goteborg is supposed to contain errors. The pore pressure sensor reponsed slowly to a pore pressure variation due to the presence of air in the sensor needle. The results are also not free from the calibration error. Due to these reasons, it became difficult to make the quantitative analysis of the results obtained from the Goteborg sample. The results obtained from Tiller clay is good to make interpretation. The pore pressure sensor also showed quick and good response to the pressure change. Both results obtained from the tilller clay shows logical varaitions.

The test result shows that there is the variation in the pore pressure between the middle and surface of the sample. During the consolidation phase, although the sample was connected to the drainage system, there was pore pressure inside the soil which causes the failure as shown by figure 4.2, 4.3, 4.9 and 4.10. Clay has impervious structure, so it takes time to dissipate the pore water pressure inside the sample when it is loaded. Due to this, the pore pressure is built up inside the sample during the consolidation also. The magnitude of this pore pressure was very high which was more than 30KPa and cannot be neglected while doing the test. So, while doing anisotropic consolidation in clay soil, after each increasement of cell pressure and additional piston load, adequate time should be provided inorder to dissipate the pore pressure present inside the soil sample as in figure 4.9. It seems that during the aniotropic consolidation test, after each load increments, the consolidation time for each step should be considered based on the time required for the soil to dissipate the inside pore water pressure rather than the time required for the water in burette to be constant. If the time required for the dissipation of the pore water pressure is considered then the consolidation time for the soil will be taken into account itself. But , if the time for the primary consolidation is only considered during the test

and the load is increased based on it, the pore pressure will be built up in each steps as shown by Goteborg sample, and at high stress level, it may fail the sample in consolidation phase.
The stress increasement in each test should be small i.e.sample needs to be consolidated in small loading steps, while doing anisotropic consolidation. If the stress increment is high then the pore pressure inside the sample will be also high. Pore water pressure inside the sample for the saturated clay is directly proportional to the stress increment. If the stress increasement is higher then the pore water pressure inside the sample is also high. Hence, while starting the test, the load steps at the initial steps should be comparatively low. At the initial step, the soil is closer to the failure line and the comparatively small increasement in the pore water pressure can push the sample to failure line or can cause high degree of shear mobilization as shown in figure 4.9. Also taking larger loading steps cause high degree of shear mobilization during the consolidation phase as shown by figure 4.3. Due to this reason, while consolidating Tiller sample, about 10KPa cell pressure was increased at each load step and even this increment tend to cause failure in the sample at initial steps. Hence, the consolidation along the K0 line should be done in small loading steps allowing the pore pressure inside the sample to dissipated to equilibrium.
It shows that there is high risk of higher degree of shear mobilization during the consolidation of the clay sample along K0 line. If higher amount of shear is mobilized during the consolidation phase then there will be less amount of shear strength remained for shearing phase. Also, the pore pressure built up at each load step and dissipation of pore pressure in each load step is supposed to create the cyclic load in the sample. The sample is supposed to be subjected to continuous loading and unloading. This cyclic load causes the decrease in the shear strength of the sample and the sample fails before reaching the failure line in the shearing phase, as seen incase of tiller clay and Goteborg clay, both of them fails before reaching the failure line as shown in figure 4.5 and 4.13. The decrease in the shear strength due to this cyclic load cannot be quantified from few tests only. It requires more number of tests.
In shearing phase, the samples has been sheared at different strain rates. For Goteborg sample, quick test was done at the rate of 3.25mm/hr and slow test is done at the rate of 0.15mm/hr. The strain rate has been reduced by 95% and the reduction in shear strength due to this is 30%. In case of Tiller clay, quick test was done at strain rate 0.8mm/hr and slow test at rate of 0.4mm/hr. Here the reduction is 50% in strain rate but it does not cause significant reduction in shear strength of the clay. This result is comparable to the result obtained by ***(Lefebvre and Leboeuf 1987)***.

In the shearing phase, the pore pressure in the middle of the sample is greater than at the surface and the difference is about 5KPa. Due to the maximum limit of the pore pressure sensor to about 33KPa, the point of equilibrium of the pore pressure inside the sample and at surface of the sample could not be determined. However the sample inside fails earlier than at surface.

When the creep phase starts, the strain rate decreases initially, then becomes constant and starts to increase indicating that the sample will fail as shown in figure 4.7. If the soil is not going to fail under the constant load or creep, then the soil will deform under the decreasing strain rate as obtained for Tiller clay in figure 4.16.

Both for Tiller clay and Goteborg clay, difference in pore water pressure obtained at the middle and periphery of the sample is high for the quick test and low for the slow test. The difference although, less significant, is higher at the low strains than at higher strains as obtained by (***Kimura and Saitoh 1983***) in his test as shown in figure 2.21.

In this experiment, under the consideration of pore pressure at surface, the test shows lower frictional angle than frictional angle considering the pore pressure at the middle. The difference is 3^0. This means if the pore pressure at the surface is being measured during the experiment then it will give less frictional angle which means degree of mobilisation is decreased by 12% i.e. the specimen is not mobilised to full degree during the test, considering the pore pressure at the middle of the specimen is correct. But, if the pore pressure at the surface is considered to be correct value and, if we are measuring pore water pressure at the middle of the sample, then the frictional angle we calculate is higher than the correct value and the specimen is apparently mobilised to value higher than 1. This can be the serious problem when using results as the design parameters. It is still in dilemma to confirm which pore pressure measurement is correct and should be adopted in the design. Few experiments cannot confirm it and require more tests to make both the quantitative and qualitative analysis in this case. The conclusion that can be made from this test are summarised below.

- Pore water pressure varies inside the soil during the undrained triaxial test.
- During the consolidation phase, the specimen is repeatedly subjected to the cyclic load due to increase and decrease of pore water pressure at each step.
- While consolidating the soil along the K_0 line, it should be done in number of small loading steps especially at initial steps. While reconsolidating sample back to the insitu stress, it should also be done in number of small loading steps.
- During consolidation along K_0 line, the clay soil is apparently mobilised to the higer degree of shear mobilisation due to the pore pressure inside the sample.

- In shearing phase, if strain rate is high then pore pressure variation at the middle and surface of the sample is also higher than at lower strain rate.
- The pore pressure variation is higher at low strains than at higher strains.
- The pore pressure at the surface shows decrease in internal frictional angle than frictional angle obtained considering pore pressure at the middle of the specimen.
- The pore water pressure inside the sample causes the failure in the sample, although the sample is subjected to lower constant stress level or creep.
- The creep process starts with the decrease in strain rate and then becomes contant and finally fails after certain time of increase in strain rate.
- If the strain rate is decreasing during creep phase, failure cannot be predicted. it may not fail at all and the test should be stopped manually. Failure by creep can only be predicted after the increase in strain rate.
- The experimental technique becomes more complex when pore pressure is to be measured at the differerent location in triaxial sample.

Chapter 6

6.0 Recommendation for further work.

The quantitative results could not be drawn due to the lack of specimen for the test and limited time available for the test and delay in getting the sample from the site. However, test results show significant difference in the pore water pressure measured at middle of the sample and at periphery of the sample. The pore pressure sensor used, has limited capacity and hence, sufficient study could not be made during the reconsolidation of the sample from the preconsolidation level to the insitu stress level. It also limited the study of the pore pressure variation at large strains and study of exact failure surface. It seems necessary to measure the pore pressure at the bottom of the sample as well. Generally, the failure surface starts at the bottom of the specimen and hence the pore pressure at this point can be governing pore pressure than other location. So, the pore pressure at bottom of the sample should also be measured. Quantitative analysis should be made to determine maximum value of strain rate below which tests should be performed to get uniformity in the pore pressure inside the specimen.

It was supposed to make the study on the shear creep in sensitive clay. But due to different constraints, it could not be completed in sufficient way. Hence, I suggest to do more undrained creep test at different degree of mobilisation and determine time of failure of the sample in creep. The minimum strain rate at different mobilisation should be calculated and plotted against time to rupture inorder to trace the failure of soil in creep. The maximum stress level should be determined at which the soil will not fail at all in creep which means that if the slope is loaded with stress greater than this maximum value, it will fail. From this we can also determine minimum pore pressure that will push the soil to failure in creep. It can also be verified if the slope is loaded with stress below its post peak strength, the slope won't fail at all in creep.

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8.0 Appendix

8.1 Loading details for Tiller clay

Consolidation Phase for test 1

Cell Pressure (σ_3') Kpa	Total axial load (σ_1') Kpa	Piston load ($\Delta\sigma_1'$) Kpa	Remarks
12.4	20	7.6	
24.7	40	15.3	
37.1	60	22.9	
49.4	80	30.6	
61.2	99.01	37.8	Insitu state
74.2	120	45.8	
86.5	140	53.5	
98.9	160	61.1	Preconsolidation state
86.5	140	53.5	
74.2	120	45.8	
61.2	99.01	37.8	Insitu State

Consolidation phase for test 2

Cell Pressure (σ_3') Kpa	Total axial load (σ_1') Kpa	Piston load ($\Delta\sigma_1'$) Kpa	Remarks
14.5	20	5.5	
28.9	40	11.1	
43.4	60	16.6	
57.9	80	22.1	
71.6	99.01	27.4	Insitu state
86.8	120	33.2	
101.3	140	38.7	
115.8	160	44.2	Preconsolidation state
101.3	140	38.7	
86.8	120	33.2	
71.6	99.01	27.4	Insitu state

8.2 Sample of Calculations

KrypTriax	09.06.2015
Header	
Operator	suresh
Site	tiller
Hole	
Depth	11.4
Sample height	50
Sample diameter	54
Comment	
Calibration data	
Steps/mm	100000
Load cell (N/V)	100
Diff sensor (kPa/V)	50
Cell sensor (kPa/V)	100
Back sensor (kPa/V)	100
Burette (cm3/V)	20.79
X sensor (x/V)	3.45
Pr2Load (N/kPa)	0.1

				Diff	Cell									Sample a	σ1	σ1'	σ3'	σ1'	(σ1-σ3)/2
Mean	Motor	Deform	Force	pr.	pr.	Burette	X	MODE	εv	ε1	time	time	u	Area		mid	mid	surface	
time	[steps]	[mm]	[N]	[kPa]	[kPa]	[cm3]	value				sec	min	surface	mm^2	KPa	KPa	KPa	KPa	KPa
------------	-------	-------	-------	-------	-------	-------	-------	-------											
21:24.3	0	0.000	0.0	0.0	0.0	0.0	0.0			0.0000	0								
21:24.3	485.1	0.005	-0.6	12.6	12.3	0.0	5.4	CONS	0.0003	0.0001	0	0.0	-0.3	2289.6	12.1	6.7	7.0	12.4	-0.1
21:34.3	1132.56	0.011	0.0	12.5	12.3	0.0	5.5	CONS	0.0004	0.0002	10	0.2	-0.2	2289.7	12.4	6.9	6.9	12.5	0.0
21:44.3	1238.5	0.012	0.2	12.4	12.4	0.0	5.8	CONS	0.0004	0.0002	20	0.3	-0.1	2289.7	12.5	6.7	6.6	12.6	0.1
21:54.2	1793	0.018	2.2	12.4	12.4	0.1	6.7	CONS	0.0005	0.0004	30	0.5	0.0	2289.9	13.4	6.7	5.7	13.4	0.5
22:04.2	2793	0.028	5.4	12.4	12.4	0.1	7.8	CONS	0.0005	0.0006	40	0.7	0.0	2290.2	14.8	7.0	4.7	14.8	1.2
22:14.2	3793	0.038	8.1	12.4	12.4	0.1	8.2	CONS	0.0006	0.0008	50	0.8	0.0	2290.5	16.0	7.7	4.2	16.0	1.8
22:24.2	4793	0.048	10.5	12.4	12.4	0.1	8.4	CONS	0.0006	0.0010	60	1.0	0.0	2290.8	17.0	8.6	4.0	17.0	2.3
22:34.2	5793	0.058	12.6	12.4	12.4	0.1	8.5	CONS	0.0007	0.0012	70	1.2	0.1	2291.1	17.9	9.4	3.9	17.9	2.7
22:44.2	6793	0.068	14.3	12.5	12.4	0.1	8.6	CONS	0.0008	0.0014	80	1.3	-0.1	2291.5	18.7	10.1	3.8	18.7	3.1
22:54.2	7564.76	0.076	15.1	12.5	12.4	0.1	8.2	CONS	0.0008	0.0015	90	1.5	-0.1	2291.7	19.0	10.8	4.2	19.1	3.3
23:04.2	7936.65	0.079	15.1	12.5	12.4	0.1	7.7	CONS	0.0009	0.0016	100	1.7	-0.1	2291.8	19.0	11.3	4.7	19.1	3.3
23:14.2	8209.54	0.082	15.1	12.5	12.4	0.1	7.5	CONS	0.0009	0.0016	110	1.8	-0.1	2291.9	19.0	11.5	4.9	19.1	3.3
23:24.2	8436.17	0.084	15.1	12.5	12.4	0.1	7.4	CONS	0.0010	0.0017	120	2.0	0.0	2291.8	19.0	11.7	5.0	19.1	3.3
23:34.2	8639.13	0.086	15.1	12.5	12.4	0.1	7.3	CONS	0.0010	0.0017	130	2.2	0.0	2291.7	19.0	11.7	5.1	19.1	3.3
23:44.2	8816.08	0.088	15.1	12.5	12.4	0.1	7.3	CONS	0.0011	0.0018	140	2.3	0.0	2291.7	19.0	11.7	5.1	19.1	3.3
23:54.2	8979.62	0.090	15.2	12.5	12.4	0.1	7.3	CONS	0.0011	0.0018	150	2.5	0.0	2291.8	19.0	11.8	5.2	19.1	3.3
24:04.2	9134.07	0.091	15.2	12.5	12.4	0.1	7.2	CONS	0.0011	0.0018	160	2.7	0.0	2291.7	19.0	11.8	5.2	19.1	3.3

				Diff	Cell													
Mean	Motor	Deform	Force	pr.	pr.	Burette	X				σ1	σ1'	σ3'	u	σ1'	(σ1-σ3)/2	time	time
time	[steps]	[mm]	[N]	[kPa]	[kPa]	[cm3]	value	MODE	εa	Area		mid	mid		at surface			
------------	-------	-------	-------	-------	-------	-------	-------	-------		mm^2	Kpa	Kpa	Kpa	Kpa	Kpa	Kpa	sec	min
0.911754	0	0.0	0.0	0.0	0.0	0.0	ZERO											
0.911754	191641	1.9	86.2	62.5	61.2	5.9	0.4	CSR	2.93231E-05	2258.6	99.3	98.9	60.7	-1.3	100.6	19.1	0	0.0
0.911869	191752	1.9	87.7	62.5	61.2	5.9	0.8	CSR	5.24072E-05	2258.6	100.0	99.3	60.4	-1.3	101.3	19.4	10	0.2
0.911985	191863	1.9	89.5	62.4	61.2	5.9	1.2	CSR	7.54913E-05	2258.7	100.8	99.6	60.0	-1.3	102.1	19.8	20	0.3
0.912101	191974	1.9	91.3	62.4	61.2	5.9	1.6	CSR	9.85754E-05	2258.7	101.6	100.0	59.6	-1.3	102.9	20.2	30	0.5
0.912217	192086	1.9	93.0	62.4	61.2	5.9	1.9	CSR	0.000121868	2258.8	102.3	100.4	59.3	-1.3	103.6	20.6	40	0.7
0.912332	192197	1.9	94.7	62.5	61.2	5.9	2.2	CSR	0.000144952	2258.8	103.1	100.9	58.9	-1.3	104.4	21.0	50	0.8
0.912448	192308	1.9	96.4	62.5	61.2	5.9	2.5	CSR	0.000168036	2258.9	103.8	101.3	58.6	-1.3	105.2	21.3	60	1.0
0.912564	192419	1.9	98.0	62.5	61.2	5.9	2.8	CSR	0.00019112	2258.9	104.6	101.8	58.4	-1.3	105.9	21.7	70	1.2
0.912679	192530	1.9	99.6	62.5	61.2	5.9	3.0	CSR	0.000214204	2259.0	105.2	102.2	58.1	-1.3	106.6	22.0	80	1.3
0.912795	192641	1.9	101.1	62.5	61.2	5.9	3.2	CSR	0.000237288	2259.0	105.9	102.7	57.9	-1.3	107.2	22.4	90	1.5
0.912911	192752	1.9	102.6	62.4	61.2	5.9	3.4	CSR	0.000260372	2259.1	106.6	103.1	57.8	-1.3	107.8	22.7	100	1.7
0.913027	192863	1.9	104.0	62.4	61.2	5.9	3.6	CSR	0.000283456	2259.1	107.2	103.6	57.6	-1.2	108.4	23.0	110	1.8
0.913142	192974	1.9	105.4	62.4	61.2	5.9	3.7	CSR	0.000306541	2259.2	107.8	104.1	57.5	-1.2	109.0	23.3	120	2.0
0.913258	193085	1.9	106.7	62.4	61.2	5.9	3.8	CSR	0.000329625	2259.2	108.4	104.6	57.4	-1.2	109.6	23.6	130	2.2
0.913374	193196	1.9	108.1	62.3	61.2	5.9	3.9	CSR	0.000352709	2259.3	109.0	105.1	57.3	-1.2	110.1	23.9	140	2.3
0.913489	193308	1.9	109.4	62.3	61.2	5.9	4.0	CSR	0.000376001	2259.4	109.6	105.6	57.2	-1.1	110.7	24.2	150	2.5
0.913605	193419	1.9	110.7	62.3	61.2	5.9	4.0	CSR	0.000399085	2259.4	110.2	106.1	57.1	-1.1	111.2	24.5	160	2.7
0.913721	193530	1.9	112.0	62.2	61.2	5.9	4.1	CSR	0.000422169	2259.5	110.7	106.6	57.0	-1.1	111.8	24.8	170	2.8
0.913837	193641	1.9	113.2	62.2	61.2	5.9	4.2	CSR	0.000445253	2259.5	111.3	107.1	57.0	-1.0	112.3	25.1	180	3.0
0.913952	193752	1.9	114.4	62.2	61.2	5.9	4.2	CSR	0.000468337	2259.6	111.8	107.6	56.9	-1.0	112.8	25.3	190	3.2
0.914068	193863	1.9	115.7	62.1	61.2	5.9	4.3	CSR	0.000491421	2259.6	112.3	108.1	56.9	-1.0	113.3	25.6	200	3.3
0.914184	193974	1.9	116.9	62.1	61.2	5.9	4.3	CSR	0.000514506	2259.7	112.9	108.6	56.8	-0.9	113.8	25.9	210	3.5
0.914299	194085	1.9	118.0	62.1	61.2	5.9	4.4	CSR	0.00053759	2259.7	113.4	109.0	56.8	-0.9	114.3	26.1	220	3.7