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Hard Soil/Soft Rock Triaxial Test Procedure

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Abstract

Hard soils/Soft rocks are critical materials which show undesirable behavior. The features of soft rocks/hard soils such as low strength, fast weathering, crumbling are hindrance for important engineering works at a site dominated by soft or weak rocks.

Triaxial testing is an important procedure in geotechnical engineering for understanding the behavior and properties of both soils and rocks. In this thesis during the literature study, the triaxial tests on different soft rocks such as clay shales, claystone, breccia, mudstone, opalinus clay were studied and examined. The features and techniques which are important for executing an appropriate triaxial testing procedure on weak or soft rocks. The challenges in sampling, storage, specimen preparation, and testing of soft rocks are discussed. Furthermore, light has been shed on factors (such as: water content, saturation, specimen geometry, loading/strain rate) that affect the properties of weak rocks.

The concepts of both soil mechanics and rocks mechanics are applied in order to establish a proper triaxial testing procedure. This is because soft rocks fall in the category between soils and rocks for their intermediate strength. The appropriate triaxial testing procedure for soft rocks, i.e., multistage testing procedure is explained. The three stages, saturation stage, consolidation and shearing stage are all discussed in detail.

Table of contents

1	Introduction	6
1.1	Background and Motivation of the Thesis.....	7
1.2	Study Objectives	7
1.3	Thesis Organization	8
2	Introduction to Soft Rock	9
2.1	Definition of Soft Rock.....	9
2.2	Types of Soft Rock.....	11
2.3	Factors Affecting the Strength of Soft Rock.....	13
2.3.1	Specimen Geometry.....	13
2.3.2	Rate of Strain Application	14
2.3.3	Water content and Saturation	14
2.3.4	Porosity and Dry Density	15
2.3.5	Weathering.....	16
2.3.6	Static friction between piston and cell.....	17
2.3.7	Temperature.....	18
3	Challenges in Lab Testing of Soft Rock	20
3.1	Recommended Sampling Techniques for Soft Rock	23
3.1.1	Double Tube Sampler	23
3.1.2	Triple Tube Sampler.....	24
3.1.3	Wireline Core Barrels	25
3.2	Specimen Preparation.....	25
3.3	Testing Equipment	25
4	Literature Research	28
4.1	Appropriate Technique for Triaxial Testing of Soft Rock.....	29
4.1.1	Sample Extraction, Storage and Preparation.....	29
4.1.2	Water Content and Saturation	30
4.1.3	Specimen Geometry.....	30
4.1.4	Rate of Strain Application	30
4.2	Triaxial Test Procedure of Soft Rock	31
4.2.1	Saturation Stage.....	31
4.2.2	Consolidation Stage	34

4.2.3 Shearing Stage	37
4.3 Assessment of Results.....	40
5 Summary and Conclusion	44
6 Publication bibliography	47

List of figures

Figure 1. Classification of rocks according to the strength by various authors (modified after (Galván 1999)).....	9
Figure 2. Weak rocks linkage by geological processes with soils and hard rocks (Nickmann et al. 2006).....	10
Figure 3. Graph of dry density vs. porosity with plots of different rock types, indicating usual limits between hard and soft rocks, and with soils (modified after (Kanji and Galván 1998))	10
Figure 4. Sedimentary rocks from the Paraná Basin, showing the decrease of porosity and increase in strength with the geological age (modified after (Bosio JJ 1998))	12
Figure 5. Formation of weak rocks and their position in the subjects of soil and rock mechanics (Dobereiner and Freitas 1986)	12
Figure 6. Influence of L/D on peak deviator stress (Chiu et al. 1983)	13
Figure 7. Influence of L/D on secant modulus (Chiu et al. 1983)	13
Figure 8. Influence of axial strain rate on pore-water pressures (Constant strain rate test) (Chiu et al. 1983)	14
Figure 9. Typical stress-strain curve for Uniaxial compression tests (Johnston and Chiu 1984)	15
Figure 10. Relationship between UCS and Dry Density (Agustawijaya 2007)	16
Figure 11. Relationship between UCS and Porosity (Agustawijaya 2007).....	16
Figure 12. Test results of the stress-strain curve (a) Original stress-strain curve (b) Corrected stress-strain curve considering friction (Dong et al. 2013)	17
Figure 13. Friction force and radial pressure at different axial loading rates (Dong et al. 2013)	18
Figure 14. Apparent volumetric strain and temperature over time (Dong et al. 2013).....	19
Figure 15. Apparent and corrected volumetric strain over time (Dong et al. 2013)	19
Figure 16. Disaggregated rock core (Kanji 2014)	20
Figure 17. Undisturbed soft rock blocks (Kanji 2014)	21
Figure 18. Drill for coring of sample	22

Figure 19. Portable sample cutting tool.....	22
Figure 20. Single Tube Core Barrel (Left) & Double Tube Core Barrel (Right)	23
Figure 21. Triple Tube Barrel (ROSCHEM Geotechnical Drilling, Exploration and Rock Drilling products).....	24
Figure 22. Layout of the testing equipment at ETH Zurich (Dong 2017)	27
Figure 23. Disassembled triaxial cell (Dong 2017)	27
Figure 24. Pressure increase during the flushing phase (Wild 2016).....	32
Figure 25. Saturation phase with back pressure stages (Wild 2016)	33
Figure 26. Values of Skempton's pore pressure coefficient obtained for individual B-check during saturation phase (Wild et al. 2017).....	34
Figure 27. Example of consolidation stage showing the increase in confinement and pore pressure, local axial and radial strains and the back volume. The change in back volume indicates the change in water content (Wild et al. 2017)	35
Figure 28. Pore pressure evolution during CD tests on opalinus clay (Wild 2016).....	38
Figure 29. Pore pressure at peak up compared to initial pore pressure u_0 as a function of confining stress. The grey area represents the 5 % interval around $u_p/u_0 = 1$ (Wild 2016)	39
Figure 30. Effective stress paths during Consolidated Undrained Tests (CU) (Wild 2016)	40
Figure 31. Possible peak strength failure envelopes for all specimens of opalinus clay tested (Wild 2016)	41
Figure 32. Ultimate ($c' = 0.8$ MPa, $\phi' = 18^\circ$) and peak shear strength ($c' = 2.6$ MPa, $\phi' = 22^\circ$) failure envelopes for all specimens tested (Favero et al. 2018).....	42
Figure 33. Comparison of the peak strength results with further data on Opalinus Clay (OPA) and Callovo-Oxfordian claystone (COx) (modified after (Favero et al. 2018))	42
Figure 34. Peak strength of Pierre II Shale (Wu et al. 1997)	43

List of tables

Table 1. Types of soft rocks (Kanji 2014)	11
Table 2. Comparison of different triaxial testing for soft/weak rocks (Dong 2017)	26
Table 3. Triaxial tests on different types of weak rocks	36

1 Introduction

Soft rocks/Hard soils are critical geo-materials as they have several types of critical issues. They show undesirable features and behaviors like, low strength, crumbling, disaggregation, high plasticity, slaking and fast weathering. These types of unsuitable behaviors are hurdles for important engineering works at a site dominated by soft rocks. Better geological conditions are important for the construction of dams and hydroelectric power plants. For tunnel and highways, it is important to implement better alignment and avoid weak zones, whenever possible. Sometimes the hard competent strata may not be available or the best feasible geological sites are already utilized. Therefore, it is necessary to understand soft rocks and to develop appropriate solutions for the encountered problems and challenges.

Triaxial tests are used to determine rock properties under confined conditions, as they occur in the in-situ rock mass. The knowledge of the shear strength for rock and rock mass is essential. Shear failure can occur in rock joints as well as in intact rock. The drained and undrained behavior of clays/clay shales are mainly studied through three types of tests in triaxial testing conditions:

- The unconsolidated undrained (UU) test
- The consolidated undrained (CU) tests
- The consolidated drained (CD) test

Pore pressures are not measured and considered in the analysis during unconsolidated undrained (UU) test. The advantage of such tests is that no back-saturation of the specimens is required, which usually is challenging and time-consuming. However, the total stress results of UU tests cannot be used for long-term calculations where effective stress properties are required. Pore pressures are measured during consolidated undrained (CD) and consolidated undrained (CU) tests and effective parameters are obtained as both consider a consolidation stage before shearing. In consolidated drained (CD) tests, the drainage is permitted during shearing (pore pressure is kept constant), while in consolidated undrained (CU) tests, the drainage is not permitted during shearing (excess pore pressure is allowed to build up).

The determination of realistic mechanical parameters for weak or highly fractured rock is a challenge, as it has intermediate strength between soils and hard rocks. Sometimes, they are too soft to be tested in rock mechanics equipment and too hard for soil mechanics equipment.

1.1 Background and Motivation of the Thesis

In many engineering applications it is important to determine both effective rock properties and the rock behavior which are representative for the problem's in-situ conditions. For this purpose, rock samples are usually extracted from the ground and brought to the laboratory to perform laboratory experiments such as consolidated drained and undrained triaxial tests. The triaxial testing procedure is well documented and standardized for soil and as well as hard rocks but with soft and weak rocks, the triaxial test could not be claimed as well-documented. Since soft/weak rocks have not been paid much attention in the past, they are, therefore, not understood and studied in depth. Therefore, in order to guarantee safety regarding construction in weak or soft rocks, uneconomical and conservative parameters are adopted. In conventional triaxial tests on rocks, the pore water pressure is neither controlled nor measured; therefore, the effective stresses cannot be determined. On the contrary in the case of standard triaxial tests for soils, the pore water can be controlled as well as measured. As weak or soft rock falls in between soils and rocks, so both soil mechanics and rock mechanics techniques should be adopted in order to accurately estimate the strength parameters.

For low permeable geomaterials, such as clay shales, core extraction, handling, storage, and specimen preparation can lead to a reduction in the degree of saturation and the effective stress state in the specimen prior to testing remains uncertain. Testing of such materials is challenging due to the low permeability which requires long test durations, the material's sensitivity to fluids, or the desaturation during sampling and handling of the specimens. Suction or partly saturated conditions can have a substantial influence on the effective strength properties and stiffness. Therefore, it is important to give more attention to the exploration, sampling and testing of procedures (Blümel 2005).

1.2 Study Objectives

The scope of this thesis is to better understand the triaxial testing procedure adopted in case of weak or soft rocks. The main objective of this thesis work is to undertake literature research about an appropriate triaxial testing procedure which can be applied to soft or weak rocks to determine reliable strength parameters in the laboratory, that are necessary for important engineering constructions. The important features and techniques that are vital for the execution of the triaxial test on soft or weak rocks such as specimen sampling and preparation techniques and the challenges faced in testing such rocks are also mentioned and discussed.

For weak or soft rock triaxial testing, concepts of rock mechanics as well as soil mechanics are applied to implement an appropriate triaxial testing procedure. The major and important key stages such as saturation, consolidation and shearing, which must be considered during the triaxial testing of soft or weak rocks are explained in this thesis. Furthermore, some of the factors such as specimen geometry, rate of strain application, porosity, density, water content, saturation, weathering and temperature which affects the strength of weak rocks are mentioned.

1.3 Thesis Organization

The thesis is structured in five main chapters with the aim of explaining the topic associated with soft or weak rocks.

In chapter 2, soft rock is defined, including types of soft rocks and a discussion about factors affecting the strength of soft rock.

Chapter 3 is dedicated to the challenges in lab testing of soft rocks with an emphasis on sampling and specimen preparation techniques as well as the testing equipment.

The appropriate triaxial testing of soft weak rocks and the important features of the test are given in chapter 4. The procedure is compiled from the in-depth study of different triaxial testing procedure on weak or soft rocks in the past by various researchers worldwide.

Finally, chapter 5 summarizes the main discussion and conclusion of this thesis.

2 Introduction to Soft Rock

2.1 Definition of Soft Rock

In 1981 the International Society for Rock Mechanics (ISRM) defined rock with an UCS (uniaxial/compressive strength) in the range of 0.25 MPa to 25 MPa as 'extremely weak' to 'weak'. Weak rock falls in a category which can be defined as "a sort of twilight zone between soils and hard rocks" (Little 1976). In figure 1, the intact rock is being classified by various researchers according to strength. The upper limit for soil is also defined to be UCS of 0.4 MPa and SPT of above 50 (Terzaghi et al. 1967).

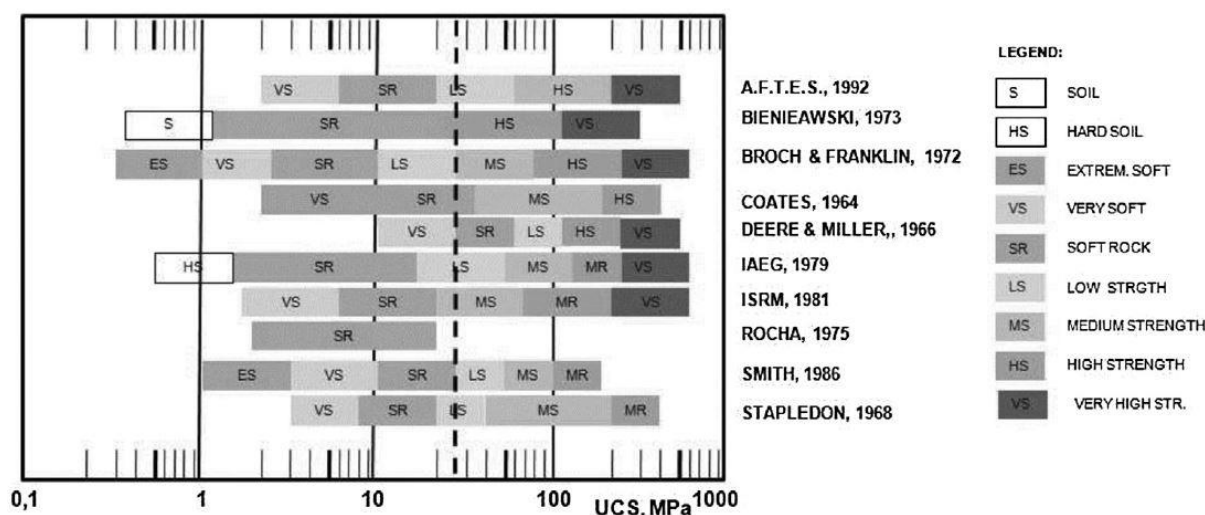


Figure 1. Classification of rocks according to the strength by various authors (modified after (Galván 1999)).

In the national standards (e.g. DIN 4022 T1, Ö-Norm B 2203 resp. DIN EN 14689-1, ASTM D 4644) weak rocks are a part of the group of rocks, in contradiction to soils. With respect to hard rock, they are sensitive to water and climatic changes. They disintegrate quickly when exposed to water. The strength loss cannot be reversed, whereas in cohesive soils it is possible due to changes in water content (Nickmann et al. 2006). Figure 2 shows that weak/soft rocks are linked by geological processes with soils and hard rocks.

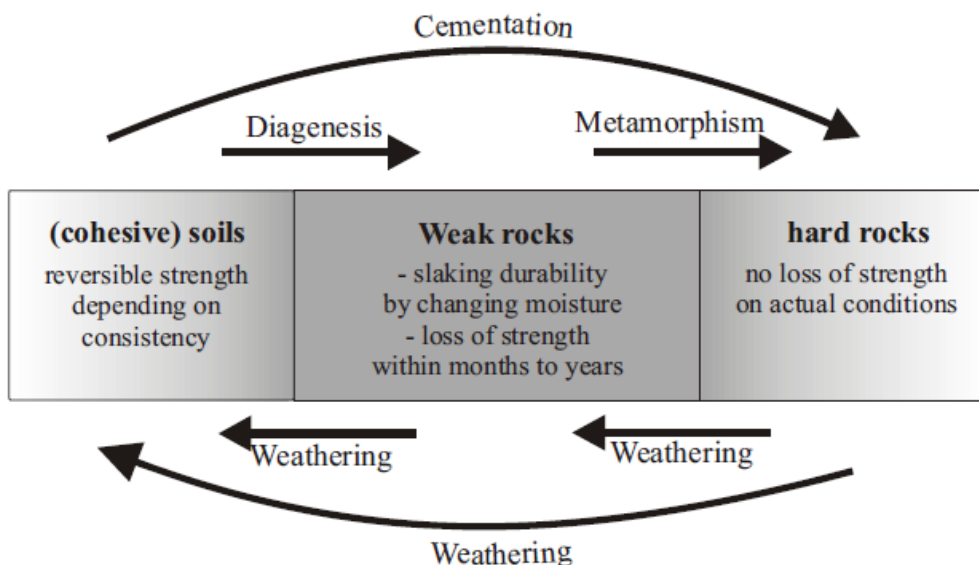


Figure 2. Weak rocks linkage by geological processes with soils and hard rocks (Nickmann et al. 2006)

(Galván 1999) studied different types of rocks and the relationship between dry density and porosity as shown in figure 3 and it can be seen in the figure that there is a continuous transition. The dispersion around the theoretical line may be due to differences in testing procedure or also it can be due to different lithologies.

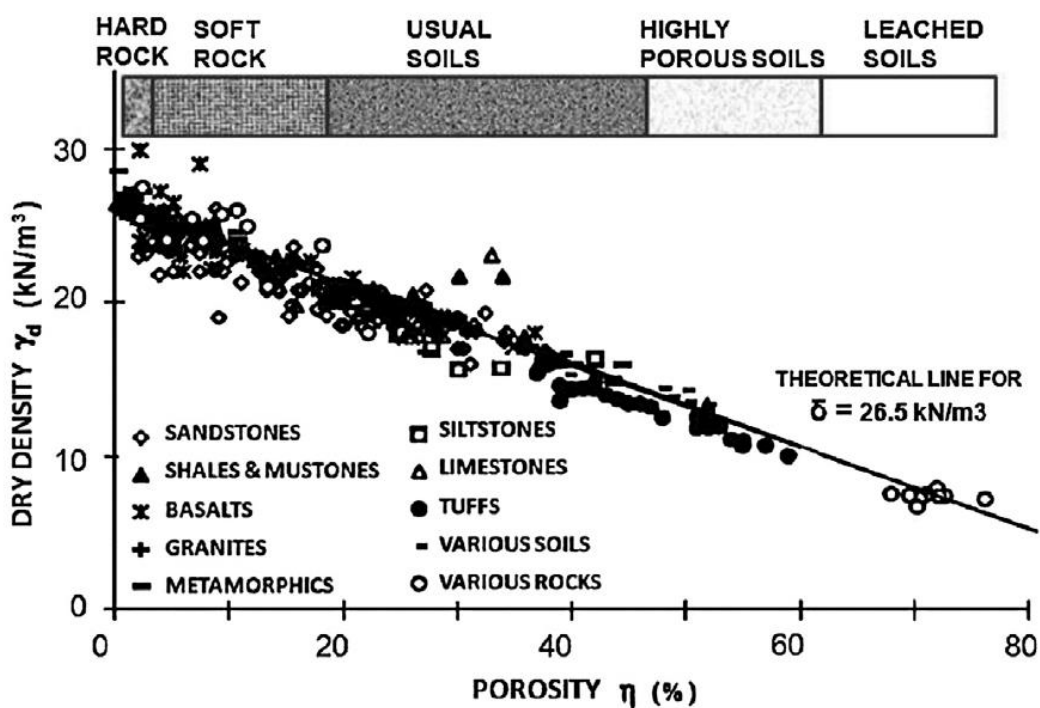


Figure 3. Graph of dry density vs. porosity with plots of different rock types, indicating usual limits between hard and soft rocks, and with soils (modified after (Kanji and Galván 1998))

2.2 Types of Soft Rock

The rocks which can called as soft rock are given in table 1.

Type of Rock	Subclasses
Sedimentary Rocks	Clastic: mudstone, shale, siltstone, sandstone, conglomerate and breccia, and marl Evaporites: salt rock, carnallite, etc. Soluble: limestone, dolomite, and gypsum, and Coal
Igneous Rocks	Volcanic conglomerates, breccia, lahar, Basaltic breccia, Pyroclastic deposits, volcanic ash, tuff and ignimbrite, and Weathering products of crystalline rocks
Metamorphic Rocks	Slate, phyllite, schists, Metavolcanic deposits

Table 1. Types of soft rocks (Kanji 2014)

According to (Kanji 2014), the geological or lithological name alone may be misleading, for example, sandstone can be very soft if the cement ratio is less or very hard if well cemented. Furthermore, (Bosio JJ 1998) illustrates that the geological age is also important when defining soft rock because the porosity decrease with age. Accordingly, due to diagenesis and burial pressure the strength also increases - as shown in figure 4. Therefore, (Kanji 2014) concluded that the concept of soft rocks is not yet well defined to date.

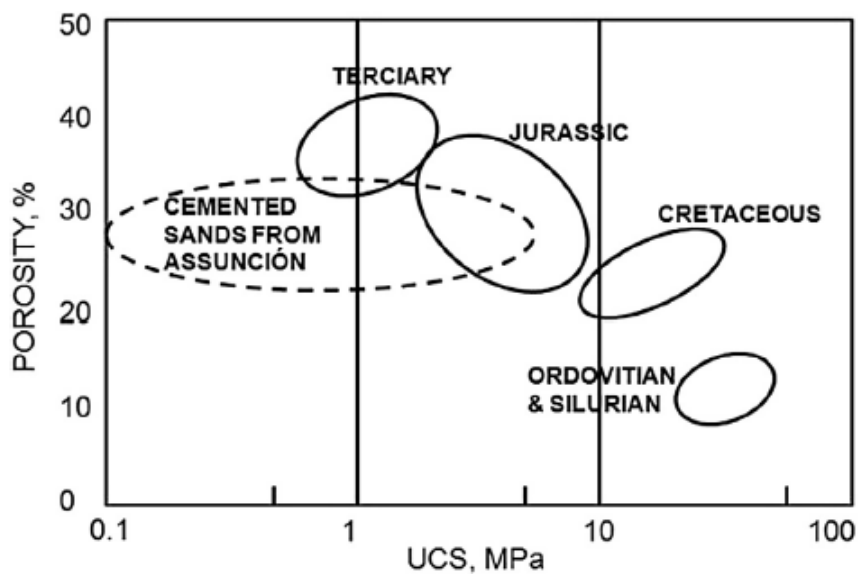


Figure 4. Sedimentary rocks from the Paraná Basin, showing the decrease of porosity and increase in strength with the geological age (modified after (Bosio JJ 1998))

In figure 5, we can see the origin and formation of weak rocks due to sedimentation and weathering and also the position of weak rocks according to soil mechanics and rock mechanics.

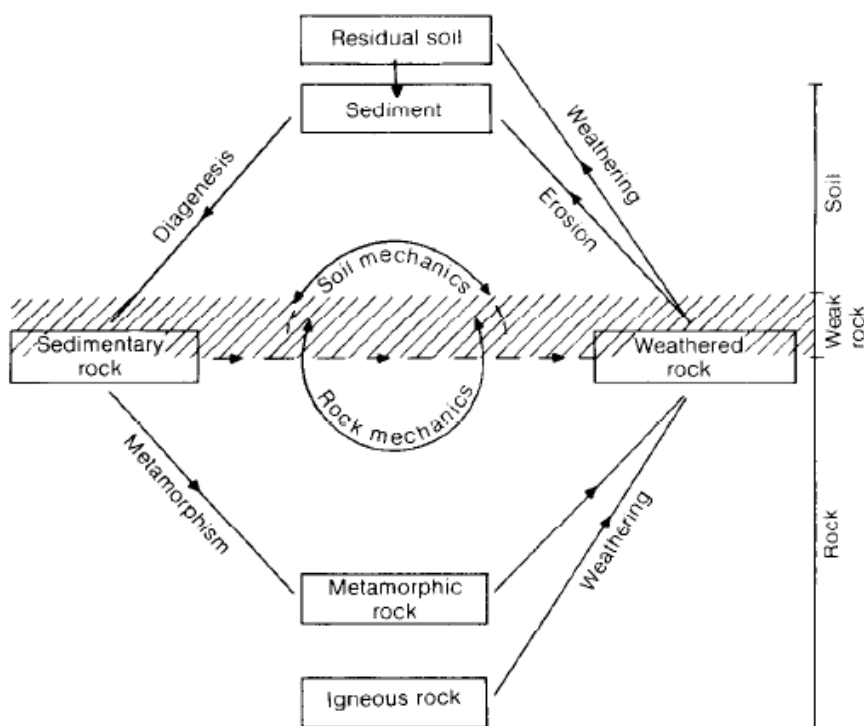


Figure 5. Formation of weak rocks and their position in the subjects of soil and rock mechanics (Dobereiner and Freitas 1986)

2.3 Factors Affecting the Strength of Soft Rock

2.3.1 Specimen Geometry

The ISRM suggested test methods recommend length to diameter ratio (L/D) of 2.0 to 3.0. (Chiu et al. 1983) performed drained triaxial tests on Melbourne Mudstone with length to diameter ratios (L/D) from 0.5 to 3.0 and found out that when the L/D ratio is atleast 2 the peak deviator stress and secant young's modulus tend to be constant as shown in figure 6 and figure 7.

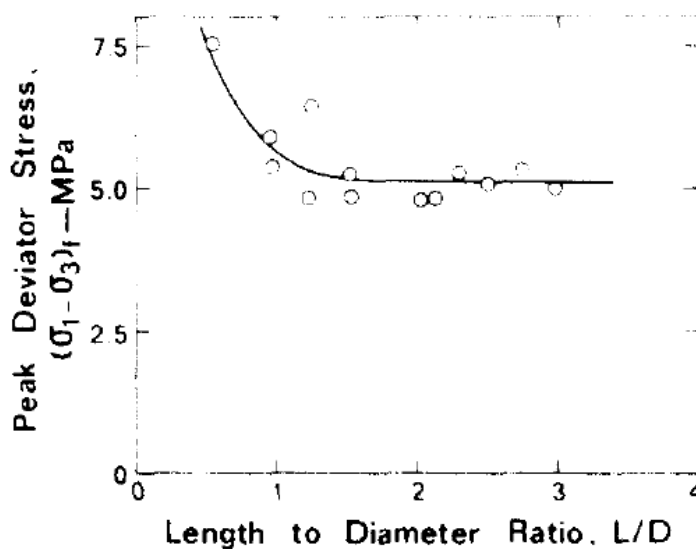


Figure 6. Influence of L/D on peak deviator stress (Chiu et al. 1983)

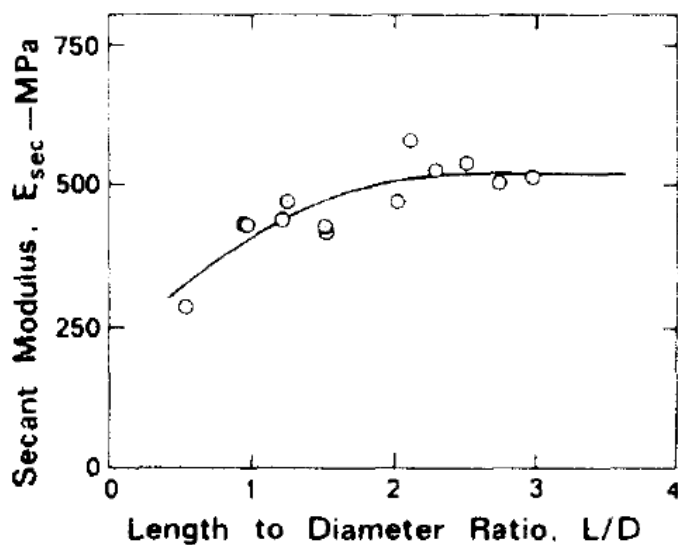


Figure 7. Influence of L/D on secant modulus (Chiu et al. 1983)

2.3.2 Rate of Strain Application

The strain rates have a significant influence on induced excess pore water pressure. The strain rates should be selected in appropriate range to ensure satisfactory dissipation of excess pore water pressure during the test. It was found out that (Chiu et al. 1983) concluded a strain rate less or equal to $3.3 \times 10^{-7} \text{ s}^{-1}$ is appropriate for complete dissipation and equalization of pore pressure for Melbourne mudstone as shown in figure 8. (Wu et al. 1997) tested Pierre II shale with lateral drainage made of fiberglass side drains placed around the specimen. A strain rate of $3 \times 10^{-8} \text{ s}^{-1}$ was found out to be sufficient for undrained test. Similarly experiments by other researchers (Deng et al. 2011; Hu et al. 2014; Islam and Skalle 2013; Dong et al. 2013; Wild et al. 2017; Belmokhtar et al. 2018) revealed that a loading/strain rate in the range of 10^{-7} s^{-1} , 10^{-8} s^{-1} and 10^{-6} s^{-1} , 10^{-7} s^{-1} , 10^{-8} s^{-1} was sufficient during drained and undrained loading respectively.

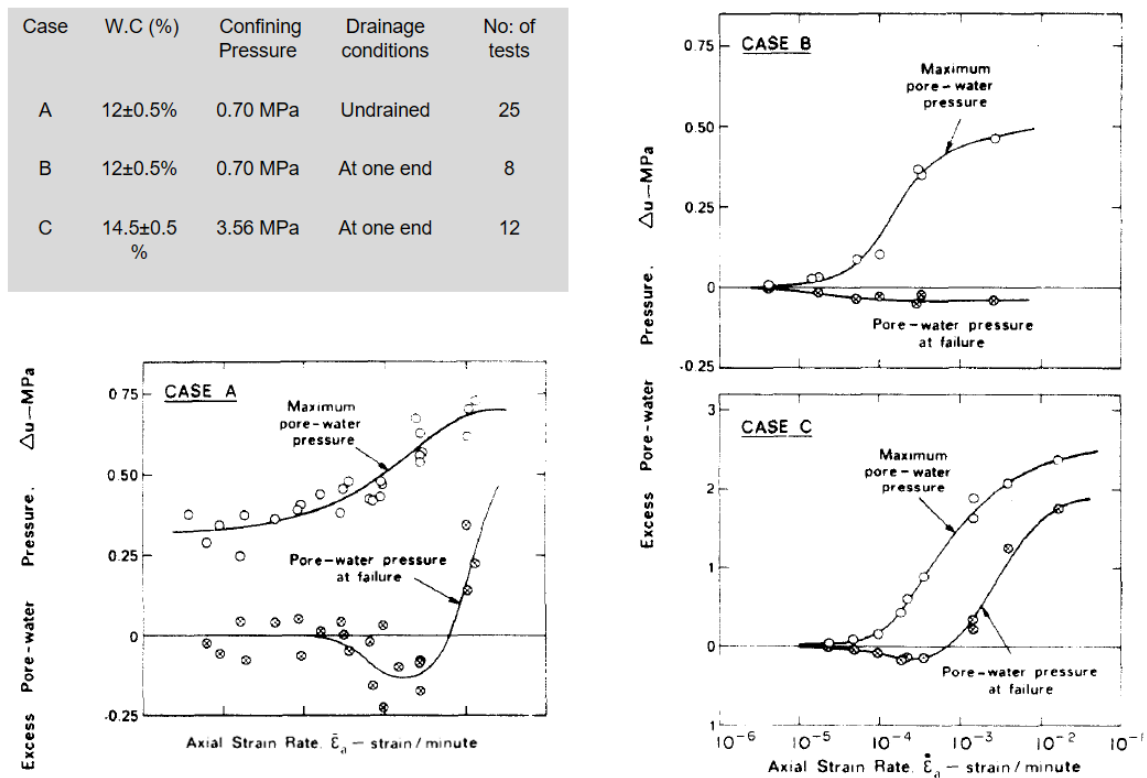


Figure 8. Influence of axial strain rate on pore-water pressures (Constant strain rate test) (Chiu et al. 1983)

2.3.3 Water content and Saturation

Water content or moisture content is the quantity of water contained in a material. The strength of rock due to changes in moisture content varies from rock to rock (Agustawijaya 2007). Generally, the strength of soft rock will decrease as the water content increases but the relationship between strength and moisture content is not always linear.

A study by (Johnston & Chiu) reveals that the UCS generally decreases with the increase of water content as shown in figure 9.

(Belmokhtar et al. 2018) showed that partial saturation may lead to biased assessment of mechanical properties of tested specimen, overestimating the shear strength with partially saturated specimens, and overestimating the young's modulus by running the partially drained tests.

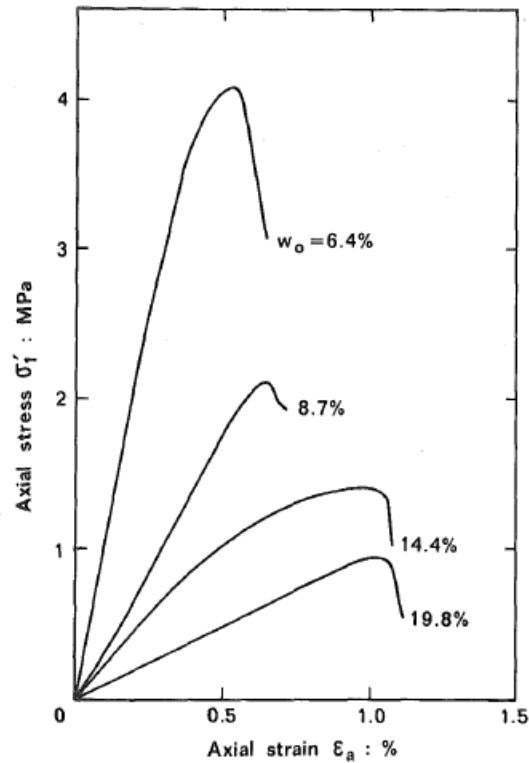


Figure 9. Typical stress-strain curve for Uniaxial compression tests (Johnston and Chiu 1984)

2.3.4 Porosity and Dry Density

Porosity can be defined as the ratio of pore volume to its total volume. Porosity is controlled by rock type, pore distribution, cementation, diagenetic history and composition. It is not controlled by grain size, as the volume of between-grain space is related only to the method of grain packing (Wikipedia contributors). The porosity of rock normally decreases with age and depth of burial.

$$Porosity = \frac{Pore\ Volume}{Total\ Volume}$$

The dry density is the ratio of the weight of the dry solids to the total specimen volume including pores. The UCS increases when the dry density increases while the UCS decreases as the porosity increases. Figure 10 and figure 11 shows the relationship between dry density, porosity and UCS.

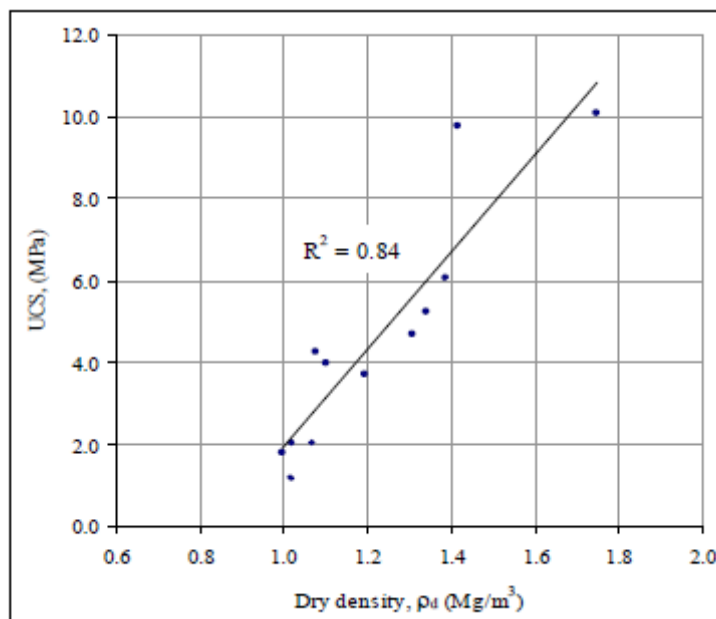


Figure 10. Relationship between UCS and Dry Density (Agustawijaya 2007)

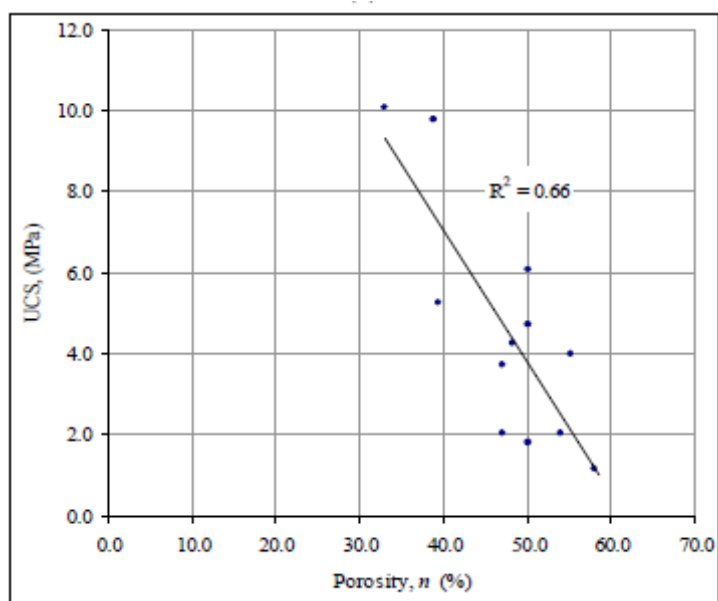


Figure 11. Relationship between UCS and Porosity (Agustawijaya 2007)

2.3.5 Weathering

The process of weathering by means of mechanical, chemical and biological action drastically affects the properties of the rock. Physical weathering results in the disaggregation of rocks without mineralogical change and chemical weathering results in the decomposition of the constituent minerals.

2.3.6 Static friction between piston and cell

(Dong et al. 2013) during the triaxial testing of weak rock, observed that static friction between piston and cell and temperature variation also affect the evaluation of the test results, therefore, they must be taken into account.

He explored that the stress strain curve of Consolidated Drained Test (CD test) shows a small axial movement of the piston of the triaxial cell which causes a large increase in the axial force at the start of loading and a large decrease in the axial force at the start of unloading as shown in figure 12a. This implies an unrealistic high stiffness of the weak rock, which is due to the static friction force that must be overcome to move the piston cell. The triaxial testing equipment used by (Dong et al. 2013) has a piston of 140 mm diameter and contains two carbon bands as guidance with O-rings for sealing and this sealing system with carbon bands and O-rings is the source of friction. The corrected stress-strain curve considering friction is shown in figure 12b.

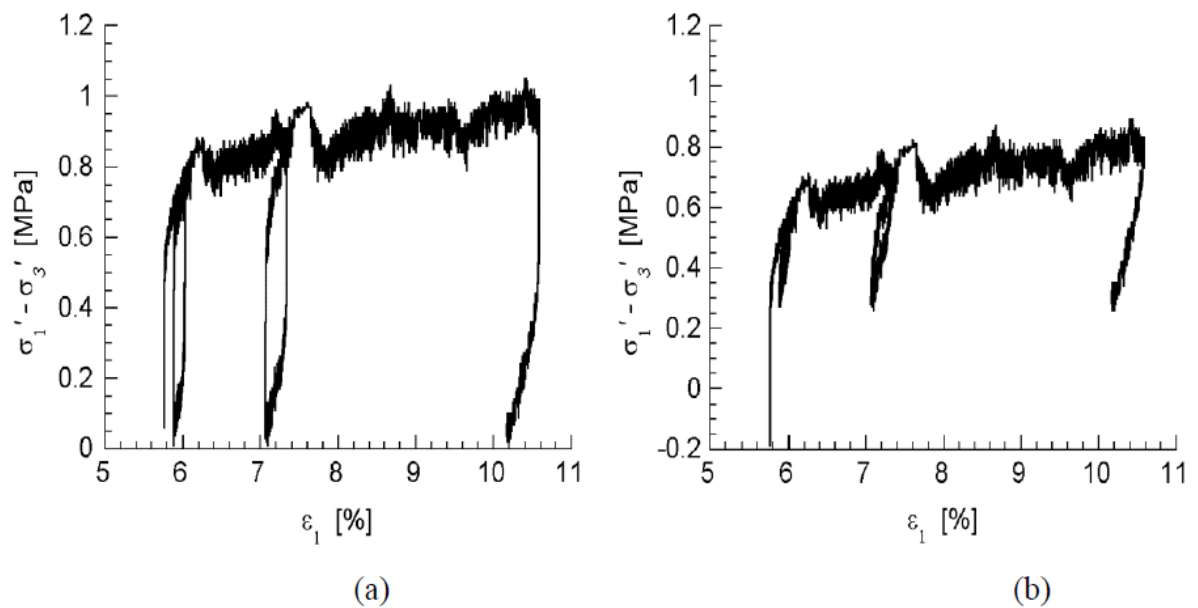


Figure 12. Test results of the stress-strain curve (a) Original stress-strain curve (b) Corrected stress-strain curve considering friction (Dong et al. 2013)

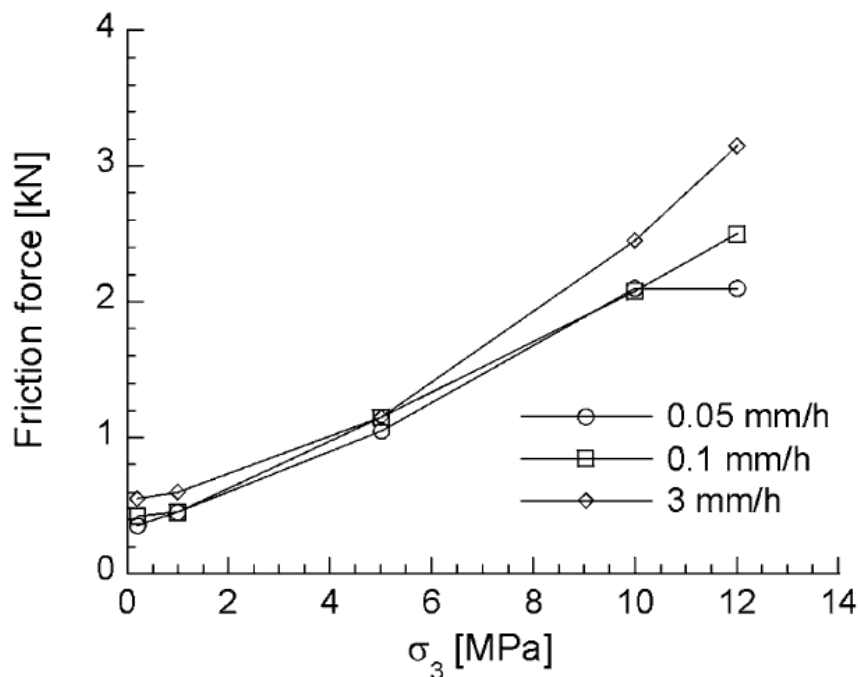


Figure 13. Friction force and radial pressure at different axial loading rates (Dong et al. 2013)

In the experiment executed by (Dong et al. 2013), a series of calibration tests were performed without putting the specimen in the cell in order to measure the magnitude of the friction force. The triaxial cell was completely filled with oil and the oil was not allowed to come into or out of the cell, by tightening the water connectors. After this, a constant displacement rate was applied to the loading piston while maintaining a constant oil pressure. The friction force concurs to the force increase necessary for allowing the piston to slide in the cell. Under different radial pressures and displacement rates several loading and unloading cycles were performed. A reversal in the direction of displacement, due to static force leads to a force change twice as large as the friction force. As shown in figure 13 above, the friction force and the oil pressure has almost a linear relationship.

2.3.7 Temperature

The temperature effect in triaxial test of extremely low permeable material may be relevant. The thermal expansion of the oil causes errors in the measurement of the volumetric strain of the specimen and should be taken into account in the calculation of the volumetric strain (Dong et al. 2013).

(Dong et al. 2013) found out that for a change in temperature up to 2-3° C, the error in the volumetric strain is around 2-3 % in case of oil, while in case of water it is 0.08-0.12 %, which is almost 30 times smaller than oil.

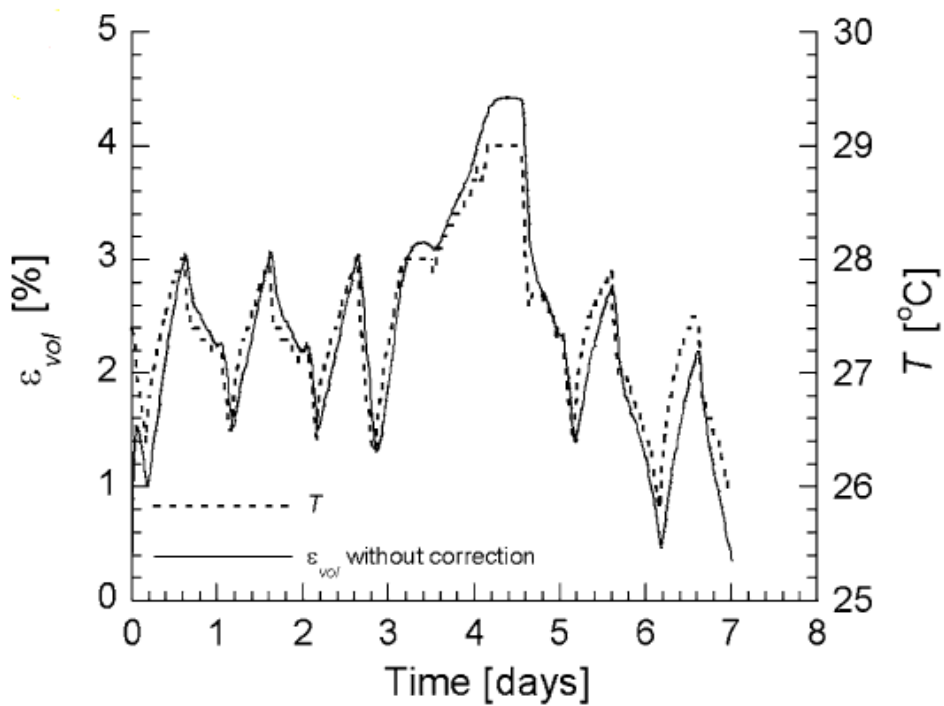


Figure 14. Apparent volumetric strain and temperature over time (Dong et al. 2013)

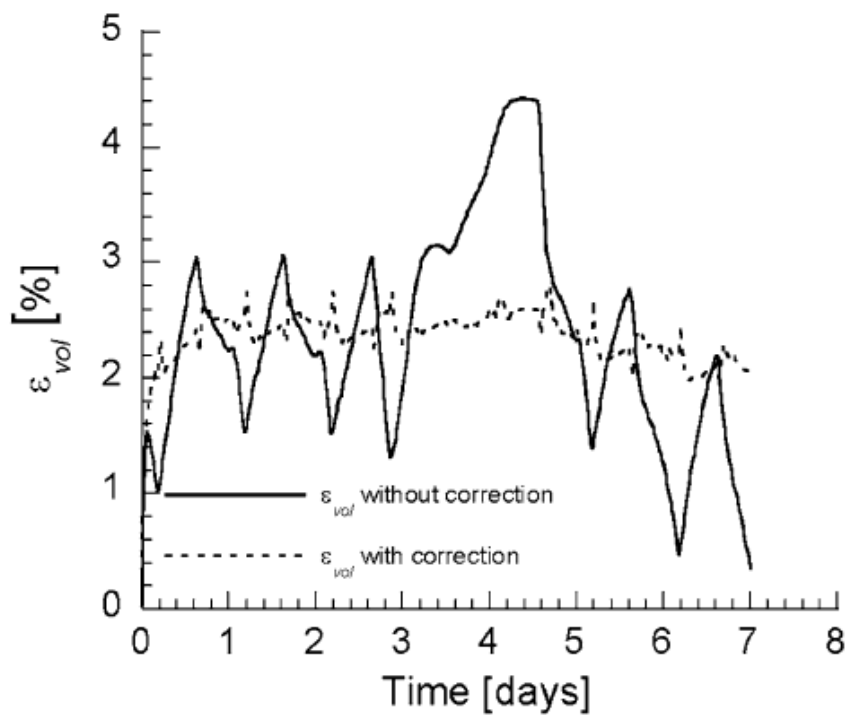


Figure 15. Apparent and corrected volumetric strain over time (Dong et al. 2013)

3 Challenges in Lab Testing of Soft Rock

In this chapter the challenges and hurdles in lab testing of soft rocks such as in their sampling and testing are discussed. The sampling of weak or soft rocks are very demanding. Sampling is difficult in most cases because of the nature of weak rocks. The techniques and procedure used for soils sometimes cannot be applied to weak rocks because the strength is often too high. The cores used for the testing of soft or weak rocks should be undisturbed because the quality of specimen is influenced by the drilling, sampling, transport, storage and specimen preparation.

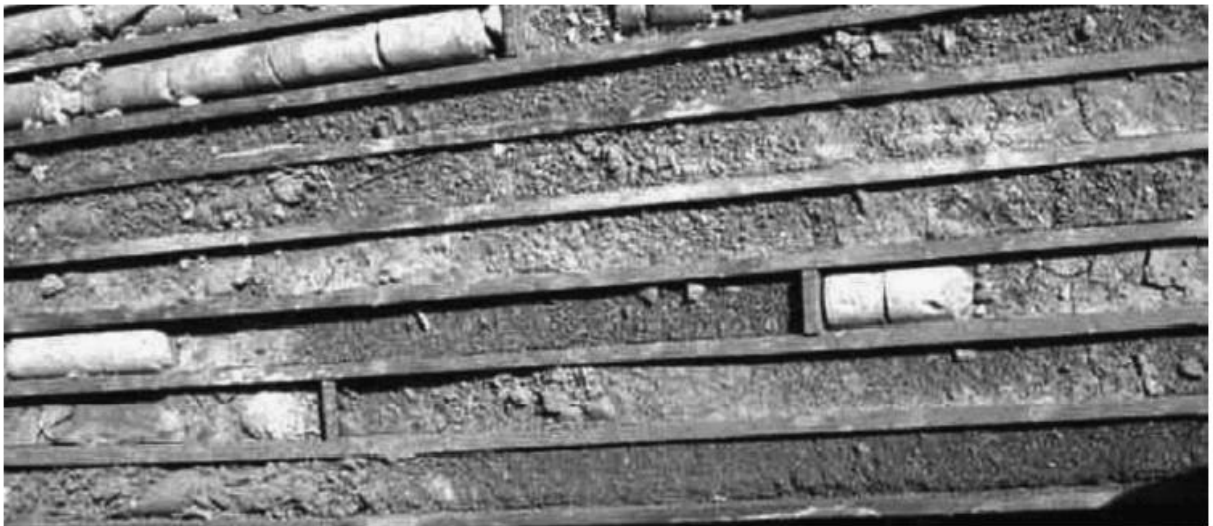


Figure 16. Disaggregated rock core (Kanji 2014)

The sampling, the storage and the specimen preparation should be according to standard guidelines. These standards are given by ISO 22475-1:2006 Geotechnical investigation and testing -- Sampling methods and groundwater measurements -- Part 1: Technical principles for execution. The standard practice for the rock core drilling and sampling of rock for site investigation is part ASTM designation: D2113-99 (American Society for Testing and Materials 2003).

For competent hard rocks drilling is usually done with diamond bits and a simple tube sampler but in case of weak soft rocks, diamond drillings destroy or partially destroy the sample as shown in figure 16, as well as the important geological feature is not recovered. Double tube sampler and especially triple tube sampler, discussed in the following sections

3.1.1 and 3.1.2, listed below, respectively, tend to improve the results obtained. Undisturbed samples of soft rocks can also be obtained by cutting as shown in figure 17 or by secant drilling to isolate sample blocks (Kanji 2014).



Figure 17. Undisturbed soft rock blocks (Kanji 2014)

In China, a new sampling system for undisturbed samples has been designed and developed by the State Key Laboratory of Geomechanics and Deep Underground Engineering, China University of Mining and Technology, Beijing, as shown in figure 18 and 19. The sampling system consists of:

- A drill which is driven with the help of compressed air
- A portable sample cutting tool
- A portable sample box

Geophysical and resistivity methods are also recommended. With the resistivity methods hidden weaker material and seismic refraction can show changes in velocity, of which weathering degrees and stratification can be derived (Kanji 2014).

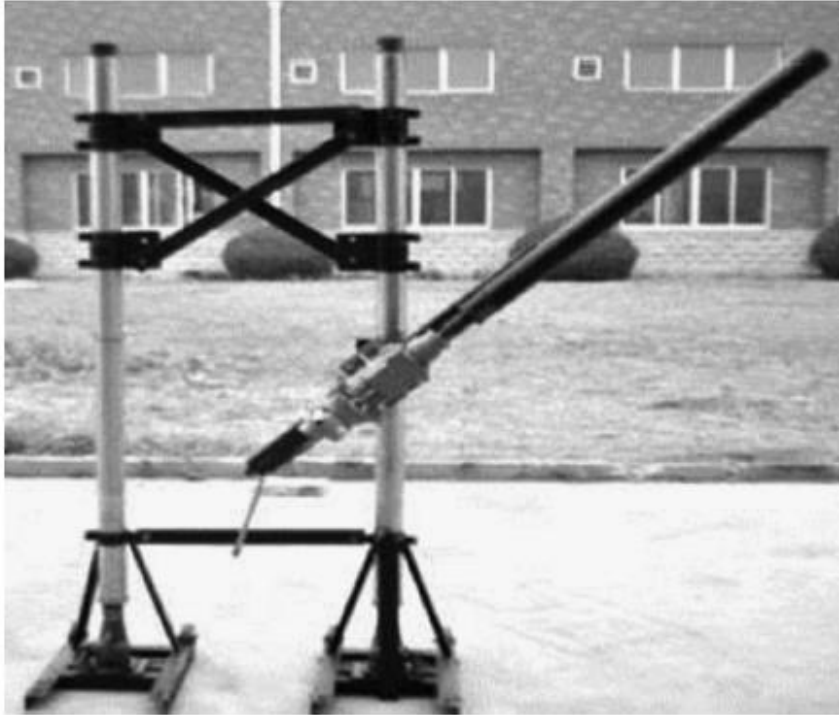


Figure 18. Drill for coring of sample



Figure 19. Portable sample cutting tool

3.1 Recommended Sampling Techniques for Soft Rock

The samples obtained from rock drilling is called rock cores. Rock coring is the process in which the sampler consisting of a tube (core barrel) with the cutting bit at its lower end cuts an annular hole in a rock mass thereby creating a cylindrical or core of rock which is recovered in the core barrel. Core barrels do function both of drilling and retaining the sample attached to drilling rod.

3.1.1 Double Tube Sampler

- The double-tube swivel type core barrels of large diameter can be used not only for quality core of sound rock, but also to provide samples of very stiff or hard clays.
- The double-tube core barrel contains an inner tube supported on a swivel, which does not rotate with the outer barrel.
- The outer barrel rotates with cutting bit and a flushing fluid passes between inner and outer tubes to the bit.
- The inner barrel is extended with a core catcher box which contains a split ring core catcher.
- The inner tube catches the core and protects the core from abrasion due to outer tube and also erosion from flushing fluid.

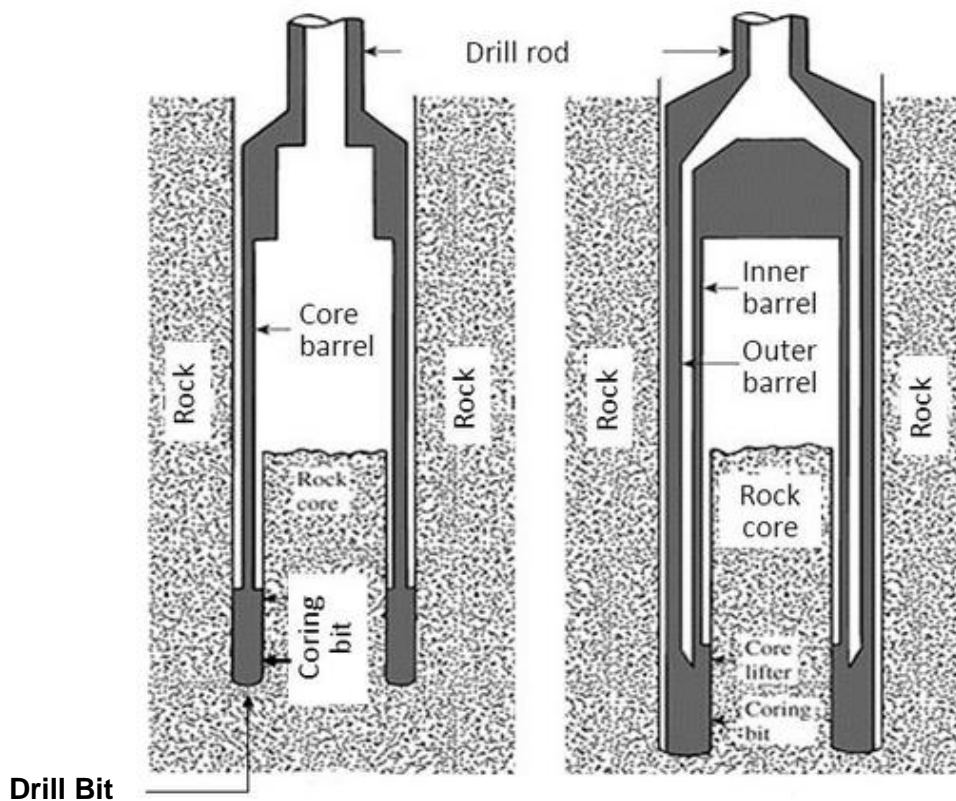


Figure 20. Single Tube Core Barrel (Left) & Double Tube Core Barrel (Right)

3.1.2 Triple Tube Sampler

- The triple tube sampler consists of three tubes, a liner (innermost tube), inner tube and an outer tube.
- Triple tube sampler can be retraceable type or a non-retraceable type.
- The non-retraceable triple tube sampler is recommended for obtaining good quality cores samples in fractured rock and highly weathered rocks.
- The initial cut is done by outer core while the second barrel is for cutting finer size. In the third barrel cored samples are collected.

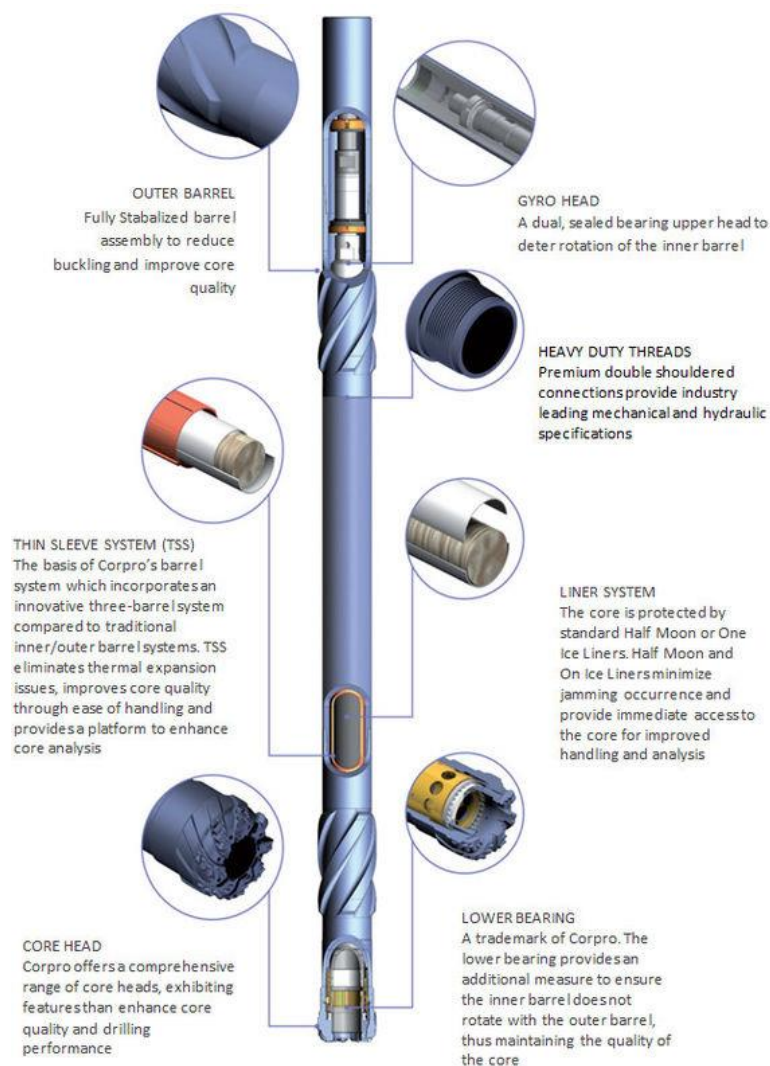


Figure 21. Triple Tube Barrel (ROSCHE Geotechnical Drilling, Exploration and Rock Drilling products)

3.1.3 Wireline Core Barrels

- Wireline core barrels can be double or triple tube wireline barrel.
- Wireline core barrels are recommended to use when coring to greater depths are required.
- By using wireline core barrels, the coupling and uncoupling of the rod string is not required when the core is retrieved.
- Direct coupling is done by dropping an overshot device on a cable, which allows fast retrieval of the core.
- An overshot device is dropped on a cable and directly couples onto the inner tube assembly, allowing fast retrieval of the core after each core run.

3.2 Specimen Preparation

The specimen preparation can vary according to the rock type. For example, clay and silt samples can be cut with knife while diamond saw can be used for rock samples. For the cutting of weak material consisting of soft matrix with hard inclusions, these techniques cannot be used. A more suitable and appropriate technique for weak/soft rocks is to use diamond band saw which operates with pressurized air instead of water cooling, and can be used for smooth and precise cutting of the sample into specimens.

3.3 Testing Equipment

In the field of geotechnical engineering, the triaxial test has been performed on soils and rocks. In case of soils, the specimen is almost saturated. The pore water conditions are measured and can be controlled and low stresses and low pressures are generally applied. To increase the degree of saturation, back pressure is applied. In undrained tests, excess pore water pressure should be dissipated during shearing so the rate of loading is low enough. While in case of undrained conditions, as the valve is closed therefore the pore water pressure is, therefore, measured so that effective stresses can be calculated. In both drained and undrained cases either the radial or the volumetric deformation should be measured with high accuracy.

In case of rocks, the pore water conditions are neither controlled nor measured. The loading is applied usually faster and the applied stresses are much higher compared to soils. Oil is used instead of water for filling the cell to generate the confining pressure. Furthermore, the triaxial testing equipment for soil and rock are different.

As the behaviour of soft rock is in between soils and rocks, therefore, the testing equipment

must consider all aspects using both soil and rock tests. The testing equipment must be able to apply extremely slow load and also it must allow the control of pore water pressure conditions (Chiu et al. 1983; Dong 2017), which is neither controlled nor measured in the conventional triaxial tests for rocks.

The soil mechanics test apparatuses can be used for testing under controlled pore water conditions but there are limitations such as low nominal loads, radial pressures (usually less than 1 MPa) and pore pressures (usually less than 0.2 MPa). Since the in-situ pressures are usually more than this, therefore, it is necessary to upgrade rock mechanics equipment with some of the capabilities of soil mechanics apparatus (Dong 2017).

As the testing setup is not standard, the modification of the testing equipment is done at several research institutes in the past. (Chiu et al. 1983) worked on triaxial testing of saturated soft rocks at Monash University, Australia. Similarly, further work is carried out on the triaxial testing of soft rocks at different institute around the world as given in table 2. The testing equipment for triaxial testing of soft rock ETH Zurich is shown in figure 22 and figure 23.

Institute	Rock type	Sample dimension	Axial load [kN]	Radial pressure [MPa]	Pore pressure [MPa]	Measurement of axial displacements	Measurement of volume change
Monash University	Mudstone	$D = 54 \text{ mm}$ $H/D = 0.5-3$	100/250	70	1.2	External DCDT	Volume of water expelled or entering the sample
MIT	Shale	$D = 84 \text{ mm}$ $H = 170 \text{ mm}$	15 [MPa]	12	1.8	External DCDT	Two proximity probes
ETH Zurich	Kakirites, Breccias	$D = 65/80/100 \text{ mm}$ $H/D = 1/2$	600	70	2	External LVDT	Volume of oil expelled or entering the triaxial cell/ Volume of water expelled or entering the sample
Politecnico di Torino	Coal	$D = 50/70/100 \text{ mm}$ $H = 200 \text{ mm}$	250 600	70	32	External and internal LVDT	Internal LVDT measuring the change of the circumference of the specimen
NGI	Sandstone, Clay-shale	$D = 38 \text{ mm}$ $H = 76 \text{ mm}$	1000	120	120	1 external and 2 internal LVDT	2 horizontal LVDT measuring the change of the diameter of the specimen
Universitat Politècnica de Catalunya	Clayey rocks	$D = 38/50 \text{ mm}$ $H = 76/100 \text{ mm}$	500	64	2	External LVDT and two internal LVDT	Radial strain-gauge extensometer

Table 2. Comparison of different triaxial testing for soft/weak rocks (Dong 2017)

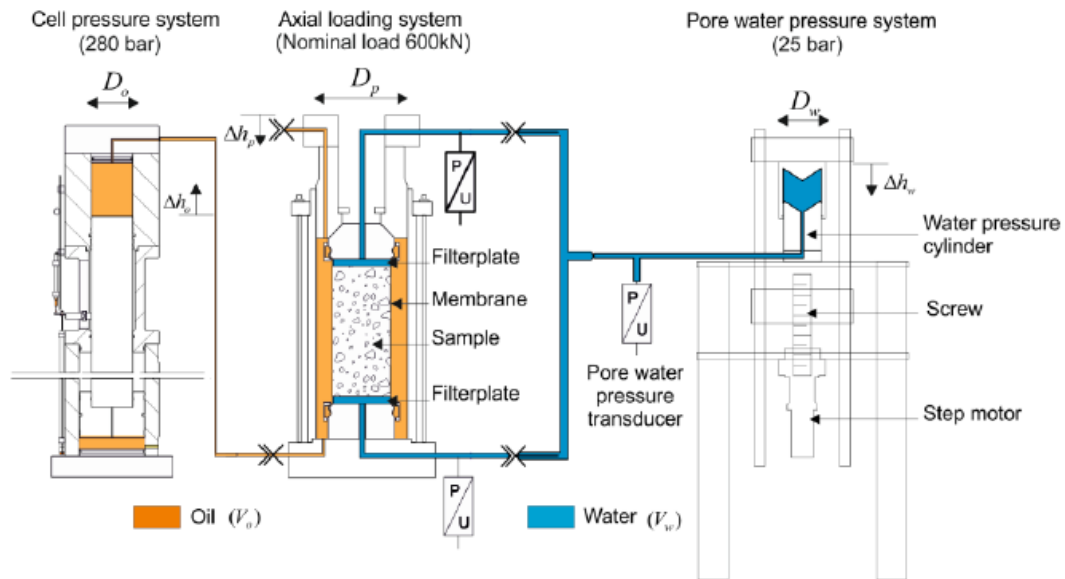


Figure 22. Layout of the testing equipment at ETH Zurich (Dong 2017)

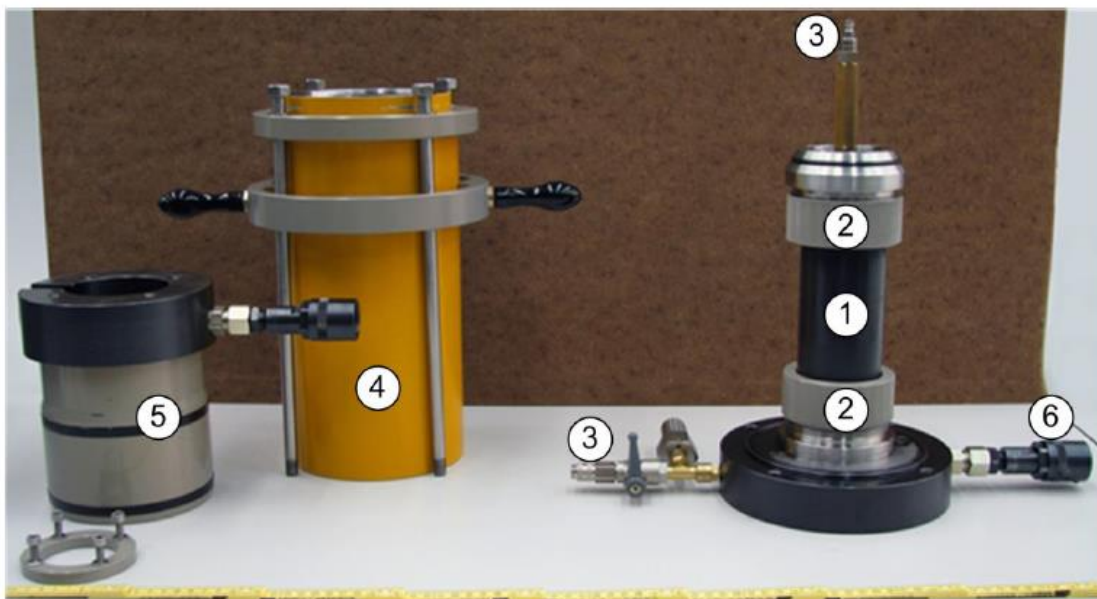


Figure 23. Disassembled triaxial cell (Dong 2017)

1. Specimen with membrane
2. Adapters
3. Water connectors
4. Cell wall
5. Load piston
6. Oil connector

4 Literature Research

The triaxial testing procedures are established by the International Society of Rock Mechanics (ISRM) or other standards like American Society for Testing and Materials. The triaxial tests procedures for typical soil and rock are well established but there are significant challenges when conducting triaxial tests for weak or soft rock as discussed in chapter 3. This is because weak rocks fall in the category of R₀ to R₂ in the ISRM class (Stacey and Martin 2018), they cannot be either categorized as soil or rock since they have intermediate strength between soils and rocks. The strength and behaviour of weak rocks depends on several factors as discussed in section: 2.3, so when conducting a triaxial test on weak rocks, these factors should be taken into account.

During the literature research it was found out that various scholars have applied different testing procedures for low permeable geomaterials and concepts from soil mechanics testing are applied to establish an appropriate triaxial testing method.

Most of the researchers performed the triaxial test for low permeable geomaterials which were comprised of three steps:

- Saturation Stage
- Consolidation Stage
- Shearing Stage

For instance the study of (Islam and Skalle 2013), on the testing of shale revealed that the procedure adopted was executed in three steps:

1. Loading to a predetermined level of pore pressure and confining pressure
2. Consolidation stage
3. Axial loading under a constant axial strain/displacement rate

In this testing procedure, saturation of the specimen was not confirmed as it was achieved through the consolidation of the specimen. It is important to fully saturate the specimen prior to testing. According to (Belmokhtar et al. 2018), partial saturation may lead to biased result of mechanical properties of tested specimen, overestimating the shear strength with partially saturated specimens, and overestimating the young's modulus by running the partially drained tests.

In some studies (Wu et al. 1997; Dong et al. 2013; Wild et al. 2017), the saturation of the specimen has been confirmed by measuring Skempton's pore pressure coefficient (B). A specimen is assumed to be saturated when the B -value was higher than a certain value or constant for subsequent measurements (Wild et al. 2017). Based on the assumption, usually made in soil mechanics (Bishop and Wesley 1975), it is assumed that undrained triaxial compression tests on fully water saturated soil specimens - give almost the same

effective stress path and thereby almost the same stress-strain properties independent of the total stress path followed cannot be true for undrained tests on rocks. In case of soils an equivalent pore pressure is measured when a cell pressure increment is applied which means that the sample is fully saturated ($B=1$). While weak rocks can be fully saturated even when the back pressure is high (Roman Makhnenko and Joseph Labuz 2013) that means B is significantly smaller than 1. Also, in addition to the effective stress the pore pressure contributes somewhat to produce strain (poroelastic response). For such weak rocks, the B value could be as low as 0.5 (Stacey and Martin 2018). (Bésuelle et al. 2014) considered a specimen to be saturated when the fluxes of water stabilized (not zero) or the pore pressure at the outlet and inlet.

In the testing procedure for the triaxial testing of soft/weak rocks by (Chiu et al. 1983), the saturation was done in two steps; first the specimen is placed under water in desiccator and a full vacuum is applied. To achieve complete saturation of the specimen, the vacuum saturation is followed by applying back pressure to fully saturate the specimen. Also (Chiu et al. 1983) conducted 45 constant strain rate tests on Melbourne mudstone to establish testing strain rates which would not influence the parameters to be determined.

After the saturation of the specimen, it is consolidated to desired values of effective stresses. The consolidation stage is then followed by shearing stage.

4.1 Appropriate Technique for Triaxial Testing of Soft Rock

4.1.1 Sample Extraction, Storage and Preparation

The sampling, the storage and the specimen preparation should be according to standard guidelines. These standards are given by ISO 22475-1:2006 Geotechnical investigation and testing -- Sampling methods and groundwater measurements -- Part 1: Technical principles for execution. The ASTM designation: D2113-99 is a standard practice for the rock core drilling and sampling of rock for site investigation. The standard practices for preserving and transporting rock samples is a part of ASTM designation D 5079-02.

After retrieval of the cores from the ground, it is important that the water content is preserved. The cores should be covered and sealed immediately after core extraction, placed usually in wooden boxes and should be stored at constant humidity and temperature. The specimen preparation procedure can be optimized and reduced to avoid desaturation because it is often followed by desiccation cracks (Wild 2016).

Diamond band saw which is operated with pressurized air instead of water cooling, can be used for smooth and precise cutting of the sample into specimens. The feed rate is manually

controlled.

When the specimen is prepared, it is measured, weighted and photographed. A rubber jacket of 2 mm thick is put over the specimen. Strips and circular filter papers are placed on the specimen top and bottom face while on the side of the specimen strips can be placed.

4.1.2 Water Content and Saturation

The dry density, porosity and saturation can be determined according to the ISRM suggested methods and ASTM designation: D2216-99 standard test method for laboratory determination of water content of soil and rock. For the water content determination, the mass of water contained in a rock sample is measured as a percentage of the oven dry sample mass. The water content is determined with respect to the weight of the specimens after 24h drying at 105°C. The water content is measured right after the drilling and compared with the water content of the specimen after preparation, which can be used as an indicator of the saturation state of the specimen as well as to quantify any severe desaturation taken place during specimen preparation. (Wild 2016).

$$\text{Water content } w = \frac{\text{Pore water mass } M_p}{\text{Grain mass } M_s} * 100\%$$

A study by (Chiu et al. 1983) reveals that the UCS is generally decreased with the increase of water content. The water content after saturation and consolidation is estimated using the weight of the specimen after saturation/consolidation (calculated by adding and subtracting the measured amount of water flown into or out of the specimen during saturation phase and consolidation phase, respectively, to the initial weight before testing). For the porosity calculation, the grain density should be in an appropriate range because the estimated values of saturation depend upon this parameter.

4.1.3 Specimen Geometry

The ISRM suggested test methods recommend length to diameter ratio (L/D) ratio of 2.0 to 3.0. Furthermore results from the research of (Chiu et al. 1983) shows that L/D ratio should atleast be 2.0.

4.1.4 Rate of Strain Application

The mechanical behaviour of both soils and rocks depends on the rate of strain. For rocks some of the researchers have shown that the uniaxial compressive strength increases with the increase of strain rates (Chiu et al. 1983). The strains rate can have significant influence on induced pore water pressure.

In the past decade researchers investigated the effect of rate of strain application. (Chiu et

al. 1983) investigated Melbourne mudstone to establish strain rate and conducted 45 constant strain rate tests. It was found out that a strain rate less or equal to $3.3 \times 10^{-7} \text{ s}^{-1}$ is appropriate for complete dissipation and equalization of pore pressure. (Wu et al. 1997) tested Pierre II shale with lateral drainage made of fiberglass side drains placed around the specimen. A strain rate of $3 \times 10^{-8} \text{ s}^{-1}$ was found out to be sufficient for undrained test. Similarly experiments by other researchers (Deng et al. 2011; Hu et al. 2014; Islam and Skalle 2013; Dong et al. 2013; Wild et al. 2017; Belmokhtar et al. 2018) revealed that a loading/strain rate in the range of 10^{-7} s^{-1} , 10^{-8} s^{-1} and 10^{-6} s^{-1} , 10^{-7} s^{-1} , 10^{-8} s^{-1} was sufficient during drained and undrained loading respectively.

4.2 Triaxial Test Procedure of Soft Rock

As discussed above, a multi-stage testing procedure is required for the triaxial testing of soft or weak rocks. The three stages i.e., saturation stage, consolidation stage and shearing stage, are challenging because of low permeability and long duration of the test. Furthermore, due to the long duration of the test, the temperature variation can have significant influence on the results obtained (Dong et al. 2013).

The important features and procedure, which is a key to the implementation of triaxial testing of weak or soft rock are explained and discussed below.

4.2.1 Saturation Stage

The possibility of loss of water and saturation during the storage and specimen preparation can be minimized but not completely avoided. In order to deal with this unavoidable desaturation and to reconstitute the in situ condition as far possible, the specimens are re-saturated with the help of back pressure.

Two different methods can be used to achieve full saturation of the specimen:

- The skeleton of the rock specimen is compressed without any change in water or total air content (undrained compression)
- Back pressure is built up and water is allowed to enter the pore space of the specimen (drained conditions)

In both cases the pore pressure will increase and therefore more air will be dissolved in the pore water. The application of back pressure means that the effective stresses are kept constant by increasing the cell pressure and pore pressure at the same time. For the complete saturation of the specimen, watering/flushing phase is used to achieve saturation of the pore pressure lines. To avoid additional gas/bubbles in the pressure lines and pores it is necessary to use de-aired water. Furthermore, the use of pore water with a composition similar to the in-situ pore water is recommended since weak/soft rocks such as clay shales

are prone to chemical reactions that may alter the geomechanical properties (Wild 2016). At the inlet and outlet small pressure gradient is applied by leaving the exit valve open so that the gas could escape from the pore space and the water permeates the specimen. The confining stress is also increased in such a way that it exceeds the pore pressure within the specimen so as that to avoid damage and swelling of the specimen. Figure 24 shows increase in pressure during the flushing phase.

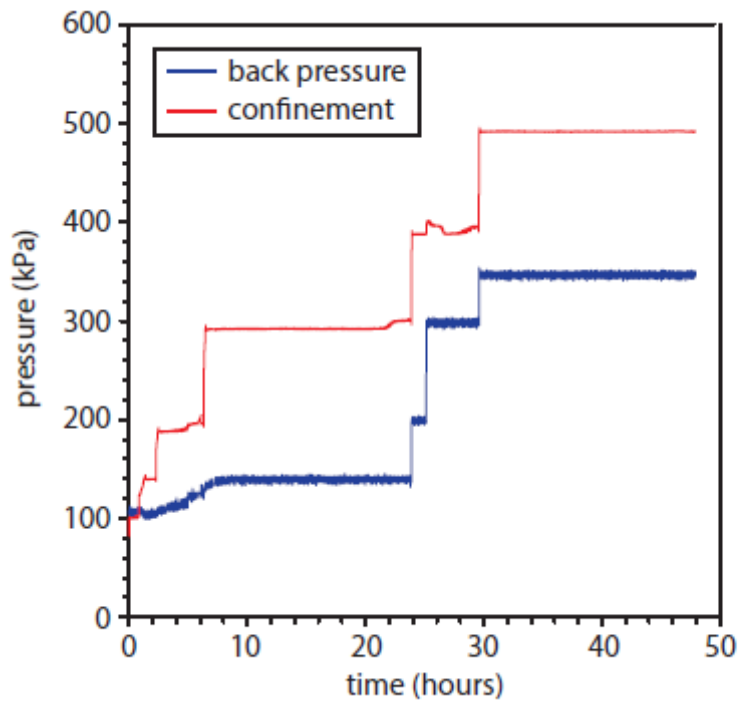


Figure 24. Pressure increase during the flushing phase (Wild 2016)

After the watering/flushing phase, the back pressure is increased at both ends of the specimen in order to further increase the degree of saturation. To control the degree of saturation of the specimen, the back pressure is increased in several stages and is maintained for hours to days. Before each back pressure increase, Skempton's B is checked in order to know the saturation of the specimen.

$$B = \frac{\Delta u}{\Delta \sigma_3} \quad (4.1)$$

Where Δu and $\Delta \sigma_3$ are change in pore pressure and confining stress respectively.

$$B = \frac{1}{1+n \frac{C_w}{C_s}} \quad (4.2)$$

Where n , C_w and C_s denote porosity, the compressibility of the water and the compressibility of the specimen skeleton respectively.

For a saturated specimen the B value is equal to 1 if the compressibility of the sample skeleton is much higher than the compressibility of the pore water. In case of soils this condition is fulfilled as soil skeleton has a compressibility higher than that of water. For rock specimens the B value may be considerably lower than 1 because the compressibility of the rock skeleton is as low as that of the pore water. The B values of saturated granite and sandstone specimens have been found to be lower than 0.5 (Lambe and Whitman 1969). When saturation is not fully achieved, the measured value of B increases with increasing back pressure. This is due to the increasing degree of saturation and decreasing compressibility of the pore fluid as the back pressure is increased. For low permeable rocks, B will stay constant for two subsequent undrained confining stress changes if the specimen is completely saturated (Wissa 1969). (V. Aristorenas 1992) noted that it is almost impossible to get same Skempton's coefficient B from subsequent B -checks. Therefore, he came to the assumption that the specimen is said to be saturated if the B values are sufficiently high and do not change significantly by more than ± 0.03 for two subsequent B checks. Figure 25 and figure 26 show saturation phase with back pressure steps with B checks after each increment of back pressure and values of Skempton's pore pressure coefficient are obtained for individual B -check during saturation phase respectively.

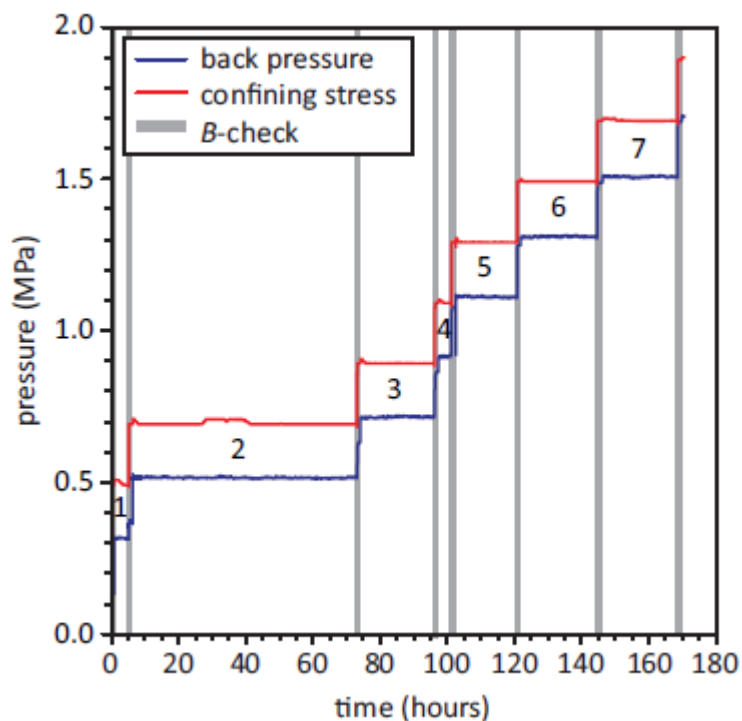


Figure 25. Saturation phase with back pressure stages (Wild 2016)

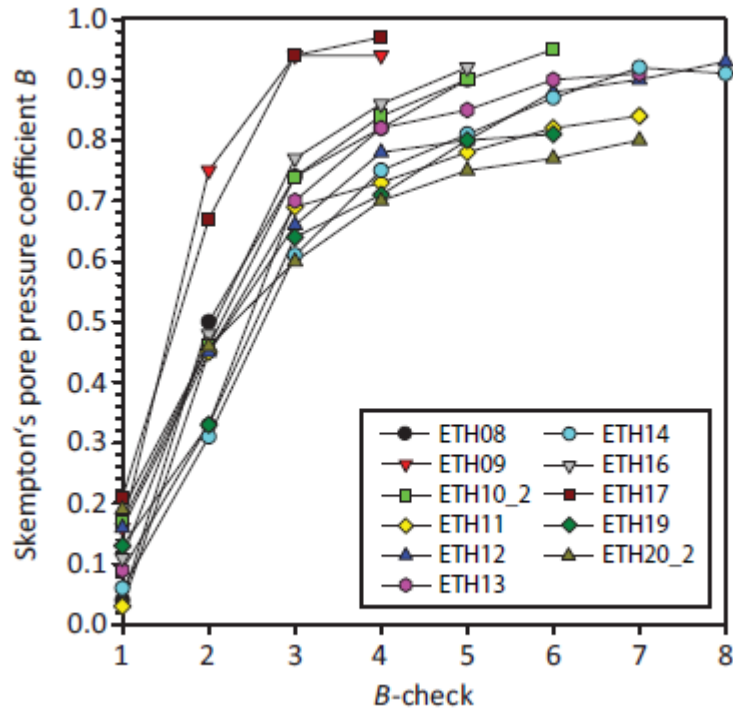


Figure 26. Values of Skempton's pore pressure coefficient obtained for individual B-check during saturation phase (Wild et al. 2017)

4.2.2 Consolidation Stage

After the saturation of the specimen through the application of back pressure, the specimen is consolidated to specified values of effective stresses, which can be taken equal to the mean effective stresses in-situ. The pore pressure valves on one or on both sides are opened. A back pressure is maintained which is equal or higher than the back pressure applied during the saturation phase and the specimen is consolidated.

The theoretical time t prior to the testing can be estimated by 1-D consolidation theory by Terzaghi using equation (4.3):

$$t = \frac{T_v D^2}{C_v} \quad (4.3)$$

Where, T_v is dimensionless time factor, D^2 is the drainage path and C_v is the coefficient of consolidation. Furthermore, the coefficient of consolidation can be defined by the following relation (4.4):

$$C_v = \frac{k E_{oed}}{\gamma_w} \quad (4.4)$$

Where, k is the hydraulic conductivity, E_{oed} is the oedometer modulus and γ_w is the unit

weight of water.

In a triaxial cell, the coefficient of consolidation for an isotropic consolidation and the coefficient of consolidation from the relation (4.4) is not the same (Head 1998). The relation to derive the coefficient of isotropic consolidation is given by (Head 1998), which is as followed:

$$\frac{K}{E_{oed}} = \frac{(1+\nu)}{3(1-\nu)} \quad (4.5)$$

The assumption for equation (4.5) is that the material is isotropic with linear elastic behavior while K is bulk modulus, E_{oed} is oedometer modulus and ν is poisson's ratio. The coefficient of isotropic consolidation can also be estimated from time settlement curves.

In practice the consolidation degree is usually controlled by examining time-dependent variations in volumetric strain. Furthermore, the change in water content due to excess pore pressure dissipation can be analyzed. The consolidation of the specimen is considered sufficient when the strain approaches a constant value and the water content remains constant (Wild et al. 2017). This is shown in figure 27.

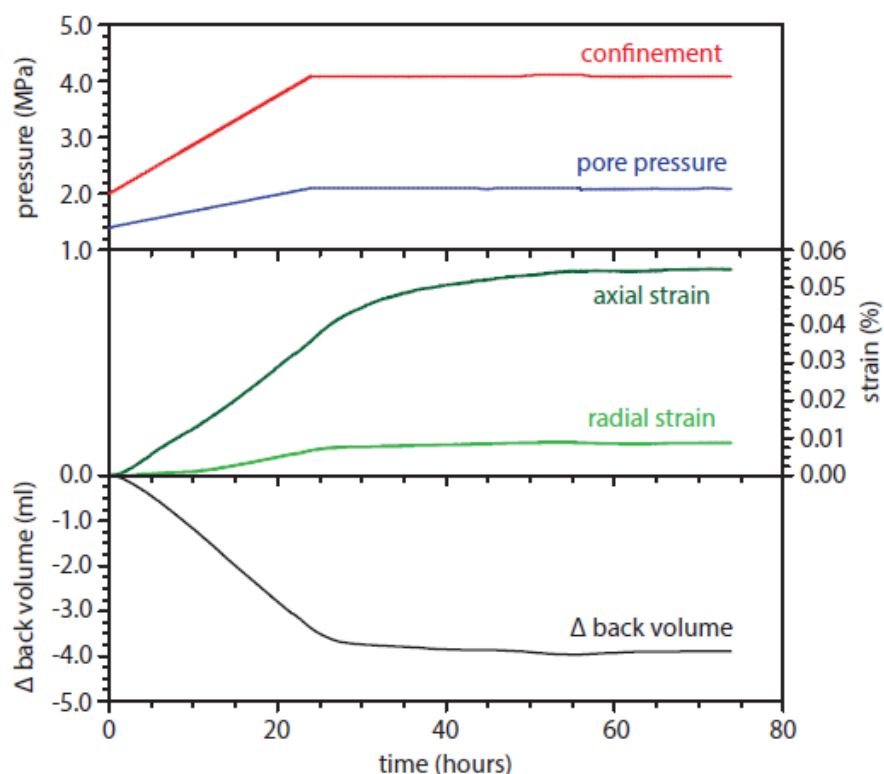


Figure 27. Example of consolidation stage showing the increase in confinement and pore pressure, local axial and radial strains and the back volume. The change in back volume indicates the change in water content (Wild et al. 2017)

Type	Sampling Method	L/D ratio	Loading/Strain rate (s ⁻¹)	W.C (%)	Sr (%)	Back Pressure (MPa)	C (MPa) φ (°)	Standard	Researcher
Melbourne Mudstone	Core drilling	2:1	3.3*10 ⁻⁷	13	96.5	0.7	-	ISRM	Chiu (1983)
Pierre II Shale	-	2:1	3*10 ⁻⁸ (CU)	22.7	-	5	1.1 14 °	ISRM	Wu (1997)
Boom Clay	Borehole drilling	2:1	2*10 ⁻⁷ (CD)	26.5	-	-	0.1-0.2 12-14°	AFNOR (2005b)	Deng (2012)
COx claystone	-	1:1	10 ⁻⁷ (CD/CU)	6.2 ± 1.2	100	1 - 1.5	3.94 21.43 °	-	Hu (2013)
Pierre I Shale	Outcrop block	2:1	2*10 ⁻⁸ (CD) 1-2*10 ⁻⁷ (CU)	9.9	-	-	-	ISRM	Islam (2013)
Breccia	Wire line system with double tube sampler	2:1	10 ⁻⁷ (CD/CU)	17	95	0.1-0.4 (watering phase) 2 (saturation stage)	0.59 10.7° (L.Zone) 0.18 22.9° (U.Zone)	ISRM	Dong (2013)
Opalinus Clay	Triple tube core barrel technique	2:1	10 ⁻⁶ to 10 ⁻⁷ (CU) 10 ⁻⁸ (CD)	7.5	98.6	0.11 - 0.35 (Flushing phase) 0.5-1.6 (saturation stage)	1 35.3 °	ISRM	Wild (2017)
COx claystone	-	2:1	6.6*10 ⁻⁸ (CD)	5.97	87.9	4	-	-	Belmokhtar (2018)

Table 3. Triaxial tests on different types of weak rocks

4.2.3 Shearing Stage

The shearing may be either drained (open drainage valve) or undrained (closed drainage valve). In consolidated drained test, the pore pressure is kept constant means the drainage is allowed while in consolidated undrained test the drainage is not allowed which means that the volume of the specimens remains constant and changes in pore pressure are measured. The shearing rate has to be slow enough during drained shearing (CD test) so that no excess pore pressure is generated, especially at a peak strength. Therefore, the loading/strain rate is selected in such a way that the heterogeneous pore pressure distribution is avoided and valid drained properties are obtained. In case of undrained shearing, the loading/strain rate is adopted in such a manner that throughout the test reliable pore pressure measurement is taken.

The time required to reach the peak strength can be estimated based on the theory of consolidation and can further be used to determine an appropriate loading/strain rate when the load or strain at failure is known (Bishop & Henkel 1962). These are usually not known in advance, which adds an uncertainty to the estimation of the appropriate loading/strain rate - in addition to the uncertainties in estimating the time that is required to dissipate any excess pore pressure or equalize non uniform pore pressure within the specimen (Wild et al. 2017).

From the literature research it was found out that acquiring an appropriate loading/strain rate is to conduct triaxial tests which utilizes rates in the range of 10^{-6} s^{-1} , 10^{-7} s^{-1} , 10^{-8} s^{-1} for CU tests. The material characteristics of the different specimens, used for evaluation, should be comparable. The loading/strain rate is adequate for CU tests on similar test material and with similar specimen dimensions if the pore pressure response does not change between two tests that utilize different loading rates (Wild 2016). Table 3 shows triaxial tests on different weak material and the corresponding features and characteristics of the tests are also given.

The adequacy of the loading/strain rate for consolidated undrained test (CU test) can be checked by measuring the product of Skempton's pore pressure coefficients AB . According to (Skempton 1954), the change in pore pressure (Δu) can be determined as follow:

$$\Delta u = B(\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)) \quad 4.6$$

The above equation 4.6 can be reduced to equation 4.7, as in standard triaxial test there is no change in confining stress ($\Delta\sigma_3 = 0$) and changes occur in axial stress ($\Delta\sigma_1 = \Delta\sigma_{ax}$).

$$\Delta u = AB(\Delta\sigma_{ax}) \quad 4.7$$

- $A = 1/3$ for an isotropic, perfectly elastic material behavior (Skempton 1954)

- For overconsolidated clays the value of A is between 0.25 and 0.5 (Skempton and Bjerrum 1957)

The value of B can be taken when the saturation stage is completed. The value of B can also be calculated as follows:

$$B = \frac{1}{1+n\frac{C_f}{C_d}} \quad 4.8$$

Where n , C_f and C_d denotes porosity, the compressibility of the fluid and the compressibility of the specimen skeleton respectively.

The theoretical values of AB can be used as an indicator or criteria for the appropriate loading/strain rate for consolidated undrained (CU test). The loading/strain rate is considered to be too fast when the AB is significantly lower than the theoretical value. With such fast loading/strain rate the actual pore pressure response of the specimen cannot be captured. It is important to keep in mind that this criterion can only be used if the saturation phase and the consolidation phase are complete. Incomplete saturation or non-uniform distribution of pore pressure during shearing would reduce the AB -value substantially (Wild et al. 2017).

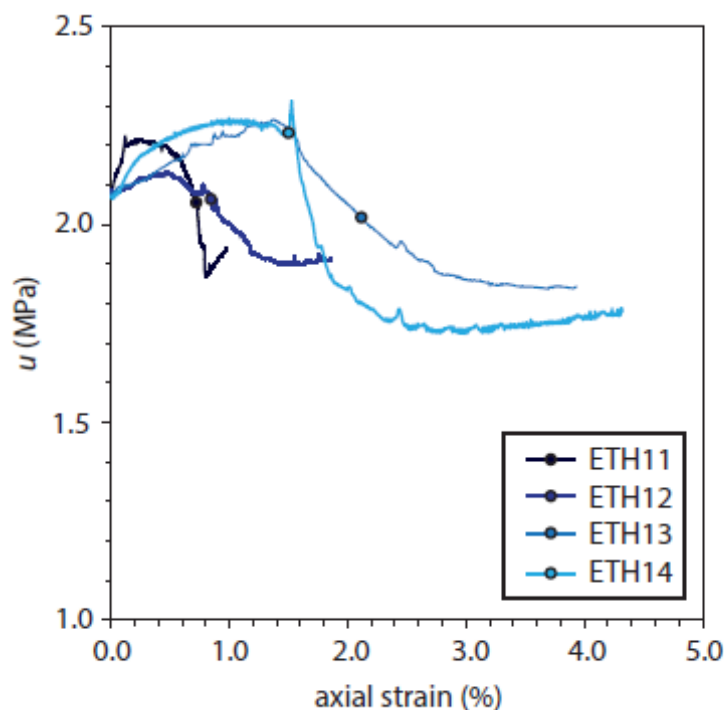


Figure 28. Pore pressure evolution during CD tests on opalinus clay (Wild 2016)

In case of consolidated drained test (CD test), the pore pressure should be insignificant especially at the peak strength. Throughout the test the evolution of pore pressure should

be minimized. Figure 28 shows the pore pressure evolution during the consolidated drained test (CD test) performed by (Wild 2016) on opalinus clay. The pore water pressure which is measured during the test (initial pore pressure u_0) can be used as a data based criteria for the suitable loading/strain rate. The ratio between pore pressure at peak (u_p) and the initial pore pressure (u_0) should be close or equal to one. The excess pore pressure developed and the peak strength value will not be the representative for drained condition, if the ratio (u_p/u_0) deviates significantly from 1 (Wild et al. 2017). Figure 29 shows the comparison of initial pore pressure (u_0) and pore pressure at peak (u_p) as function of confining stress.

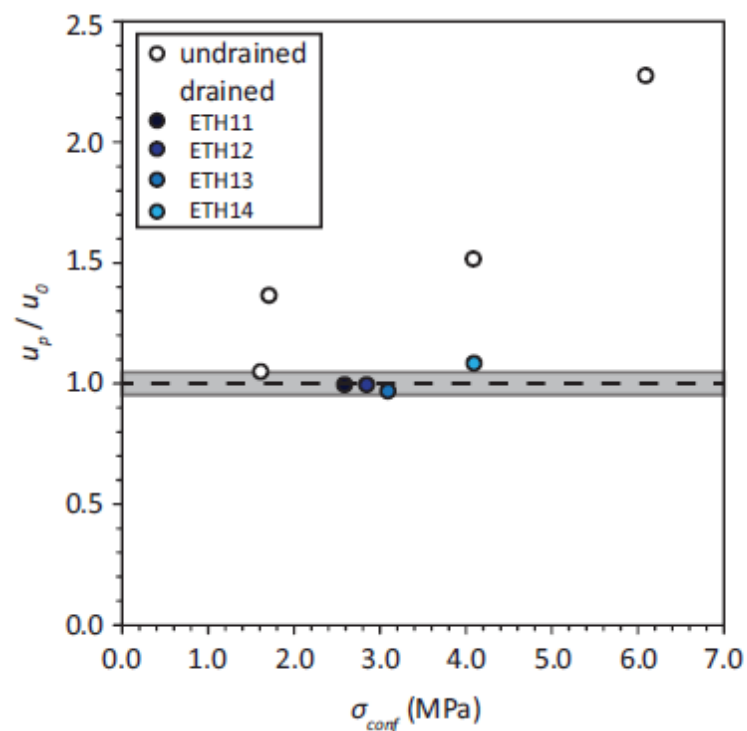


Figure 29. Pore pressure at peak u_p compared to initial pore pressure u_0 as a function of confining stress. The grey area represents the 5 % interval around $u_p/u_0 = 1$ (Wild 2016)

In figure 30, the effective stress paths of the opalinus clay specimen during the consolidated undrained tests (CU test) are illustrated. It can be seen that at low effective stresses (over-consolidated specimens), the effective stress path turns to the right when the deviatoric/differential stresses are increased. This shows dilation and decrease of the pore pressure, which results in the increase in mean effective stresses. While the specimen which are consolidated at higher effective stresses (normally consolidated specimens), the effective stress path turns to the left. This shows contraction and the specimen will contract even after peak strength is reached before they start to dilate in the post-peak region.

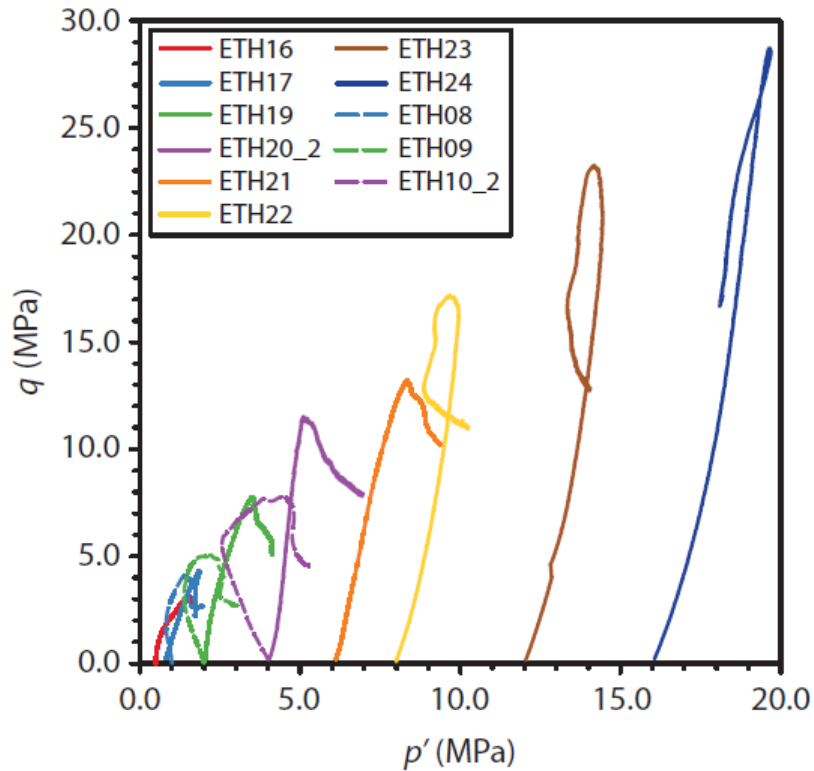


Figure 30. Effective stress paths during Consolidated Undrained Tests (CU) (Wild 2016)

4.3 Assessment of Results

The effective strength parameters of opalinus clay obtained by (Wild 2016) are shown in figure 31 and all the linear regressions show a coefficient R^2 of 0.93 or 0.98. The solid black line in figure 30 is fitted with a linear Mohr-Coulomb failure envelope which results in an effective friction angle of 35.3° and an effective cohesion of 1.0 MPa. The blue dashed line represents a bi-linear Mohr-Coulomb failure envelope which shows an effective cohesion of 0.20 MPa and a higher friction angle of 47.1° for specimen consolidated at low effective stresses (less than 5-8 MPa, i.e. over consolidated specimens) and for the specimen consolidated at higher confinement (greater than 5-8 MPa). It shows a lower friction angle of 31.9° and an effective cohesion of 2.01 MPa. The grey line shows Mohr-Coulomb fit of the ultimate strength and gives an effective cohesion of 0.49 MPa and an effective friction angle of 22.5° .

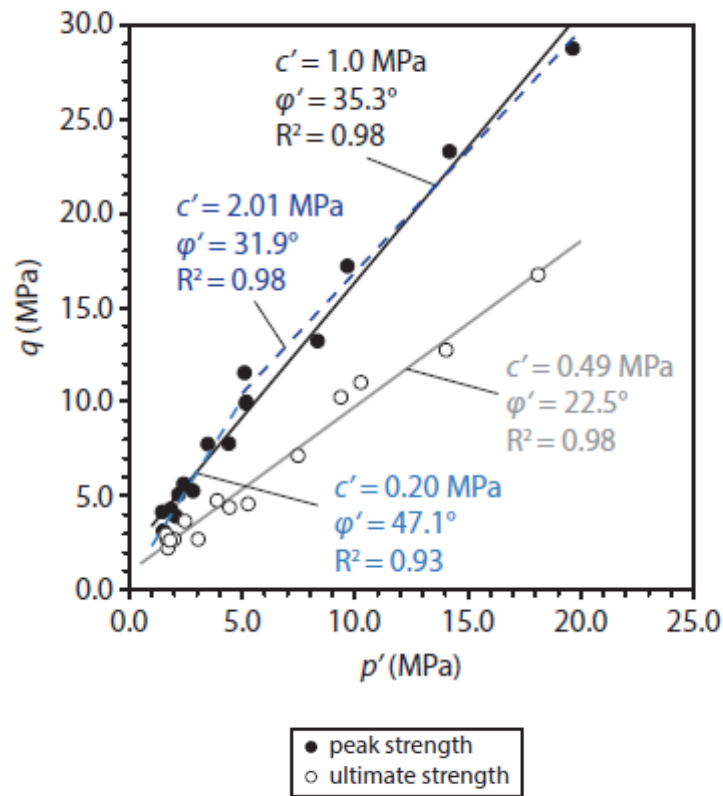


Figure 31. Possible peak strength failure envelopes for all specimens of opalinus clay tested (Wild 2016)

(Favero et al. 2018) tested opalinus clay specimens both parallel to bedding (P-specimen) and perpendicular to bedding (S-specimen). The peak and ultimate shear strength results are plotted in terms of deviatoric stress (q) and mean effective stress (p'), as shown in figure 32. It can be seen in the below plot that the peak strength is always higher than the ultimate strength. This is even true when the material is approaching the normally consolidated conditions. It is because rock-like geomaterials show strain-softening behaviour following the quasi-brittle failure mode. The Mohr-Coulomb parameters in case of peak strength are 22° (an effective friction angle) and 2.6 MPa (an effective cohesion), while 18° (an effective friction angle) and 0.8 MPa (an effective cohesion) are for ultimate strength.

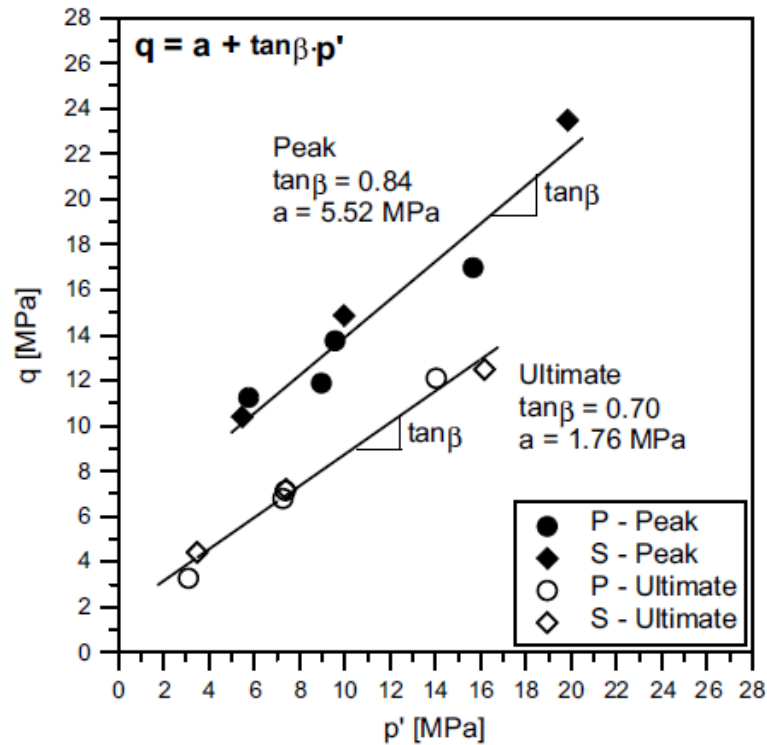


Figure 32. Ultimate ($c' = 0.8 \text{ MPa}$, $\phi' = 18^\circ$) and peak shear strength ($c' = 2.6 \text{ MPa}$, $\phi' = 22^\circ$) failure envelopes for all specimens tested (Favero et al. 2018)

The results obtained by (Favero et al. 2018) are compared with the data of (Wild 2016) opalinus clay and also with callovo-oxfordian clay (Belmokhtar et al. 2018; Menaceur et al. 2015; Hu et al. 2014) as shown in figure 33. It can be seen, in the plot listed below, that they show a similar response in terms of shear strength as both geological formation resembles in terms of mineralogy, microstructure and porosity.

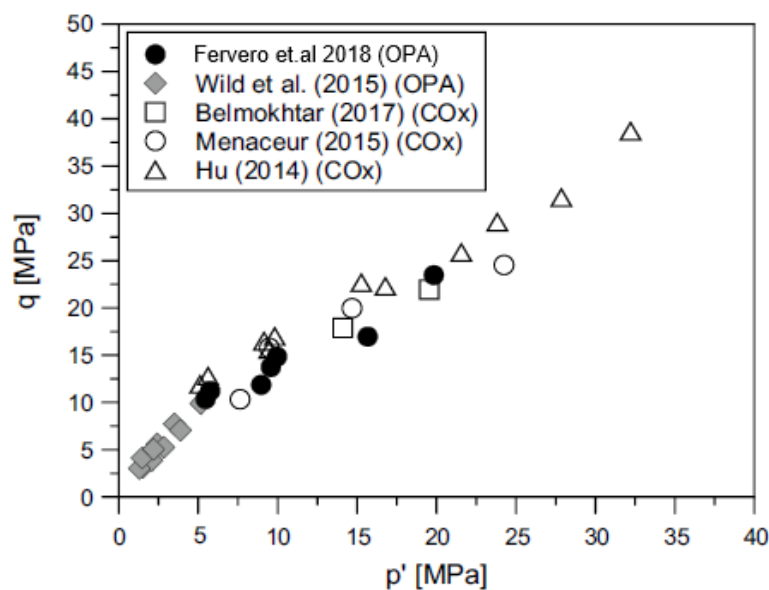


Figure 33. Comparison of the peak strength results with further data on Opalinus Clay (OPA) and Callovo-Oxfordian claystone (COx) (modified after (Favero et al. 2018))

(Wu et al. 1997) conducted a total of six consolidated undrained triaxial test on Pierre II Shale. He described the peak strength behaviour of Pierre II Shale using Mohr-Coulomb criterion, as shown in figure 34. The value of cohesion and friction angle obtained are 1.1 MPa and 14°.

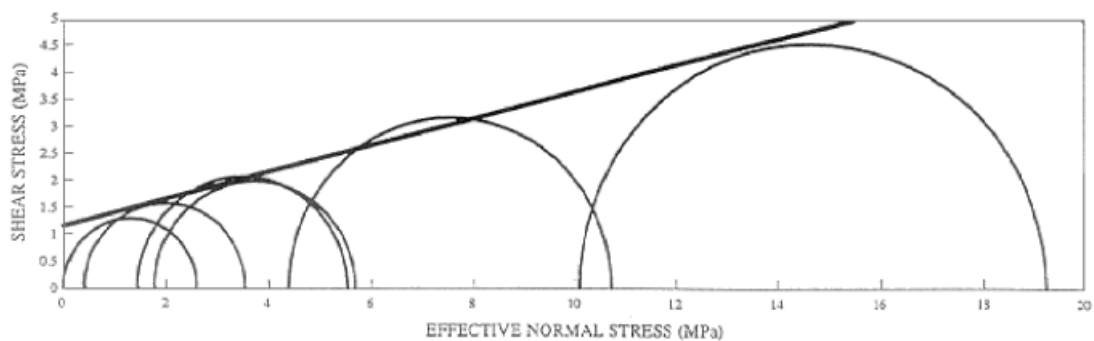


Figure 34. Peak strength of Pierre II Shale (Wu et al. 1997)

5 Summary and Conclusion

The triaxial test procedures are well established and documented for both soils and rocks but in case of weak/soft rocks it is not well documented. In the past two decades, researchers have put efforts to establish a triaxial testing procedure to determine reliable properties of weak/soft rocks. In the literature study, the triaxial tests on different soft rocks such as clay shales, claystone, breccia, mudstone, opalinus clay were studied and examined. The important aspects and features that are key to the implementation of the triaxial test procedures are given below:

1. The cores used for the testing of soft or weak rocks should be undisturbed because the quality of specimen is influenced by the drilling, sampling, transport, storage and specimen preparation.
2. The water content should be preserved after retrieval of the cores from the ground.
3. The sampling, the storage and the specimen preparation should be according to standard guidelines. These standards are given by ISO 22475-1:2006 Geotechnical investigation and testing -- Sampling methods and groundwater measurements - Part 1: Technical principles for execution. The ASTM designation: D2113-99 is a standard practice for the rock core drilling and sampling of rock for site investigation. The standard practices for preserving and transporting rock samples is a part of ASTM designation D 5079-02.
4. The water content of samples is measured right after the drilling and is compared with the water content of the specimen after preparation, which can be used as an indicator of the saturation state of the specimen.
5. The factors, which affect the properties of soft rock such as specimen geometry, rate of loading/straining, water content, saturation, porosity, dry density, weathering, temperature should also be taken into account when executing a triaxial test procedure on soft rocks.
6. The ISRM recommends L/D ratios of between 2.0 to 3.0. The experiments suggest that the L/D ratio of atleast 2.0 should be adopted.
7. The possibility of water and saturation loss during sampling, storage and the preparation of specimen cannot be entirely excluded. Therefore, in order to restore the in-situ conditions as far as possible, the specimen is re-saturated by the application of back pressure. The back pressure is applied in several steps.
8. The back pressure allows the entrapped gas from pore space and hence increases the degree of saturation.
9. The value of Skempton's coefficient B can be used to indicate the degree of saturation of the specimen.

10. The specimen is considered to be saturated if the Skempton's coefficient B does not change significantly for two successive steps (change in B (ΔB) smaller than or equal to about 0.03).
11. When saturation is not fully achieved, the measured value of B increases with the increasing back pressure. This is due to the increasing degree of saturation and the decreasing compressibility of the pore fluid as soon as the back pressure is increased.
12. For soils the B value for saturated specimen is practically equal to 1. But in case of rocks, the B value can be significantly lower than 1 because the compressibility of rock skeleton is as low as that of water.
13. Particular attention should be paid to saturation phase to avoid any possible damage.
14. After the saturation stage, the specimen is consolidated to the specified values of effective stresses, which can be taken equal to the mean effective stresses in-situ.
15. The consolidation of the specimen is considered adequate when the strain and the change in water volume remains constant.
16. The consolidation stage is then followed by the shearing stage, in which the specimen is sheared under drained or undrained conditions.
17. The shearing rate has to be slow enough during drained shearing (CD test) so that no excess pore pressure is generated, especially at the peak strength. Therefore, the loading/strain rate is selected in such a way that the heterogeneous pore pressure distribution is avoided and valid drained properties are obtained.
18. For CD test, the pore water pressure which is measured during the test (initial pore pressure u_o) can be used as a data based criteria for the suitable loading/strain rate. The ratio between pore pressure at peak (u_p) and the initial pore pressure (u_o) should be close or equal to one.
19. The excess developed pore pressure and the peak strength value will not be the representative for drained condition if the ratio (u_p/ u_o) deviates significantly from 1.
20. In case of undrained shearing, the loading/strain rate is adopted in such a manner that throughout the test reliable pore pressure measurement is taken.
21. It was found out during the literature research that the appropriate loading/strain rate for consolidated undrained test (CU test) can be derived by conducting several triaxial tests using loading/strain rate in a range of 10^{-6} s^{-1} , 10^{-7} s^{-1} , 10^{-8} s^{-1} .
22. The adequacy of the loading/strain rate for consolidated undrained test (CU test) can be checked by measuring the product of Skempton's pore pressure coefficients AB .

23. The loading/strain rate is considered to be too fast when the AB is significantly lower than the theoretical value. With such fast loading/strain rate the actual pore pressure response of the specimen cannot be captured. It is important to keep in mind that this criterion can only be used if the saturation phase and the consolidation phase are complete.

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