CRITICAL STATE LINE OF SOILS: GEOTECHNICAL ENGINEERING RESEARCH

By: Kristabelle A. Mendoza
Department of Civil and Environmental Engineering

Professors of Record:
Emir J. Macari, Ph.D Professor and Dean of Engineering
Barbara Schneider-Muntau, UIBK Professor of Geotechnical and Tunnel Construction

Date Submitted: 15 October 2017

In partnership with the Austrian Marshall Plan Foundation, Vienna, AUSTRIA and the University of Innsbruck, AUSTRIA

Critical State Line of Soils

Abstract

The contents of this research report are focused on the full examination of the Critical State Line of Soils. By beginning with the identification of the critical state problem, the report leads to a step-by-step process of the laboratory activities performed by the student. Each procedure is not only explained in theoretical and conceptual forms, but also detailed descriptively as experienced in the geotechnical laboratory environment. In order to assess the contents in this report, the reader must have minimal or prior scientific or engineering knowledge to interpret the graphs, figures, and equations. Further in the conclusion of the report, the student felt it was necessary to include the challenges encountered during the assembly of experience, and, nevertheless, providing detailed solutions to minimize them. With versified help from university staff, faculty, professors, and colleagues, the student was able to fully grasp the Critical State concept and present it to the public at the University of New Orleans via a PowerPoint presentation.

Table of Contents

TITLE PAGE 1

ABSTRACT 2

TABLE OF CONTENTS 3

SOIL MECHANICS BACKGROUND AND DEFINITION OF SHEAR STRENGTH 4-5

DEFINITION OF CRITICAL STATE CONCEPT 6-10

BASICS OF TRIAXIAL TEST EXPERIMENTS 10-15

PHOTOGRAPHS FROM UIBK GEOTECHNICAL ENGINEERING LAB 15-28

ASSEMBLING THE TRIAXIAL TEST SPECIMEN 29

DETAILED TRIAXIAL TEST PROCEDURE 29-33

I. CHECK MEASURED VALUES 29-30

II. SATURATION 30-31

III. B-TEST 31

IV. CONSOLIDATION 31-32

V. SHEAR 32-33

TO DISASSEMBLE THE TRIAXIAL CELL 34-37

DATA OBTAINED AND ANALYSIS 34-37

I. GRAPH 1 34

II. GRAPH 2 35

III. GRAPH 3 36

IV. GRAPH 4 37

CHALLENGES ENCOUNTERED AND SOLUTIONS 38-39

CONCLUSION 39-40

ACKNOWLEDGEMENT 40

APPENDIX 41-49

REFERENCES 50

Soil Mechanics Background and Definition of Shear Strength

A common parameter measured in soil mechanics is the shear strength of soil. Shear strength is simply defined as the magnitude of the shear stress that a soil can sustain. In the engineering field, the value of the soil's shear strength is needed for the development of foundation engineering projects. To acquire the shear strength of a soil, specific parameters are measured distinctly depending on the soil composition. For this project, the critical state line of sand material will be analyzed, therefore measurements will be focused on granular soils. In granular soils, shear strength depends on the following factors: the stress history, water content, degree of saturation, void ratio, drainage conditions, isotropic media in the soil, rate of loading, etc. These factors are defined and elaborated in detail as following:

- The stress history of the soil is determined based on its consolidation history. There are two classifications of the soil history: normally consolidated soil (NCS) and over consolidated soil (OCS). The sand in this project is remolded (subjected to different bulk densities*) into loose or dense samples. Based on the density of the sand and its expected behavior, we can qualify the material as NCS or OCS. The behavior expected from the NCS and the OCS samples will be confirmed by the graphs and compared to the theoretical behavior (see **Figure 5**). Normally consolidated soil is qualitatively defined as the maximum stress that the soil is experiencing at its in-situ state*. Once loading is either added or removed at a specific point on the soil, the soil is defined to be over-consolidated.
- The water content is a principal parameter measured in the soil sample, since the density of the soil will be dependent on the water content. During the experiment, water content is determined by simply taking a small amount of the saturated soil, placing it in an empty container of known weight, and obtaining the oven-dry weight of the soil. To determine the percentage of the water content, the following formula is used:

Equation (1) $$w = \frac{W_2 - W_3}{W_3 - W_1} * 100$$

where:

W_1 is the weight of the empty container (grams)
W_2 is the weight of the container + wet soil (grams)
W_3 is the weight of the container + dry soil (grams)

The water content percentage calculated should be equal or approximate to the initial water content added to the soil.

- The void ratio is defined as the ratio of the volume of voids to the volume of solids. Voids within the soil may be filled with either water or air. When stresses are applied on the soil, the void ratio of a drained soil sample changes, because air exits the particles as loading is applied. However, during the loading of an undrained test, the air within the particles is taken out before loading is applied, so that the ratio of water and particles remains unchanged (to acquire a pore-water pressure value). The undrained test will be further discussed in the triaxial test procedure.

- Drainage conditions are chosen prior to applying loading to the soil sample. To prepare an undrained test on soil, the soil will be submerged in water inside of a cell, which will be used to apply the pore-water pressure, at no change in void ratio or volume. For a drained test, there will thus be no change in pore water pressure, and normal stresses and strains are analyzed to find the soil's shear strength.
- The isotropic media of the soil means that the fluid passing through a homogeneous soil* has a uniform direction.
- The rate of loading can either be constantly or increasing. In this experiment, the axial loads will increase until the soil comes to a failure and the shear strength is determined.

Shear Strength in Triaxial Tests

The shear strength of a soil's sample is found by applying an axial load (stress) to a molded cylindrical soil sample while observing the response of the soil (axial strains). The soil then comes to a failure state which is referred to as the unconfined compression strength. The triaxial test is the more sophisticated test procedure used to determine the shear strength of soil. For the triaxial test, the cell pressure is held constant while the axial load increases. In the lab, the force from the loading ram applied to the rigid top cap of the sample provided the axial stress. This will help produce a stress versus strain curve, where strain increases as the stress remains constant at the failure state.

***Important definitions for this section:**

stress: weight and pressure imposed on the soil
strain: a change in soils whether by compression (contraction) or tension (expansion)
bulk density: the ratio of the soil's particle mass to the volume occupied
homogeneous soil: same body of properties entirely
in-situ: soil existing in its natural state or position

Table 1 - Summary of Test Conditions During Shear Stage

Test Type	Rate of axial strain	Drainage
UU	Typically fastest, reaching failure criterion in 5 – 15 minutes	Closed, no excess pore pressure measurement
CU	Slow enough to allow adequate equalization of excess pore pressures	Closed, record excess pore pressure
CD	Slow enough to result in negligible pore pressure variation	Open, record ΔV & maintain constant back pressure

Definition of Critical State Concept

The Critical State concept, it is defined as the ultimate condition in which shearing could continue indefinitely without changes in volume or effective stresses. As also stated in *Soil Behaviour and Critical State Soil Mechanics*, the critical state line operates as a limit on the changes of effective stress (p'), deviator stress (q'), and specific volume (v) that occur in a test.

Using Equation (2): $\sigma = \sigma_{ef} + u$

Where:

σ = total stress (overall)
σ_{ef} = effective stress (active)
u = neutral stress (pore water pressure)

Two types of tests are considered: tests on normally consolidated soil or over-consolidated soil. To add to the previous section, normally consolidated soil is the maximum stress that soil experiences in the present, while void ratio decreases as the effective stress increases.
Over-consolidated soil are soils which top layers have been removed, while void ration increases and stress decreases.

Purpose

- To investigate the stress:strain behavior of soil
- To state the difference between unconfined compression (undrained vs. drained) within the specific type of soil

Parameters Measured at the time of Loading
The quantities measured in an Unconfined Compression Triaxial Test are the following:

- Pressure in cell fluid
- Axial Force
- Change in Length
- Change in volume
- Pore water pressure (drainage not permitted)

The most commonly performed triaxial test is when the cell pressure is held constant and the axial load is increased (Kolymbas 19).

Figure 1

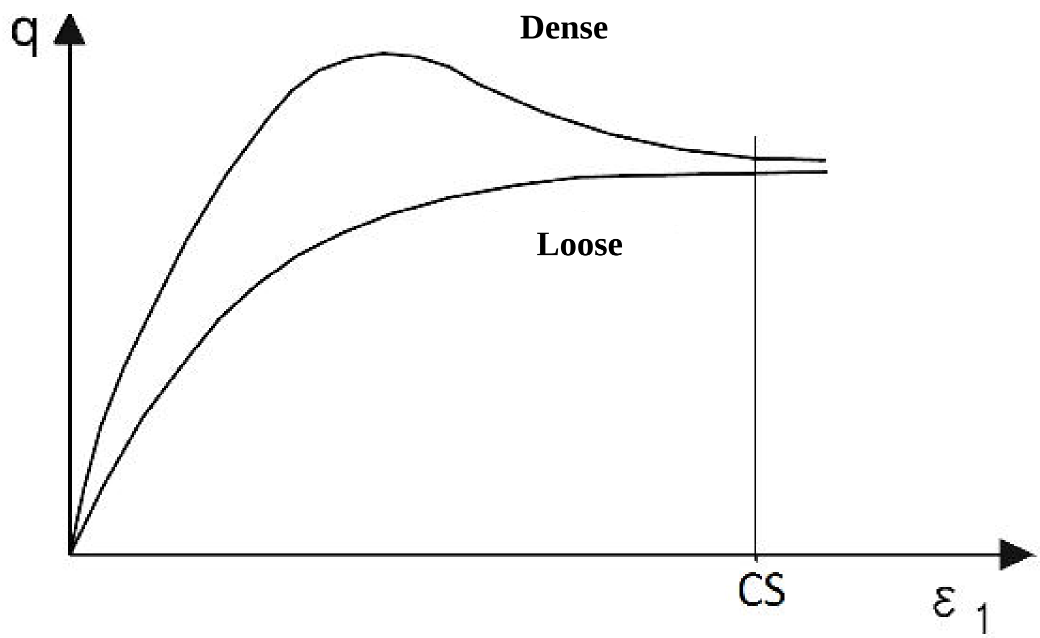


Figure 1 makes it clear that the critical state model is not the state with the maximum tensions, but the state in which the stresses no longer change (as the curve flattens).

Figure 2

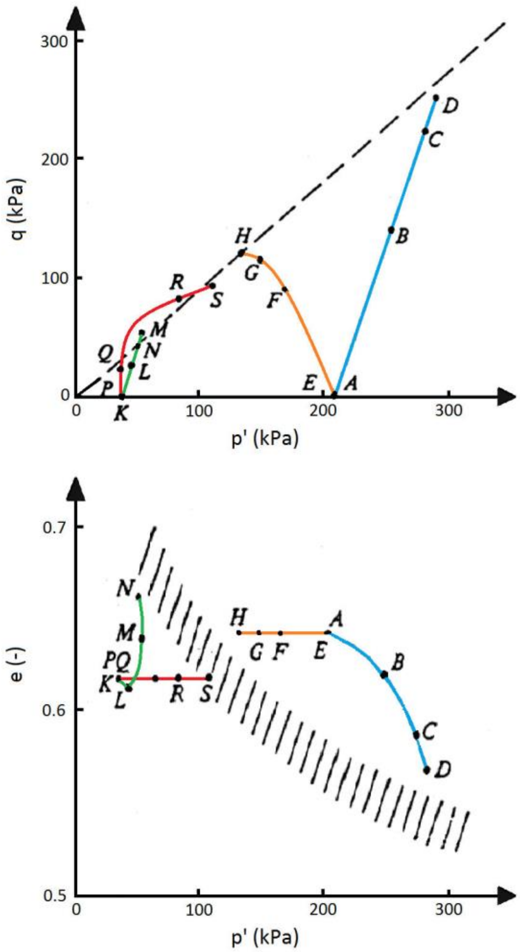


Figure 2 shows these two diagrams, which are the two projection planes of the Critical State Line. The Critical State Line is shown as a dashed line. This is the comparison between dense and loose samples graphed and the differential area in between is the pore water pressure while comparing undrained and drained tests.

Figure 3

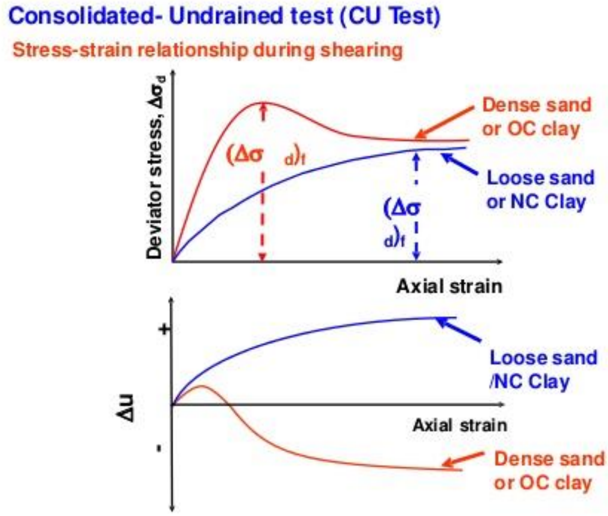


Figure 4 shows graph of axial strain versus deviatoric stress, and the curves of two tests reaching the critical state line. The red line represents a dense sample and the blue line represents a loose sample, as explained in the *Theory* section. These graphs support the theoretical conclusion which states that loose soils contract (normally consolidated) while dense soils expand (over consolidated). The second graph shows the axial strain versus the change in pore water pressure, and how the loose sample yields a positive value while the dense sample a negative. Both come to a critical state behavior as the curve reaches plateau.

Figure 5

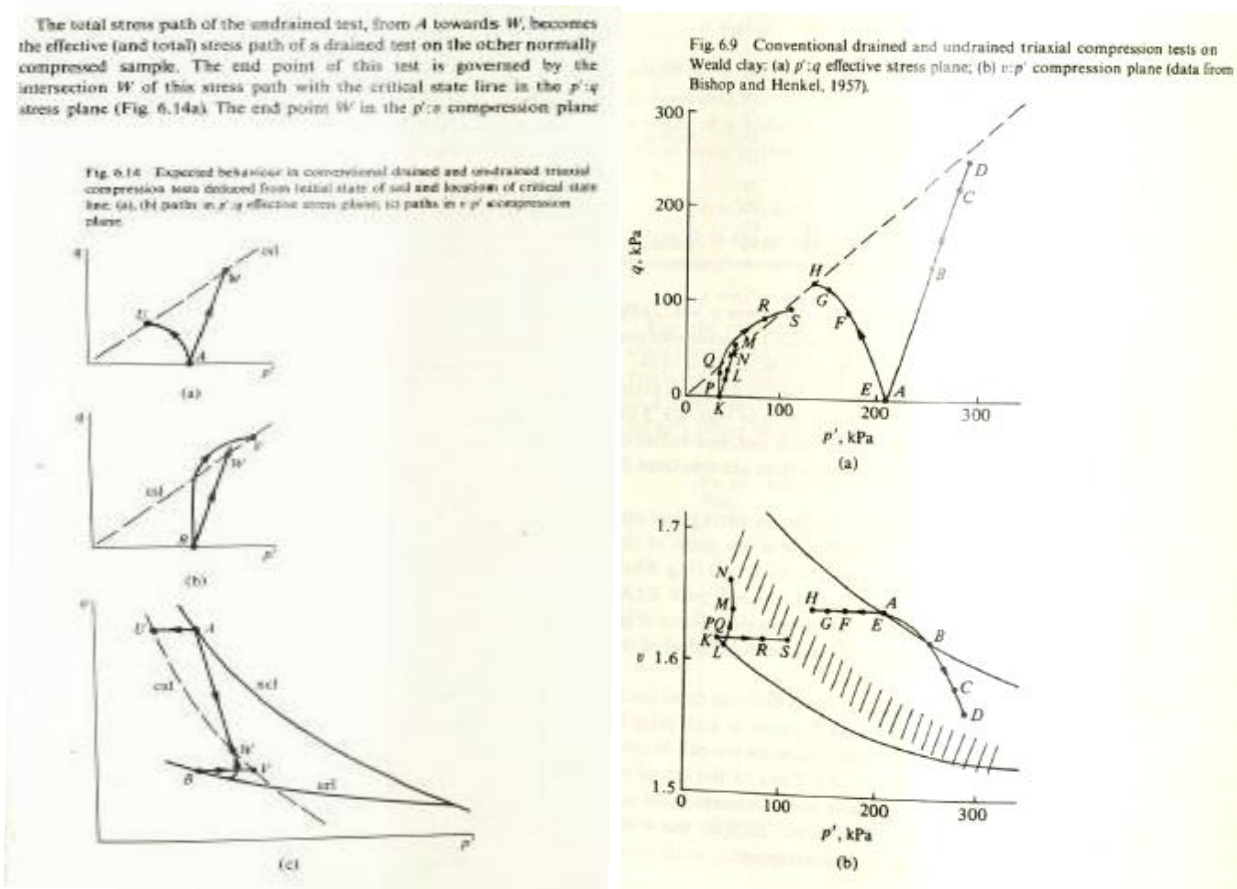

The total stress path of the undrained test, from A towards W, becomes the effective (and total) stress path of a drained test on the other normally compressed sample. The end point of this test is governed by the intersection W of this stress path with the critical state line in the $p':q$ stress plane (Fig. 6.14a). The end point W in the $p':v$ compression plane

Fig. 6.14 Expected behaviour in conventional drained and undrained triaxial compression tests deduced from initial state of soil and location of critical state line: (a), (b) paths in $p':q$ effective stress plane; (c) paths in $v:p'$ compression plane.

Fig. 6.9 Conventional drained and undrained triaxial compression tests on Weald clay: (a) $p':q$ effective stress plane; (b) $v:p'$ compression plane (data from Bishop and Henkel, 1957).

Figures 5 shows graphs with the standard response of loose and dense samples at the CSL (Wood 152, 159). The student should use attempt to get similar results after generating the graphs (see *Appendix* for student’s results).

Basics of Triaxial Test Experiments

Definition and Purpose of a Triaxial Test

The triaxial test is one of the most versatile and widely performed geotechnical laboratory tests, allowing the shear strength and stiffness of soil and rock to be determined for use in geotechnical design. Advantages over simpler procedures, such as the direct shear test, include the ability to control specimen drainage and take measurements of pore water pressures.
The Triaxial Test was preferred for this research experiment because it allows a sample of fine sand to remain undrained at a constant volume. The primary parameters obtained from the triaxial test include the pressure in the cell fluid, change in volume, pore water pressure, axial force applied, and change in length.
The name “triaxial” was given due to the stresses applied to the soil specimen in the x, y, and z directions, as seen in Figure 6. The stress state during a triaxial test is an elemental parameter of focus. The confining stress is applied by pressurizing the cell fluid surrounding the specimen. The deviator stress is generated by applying an axial strain to the soil. The stress state is said to be isotropic when $\sigma1 = \sigma3$, and anisotropic when $\sigma1 \neq \sigma3$ (GDS Instruments).
The test specimen itself must firstly be prepared from a sample of soil before placing into the triaxial cell. For cohesive soils (clay), this may involve trimming undisturbed specimens extruded from Shelby tubes or cut from block samples, while for granular soils (sand), the specimen may require preparation directly on the pedestal using a split-part mold. In the case of cohesive specimens, a membrane suction stretcher can be used to place the rubber membrane around the soil once in position on the pedestal. Note that disturbance to the specimen should be kept to a minimum during preparation.
The preparation of triaxial tests on fine sands is a time consuming process and the student has to allow time for failed tests. An example of failure during triaxial testing is when a molded sand sample collapsed if not compacted sufficiently (using blow hammer). More sources of error are explained in depth in the *Challenges* section of this report.

Triaxial trials were carried out on fine sand in order to obtain the Critical State line. Several undrained triaxial tests were performed using different bearing densities and voltage levels. After obtaining results showing either values, the critical state line could be created by interpolation. The Measurement results were evaluated using the Matlab test.

Figure 6: Diagram of Triaxial Cell Apparatus.

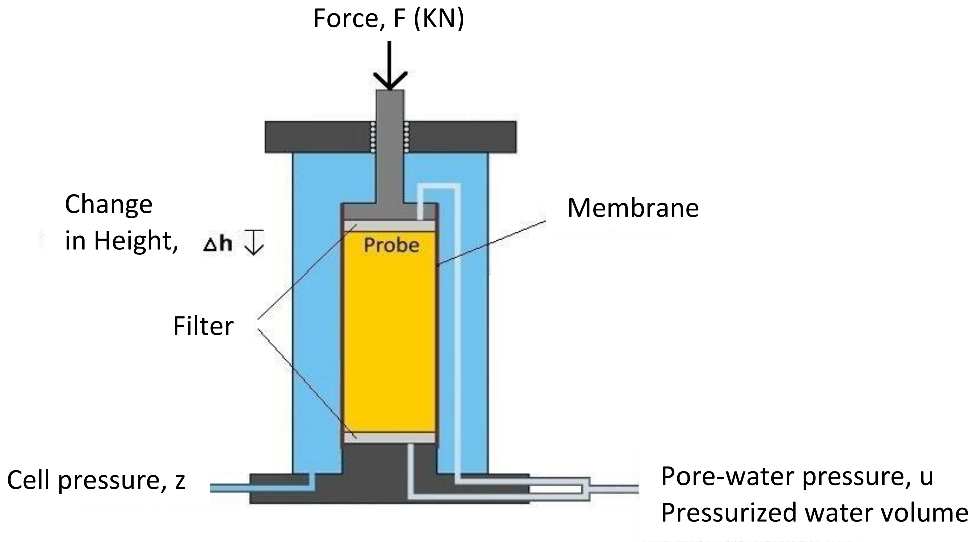
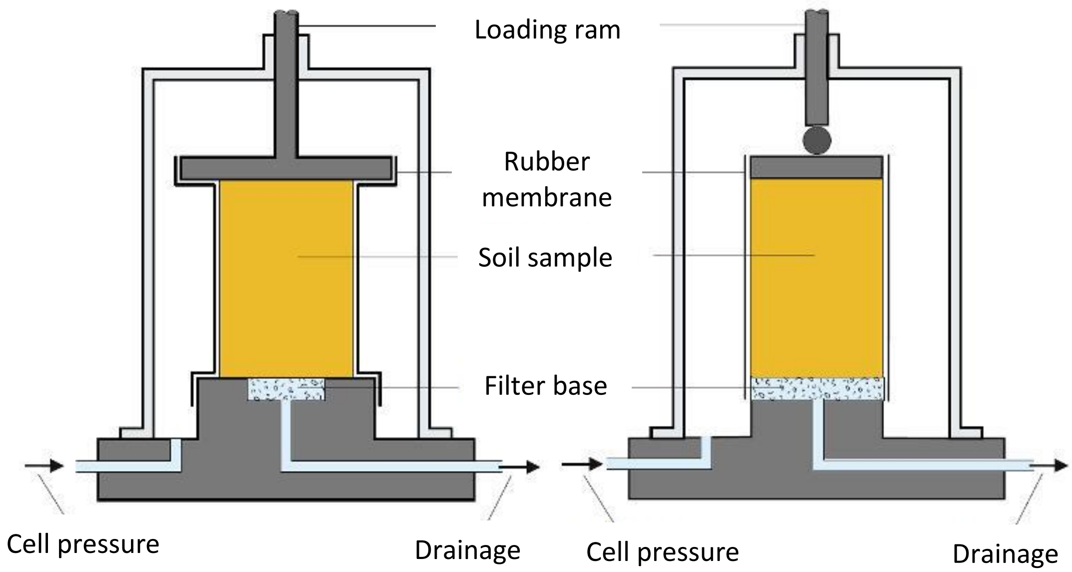


Figure 6 shows an ideal design of the conventional assembly of a triaxial test.

Figure 7: Simple and conventional design of Triaxial Cell Apparatus



As seen in **Figure 7**, and during the shearing process, the following measured values are recorded:

- Time, t
- Loading force, F
- Vertical compression of the probe height
- Cell pressure, p
- Pore water pressure, u and pressurized water volume from the sample, v

Figure 8: Cylindrical sample under an axially symmetrical stress state (x, y, and z axes).

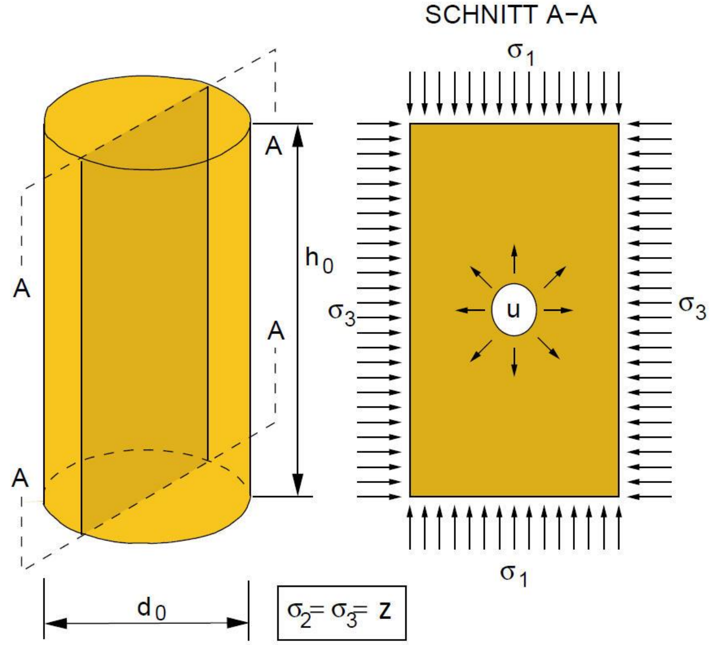


Figure 8 shows the total stresses σ_1, σ_2, and σ_3, the effective stresses σ'_1, σ'_2, and σ'_3, with the help of the pore water pressure u (or called the backpressure) can be determined with the following formulas:

$$\sigma'1 = \sigma1 - u \quad (1)$$

$$\sigma'2 = \sigma2 - u \quad (2)$$

$$\sigma'3 = \sigma3 - u \quad (3)$$

Photographs from UIBK Geotechnical Engineering Laboratory

Figure 9: Fine Sand used at the UIBK geotechnical laboratory

Description of Soil

The material used in this laboratory experiment is Fine Sand. Fine sand lies approximately in the middle of the different sand ranges (with 0.0049–0.010 in per grain).

In the course of this work fine sand (see **Figure 9**) was used for the performance of all the triaxial tests. More specifically, this sand is referred to as Ottendorf-Okrilla quartz sand. Below are the following physical characteristics:

• Grain size distribution: 0.1 mm - 0.5 mm
• Mean grain diameter d50: 0.24 mm
• Non-uniformity number CU: 1,9 (-)
• Corn shape: angular to rounded
• grain density ρ_s: 2.635 g / cm3
• Max. Pore count e_{max}: 0.87 (-)
• Min. Pore count e_{min}: 0.569 (-)
• Critical angle of friction φ_c: 34.6◦

Figure 10: Fine Sand mixed with given water content (w = 15% of the initial mass).

Sample Calculation for this step:

$$m_0 = 4201 \text{ g}$$

Equation (3) $$4201 * \frac{15}{100} = 630\ g$$

Thus, 630 mL of water are added to the 4201 g of sand and until both are mixed uniformly, as seen in **Figure 10**.

Figure 11: Proctor Compaction Apparatus

Initially, one has to assemble the Proctor Compaction Apparatus prior to beginning a new triaxial test, as shown in **Figure 11**. The center piece is used to tighten the three layers into place so that a soil cylinder with a height of 20.2 cm and a diameter of 10 cm can be compacted inside.

Figures 12 and 13: First Compacted Layer of Sand Inside of the Proctor Compaction Appparatus.

This step can be done either without the rubber membrane (as shown in **Figure 12**) or by initially inserting the rubber membrane (as shown in **Figure 13**) respectively. The rubber membrane is needed when the sample has a low bulk density (loose sand sample) and by applying minimal compaction to reach a height of approximately 20.2 cm (proctor height).

Sample Calculations and comparison between a dense and a loose sand sample:

Equation (4) $$\text{Volume, } V = r^2 * \pi * \text{h}$$

$$V = 5^2 * \pi * 20.2$$

$$V = 1{,}586.50 \text{ cm}^3$$

Bulk Density value for loose sample:

Equation (5) $$\rho_f = 1.50 \text{ g/cm}^3$$

Rearranging Equation (5) to find mass, m: $\rho f = \frac{m}{v}$

$$m = 1{,}586.5 * 1.50$$

$$m = 2{,}379.75 \text{ g}$$

Thus, 2,379.75 g of wet sand are used to build the cylindrical specimen for the triaxial test at a density of 1.50 g/cm^3 (loose sample).

Bulk density value for a dense sample: ρ_f = 1.60 g/cm^3

Using Volume, V = 1,586.50 cm^3

And solving for mass, m

$$m = 1{,}586.5 * 1.6$$

m = 2,538.40 g

Thus, 2,538.40 g of wet sand are used to build the cylindrical specimen for the triaxial test at a density of 1.60 g/cm^3 (dense sample).

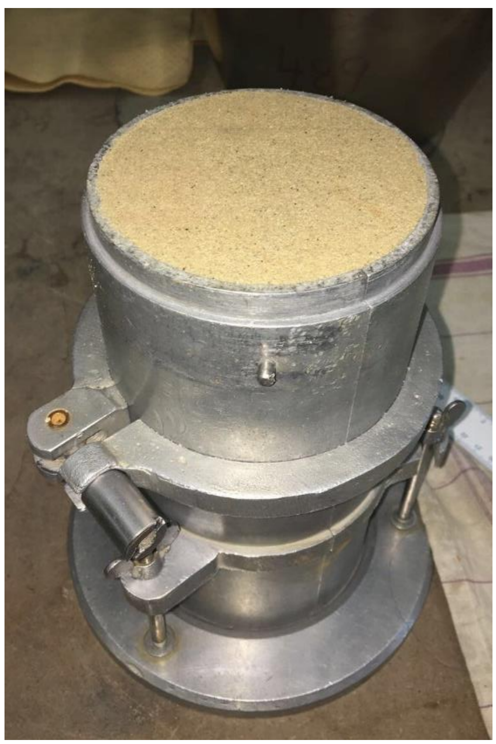
Physical properties on how to determine whether sample is loose or dense:

When compacting the sand inside of the proctor, a loose sample will tend to have a smaller height than a height of 20.2 cm. Loose samples tend to have a Bulk density of ρ_f < 1.55 g/cm^3.

Figure 14: Thin Filter stone and Filter Fleece (diameter = 10 cm)

Figure 14 shows both the filter stone and the filter fleece which are damped with water and then placed on top layer of the cylindrical specimen before it is placed on the filter base. A thick filter stone is then placed on top where the top filter base is placed (which permits water to flow through the material using pipes).

Figure 15: Wet Sand Inside proctor device with height of 20.2 cm



At the step shown in **Figure 15**, the proctor and the wet sand are weighted and prepared to be placed into filter bases and prepared for the loading frame.

Figure 16: Proctor device with sand and loading frame base

Figures 17 and 18: Placement of Cylindrical Sand Sample

In **Figure 17**, once the thin filter stone and filter fleece are placed on the first layer, the proctor device is placed on top of the filter base. Once placed, the thick filter with a damp filter fleece are placed in top of the exposed stop sand layer.

Following, the proctor device is removed from the cylindrical sample very slowly and carefully without damaging the cylindrical sand sample, until it is discovered intact as shown in **Figure 18**, respectively.

Figure 19: Rubber Membrane(s) Covering the Sand Cylindrical Specimen

After coating the rubber membranes with powder magnesium (as explained in the procedure) a membrane stretcher is used to tightly place the rubber membranes around the sand sample to keep it stable and upright.

Figure 20: Placement of Filter Bases with Pipes

The two black valves on the access ring control the water and air in and out of the specimen (during the saturation step). To allow water to flow through the base platen and in-and-out of the material, and also to fill up the cell chamber, valves are connected to the three rear openings, as seen in **Figure 20**.

Figures 21 and 22: Placement of Loading Frame Cover and Loading Ram

Figures 21 and **Figure 22** show the triaxial cell assembled in full right before loading. The final step us to carefully insert the loading ram through the opening at the top, as seen in **Figure 22**.

Figure 23: Assembly of Triaxial Cell with Respective Pipes

In this step, the triaxial cell is ready for loading to begin. The three valves seen connected to the loading frame, as shown in **Figure 23**, will be used to allow water to flow in and out of the material coming from the rear tanks and burettes. The two clear pipes connected (at the far left) are used for air bubbles to flow out of the material. The frame is then elevated using the small and large hand wheels until the loading ram is ready for loading (in full contact with the probe).

Figure 24: Probe Bulging at Loading

After the Shearing stage is completed, the cell is left to experience 10 kN-20 kN of loading force keeping a constant volume, until it has fully bulged and failed.

Figure 25: Specimen at the end of Shearing

Figure 25 shows the triaxial cell and the probe at its final stage of the triaxial test – bulging and failure. Once the triaxial database has completely recorded the values of the forces applied, the change in height, the path from the distance sensor, both the cell and pore water pressure, and the changes in volume, the test is concluded and ready for analysis.

Assembling the Triaxial Test Specimen

In the UIBK geotechnical laboratory, there were particular procedures taken by the student and the lab assistant each time the sand material was prepared for a Triaxial Test. The usual and recommended working steps are the following:

1. Weigh the bowl where sand will be placed
2. Fill bowl about halfway/with calculated amount of sand depending on the bulk density
3. Weigh the bowl with sand and subtract weight of bowl. Weight is approximately 4042 g.
4. 4042 x (w = 15%) = 606.3 g (take account the water content in the sand)
5. Fill beaker with 606.3 g mL of water and pour into sand
6. Mix sand and water until it is mixed uniformly
7. Weigh small container to take small amount of sand mix to obtain the water content. Place container + sand into a 105-degree oven.
8. Weigh proctor probe. Weight is approximately 2428.5 g.
9. Place three layers of sand mix into the proctor apparatus
10. Using the proctor hammer/stamp, compact each sand layer with two blows (or more) per layer
11. Weigh proctor apparatus with compacted sand
12. Remove the metal cover and carefully place rubber membrane around the sand cylinder
13. Place triaxial cell chamber to enclose the compacted cylinder

Detailed Triaxial Test Procedure

This section of the report is a detailed procedure exactly as experienced in the UIBK geotechnical laboratory. The procedure and parameters were all explained to me initially in Standard German, and all of my records from the several triaxial tests performed are both in English and German. The following steps were used to perform the loading to the triaxial sand specimen, and were done several times until reliable results were obtained. Following the steps, a small theoretical introduction of each operation is given as background to the reader.

Messwerte Pruefen – Check Measured Values

1. Wear safety glasses to avoid water to enter the eyes.
2. Tie long hair back so that eye perspective is completely clear.
3. Check that all valves and pipes are plugged into the cell and locked correctly
4. Raise cell using the hand wheels until the ‘stamp’ makes contact with the loading point. Check that ‘Kraftmessdose’ (force value of 10 or 20 kN) is at least at 0.01.
5. Raise cell pressure to 0.2 bar. Keep pressure at this value; pressure will rise as cell is being filled, so adjust it manually.

6. To fill up cell, fill up the ‘zellwasser’ (or water in the cell) by setting the green lever on ‘zu’ (closed) and the ‘zellefuellen’ (fill cell) blue lever on ‘auf’ (opened). Check that the burette valve is closed.
7. Close the first blue valve slightly and then re-open when the screw is closed.
8. While cell is filling up with water, fill the rear tank simultaneously by opening the first blue valve (right side). Once tank is full, close the valve.
9. Close the screw at the top of the cell once the cell is filled (when water begins to come out).
10. Decrease pressure to 0.2 bar.
11. Adjust the ‘Wegaufnehmer’ (distance sensor) until it makes contact (at 0.00 mm). Make sure that its path is clear so that it won’t touch any other surfaces during the loading.
12. For water to flow through the specimen “durchstroemen” (to let water exit out of external pipes into a pan):
    a) Open second blue valve (right side) to burette
    b) Open the third blue valve (right side)
    c) Open second blue valve (left side) without inner pipe and the corresponding black valve. Close both valves when air is out and water flow is steady.
    d) Open fourth blue valve (right side) with inner pipe and the corresponding black valve. Close both when air is out and water flow is steady.
    e) Open second blue valve (left side) without pipes and black valve with pipes to let the water flow through the specimen. Close both valves when air is out and water flow is steady.
    f) Open the blue valves to access the water.
    g) Check that both blue valves and black valves remain open.

*Note: When the tank below is empty, the water will start bubbling while filling up the cell. To refill the tank, turn the ‘zellefuellen’ lever to ‘zu’ (closed) and the ‘zellwasser’ lever to ‘auf’ (open). Open the black valve in the rear left and the black valve below the apparatus until the pipe below is filled halfway. To fill up rear tank faster, raise pressure 0.35 bar. Ask for assistance if necessary.

Saettigen – Saturation

1. Raise ‘zelldruck’ (cell-pressure) to 9.2 bar and ‘Porenwasserdruck’ (pore water pressure) to 9.0 bar simultaneously.
2. If the ‘stamp’ comes off contact with the cell, use the hand wheel to raise the cell and set it back into place.
3. Adjust the ‘Wegaufnehmer’ (path recorder) until it makes contact (at 0.00 mm).
4. Let the test run for ten minutes.

Saturation Theory - The saturation process is designed to ensure all voids within the test specimen are filled with water, and that the pore pressure transducer and drainage lines are properly de-aired. This may be achieved by firstly applying a partial vacuum to the specimen to remove air and draw water into the transducer and drainage lines, followed by a linear increase of the cell and back pressures. At saturation, the pressure in the sample goes from the pressure level: cell pressure z = 0.2 bar and backpressure u = 0 bar, slowly to the desired saturation pressure elevated. This difference of 0.2 bar between cell pressure and pore water pressure should be maintained for a duration of at least 10 minutes. If the pressure difference is too large, over-consolidation takes place instantly. On the other hand, if the sample experiences an excessively small pressure difference or a negative pressure difference, its overpressure arises and damages the sample by making it bulge. To assist the specimen in reaching full saturation, the following steps may be taken: Use of de-aired water to fill specimen voids. Increase of back pressure to force air into solution. The saturation pressure level is then held constant for a certain time to allow the remaining air in the sample into an air solution. It is important that no air remains inside the sample, since the air can cause the sample to be compressed. If the sample undergoes compression during saturation, the results are not as clear and can also attribute to a deformation of the sample.

B-Test

1. Close the third blue valve (right side) which permits access to bottom blue valves.
2. Raise the ‘zelldruck’ to 10 bar. Do not adjust the ‘Porenwasserdruck’ (pore water pressure).
3. Check that ‘B-Wert’ (B-value) reaches approximately 100%.
4. Return to the main screen where all tests are displayed. Lower the ‘zelldruck’ back to 9.2 bar, until it says ‘OK’ (green sign).
5. Open the third blue valve to access water and close the second blue valve to burette (right side).
6. Open the first blue valve (left side) to take water out of the burette into a bowl by holding out the transparent pipe by the valves.

B-Test Theory - Whether the sample is sufficiently saturated is controlled by the B-test. The B-test is carried out with the cable closed by increasing the cell pressure by 10%. This results in increasing the pore water pressure, and then it is measured. The B value must be at least 95%. After the B test is finished, the sample is again placed under the saturation pressure and the third blue valve (to access water) is opened.

Konsolidierung – Consolidation

1. On the graph generated by the program, change to x-axis to ‘Wurzel der Zeit’ (root of time).
2. Raise ‘zelldruck’ (cell-pressure) to 12 bar.
3. If the ‘stamp’ comes off contact, use the hand wheel to raise the cell and set it back into place. Check the ‘Wegaufnehmer’ (distance sensor).
4. Watch the burette. If it fills up, click on ‘burette regeln’ (empty/clear burette) or F5 on the keyboard to regulate it. Repeat B-Test - step 6.
5. Watch the ‘Volumenaenderung Burette (cm^3) and Wurzel der Zeit (sec)’ (Change in volume and root of time) graph until line becomes horizontal and starts increasing again.

Consolidation Theory - The consolidation stage is used to bring the specimen to the effective stress state required for shearing. It is typically conducted by increasing the cell pressure while maintaining a constant back pressure (often equal to the pore pressure reached during the final saturation B-check). This process is continued until the volume change ΔV of the specimen is no longer significant, and at least 95% of the excess pore pressure has dissipated. During consolidation, the cell pressure z is adjusted to the desired starting value at which the test sample is sheared, while the pore water pressure remains the same. As a result, the sample deforms slightly, which must be taken into account during the evaluation. This causes water to leak out of the water sample is pressed and the water volume in the burette increases. This change in volume in the burette ΔV is measured. The consolidation of the sample is completed when the volume and the water volume in the burette no longer change significantly. By the end of consolidation, the sample is under a hydrostatic stress condition, shown by the stress relationship σ'1 = σ'2 = σ'3 (see **Figure 8**).

Before the consolidation phase, the sample has its initial dimensions - the height h_0 and the initial diameter d_0. In the consolidation phase, the dimensions of the sample change to the consolidation height h_c and to the consolidation diameter d_c. This is due to the increase in cell pressure.

Abscheren – Shear

1. For 1 mm load control wheel, set ‘geschwindigkeit’ (speed) to 0.5 mm/min.
2. Check the ‘Wegaufnehmer’.
3. Close the third blue valve (right side) to access bottom valves.
4. Start ‘Abscheren CU’(consolidated undrained shear) and raise ‘max Stauchung’ (compressive force) to 33%.
5. Check that the ‘handrad’ (hand wheel) is spinning clockwise.
6. Let the test run for seven to eight hours.

Shear Theory - The soil is sheared when an axial strain ε_a is applied to the test specimen at a constant rate through upward (compression) or downward (tension) movement of the load frame platen. This rate, along with the specimen drainage condition, is dependent on the type of triaxial test being performed. Specimen response during the shear stage is typically monitored by plotting the deviator stress q or effective principal stress ratio σ´1/σ´3 against the axial strain ε_a. The stage is continued until a specified failure criterion has been reached, which may include identification of the peak deviator stress or peak effective principal stress ratio, observation of constant stress and excess pore pressure / volume change values, or simply a specific value of axial strain being reached (for example ε_a = 20%).

The shear is the actual axial load y on the sample. The sample is removed by increasing the vertical stress σ_1 below an axial-symmetrical load condition. The shear can be force-controlled or performed by steering. In the case of the force-controlled shear, sheering is reached until the sample fails. One drawback of this variation is that the sample is only up to a voltage peak or to the failure input. By the remote control, the failure is caused by a constant compression of the sample. More often, the path-controlled variant is carried out, since the path-controlled variant is also the behavior with respect to a voltage peak. The rate of shear is the speed, or how fast the sample is being compressed. Shearing is dependent on the following parameters:
• Type of experiment
• Type of draining ξ (at one end or at both ends)
• Degree of solidification χ, mostly 0.95 (-) (practically completely solidified)

• total solidification time t_{100}, which in turn is dependent on the composition of the sample
• Ratio α of the initial sample height to the sample height at the beginning of the fracture process
• Assumed displacement of the sample at break, Δh_B

For example, for the rate of shear in drained attempts, the pore water squeezed out needs to be less than in the case of undrained. The shearing process and thus the triaxial test is considered finished when the voltage maximum, or the vertical compression ε_1 of the sample, reaches 20%. The vertical compression is ϙ1 with $\varepsilon_1 = \Delta h$ defined. As already mentioned, Δh is the vertical compression of the sample into a specific time, t and h is the sample height after the consolidation phase. The achievement of the voltage maximum is equated with a fraction. It refers to a shear fracture, which usually occurs in dense samples. When loose samples bulge the sample, this is due to many microscopically small samples experiencing shear cracking. If no voltage maximum occurs, the test is carried out at a 20% compression.

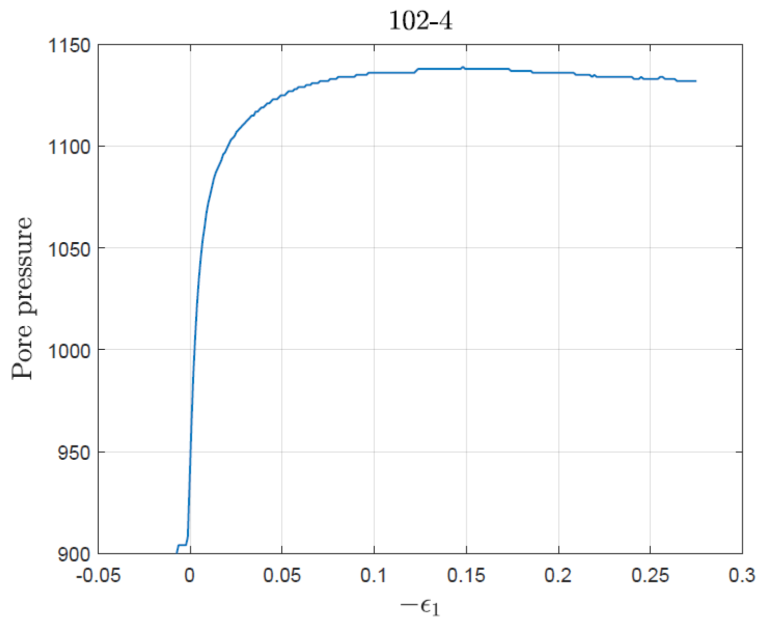
To Disassemble the Triaxial Cell

1. Lower both the cell and pore water pressure back to zero.
2. Use the large hand wheel to lower the cell back into the base
3. Open the green lever to ‘auf’(open) to drain the water, and open the top screw to release the internal pressure
4. Gather data files from folder generated by the Triaxial database
5. Remove all apparatus carefully
6. Take the wet sand from the experiment and place it on over for it to be reused
7. Clean and wash off any remaining sand particles left on the apparatus
8. Allow it to air dry for it to be used the next day

Data Obtained and Analysis

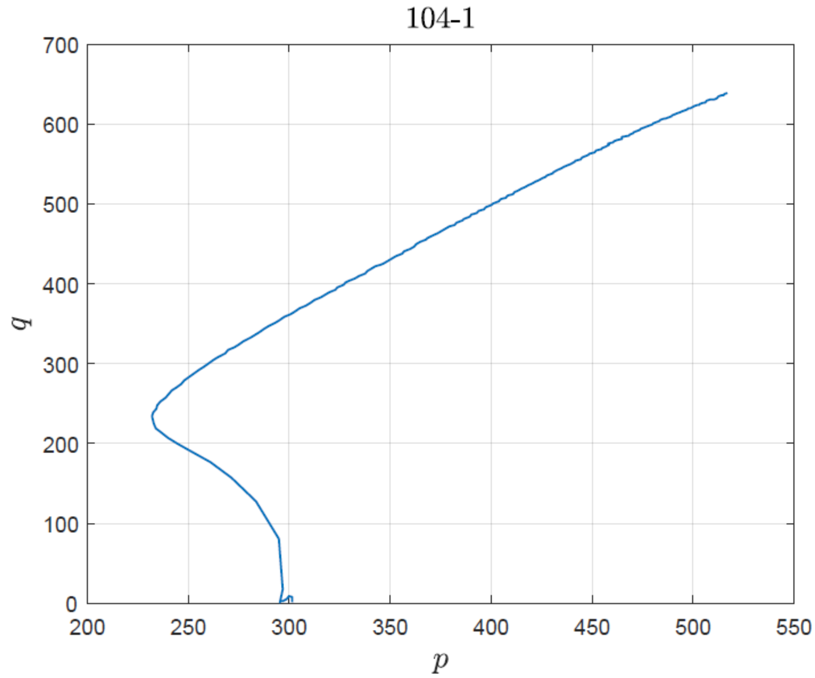
The following graphs were generated using the Matlab software. Note that these graphs were chosen because they produced the clearest results showing the quality of the critical state line. The parameters inputted into the program are the following: bulk density (g/cm^3), initial height (cm), Force (kN), initial diameter (cm), changes in volume (zero for undrained tests), cell pressure (bar), pore water pressure (bar), and change in time (sec).

Graph 1: Pore water pressure vs. Strain



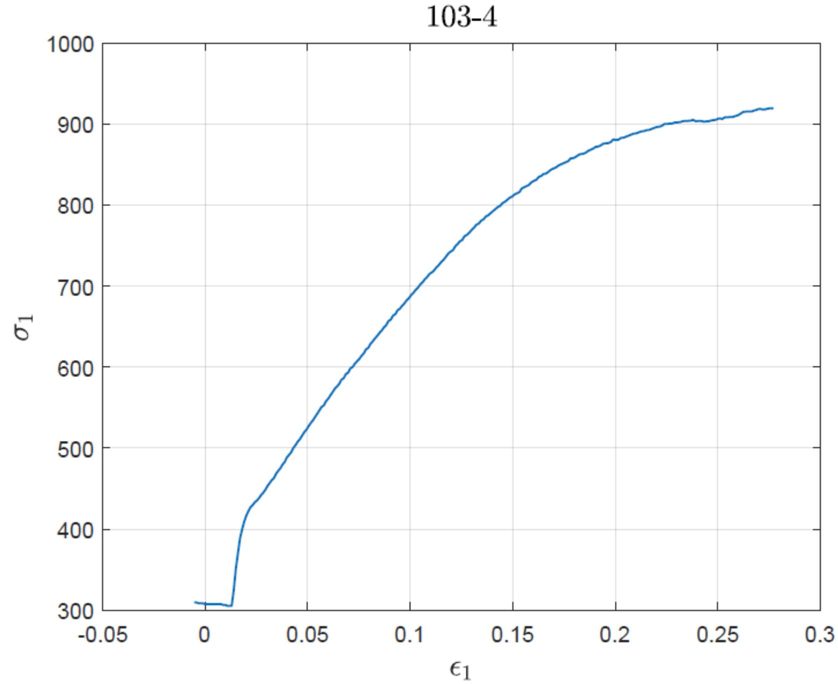
Graph 1 shows the sample reaching a Critical State limit as strain in sample increases and pore water pressure increases.

Graph 2: Deviatoric Stress vs. Effective Stress



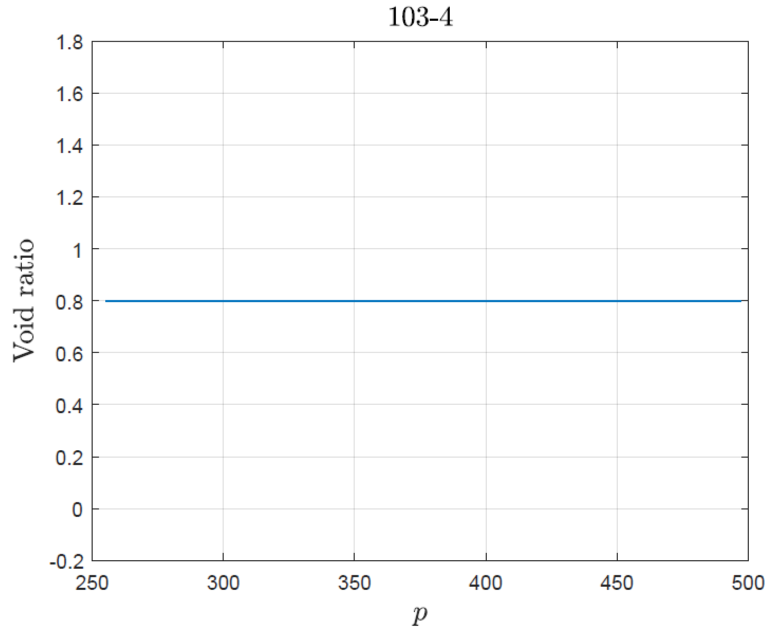
In **Graph 2**, a particular behavior is observed. The 'elbow' shape to this graph means that the sample behaved as a loose sample initially (with decrease in effective stress) and shifted to a dense sample until the end of the test period. This theoretical complexity is identified in *Soil Behavior and Critical State Soil Mechanics* book as "unexpected behavior". In theory, loose samples tend to contract while dense samples tend to expand at shear. The Critical State Line is the proportional increase of p and q, as seen in **Graph 2**. These concepts can be compared to **Figure 2** for a better understanding of loose and dense samples.

Graph 3: Total Stress vs. Strain



As the Stress vs. Strain reaches a maximum, as seen in **Graph 3**, a Critical State Limit is reached as the curve comes to a pleateau, or a constant stress.

Graph 4: Void Ratio vs. Effective Pressure



The Void Ratio vs. Effective Pressure shown in **Graph 4** is a straight line with no change in void ratio. This is due to the zero change in volume for all of the undrained tests performed. For drained tests in clay or sand, this graph would tend to have a positive slope instead.

Additional graphs are found in the *Appendix* section, and compared to the graphs in the *Data* section.

Challenges Encountered During Testing and Solutions

The beginning of testing was usually challenging and time consuming because of the molding of the specimens (time span: approximately 3-6 hours). With soil laboratory experiments, it was usually difficult to plan a set schedule, as often unexpected problems occurred. For example, special care was needed while preparing cohesion-less specimens, where test delay and sources of error occurred from misplacing the mold membrane in proper position. After molding the soil, the specimens were consolidated at defined mean effective stresses *p*. Each experiment was repeated at least once (using a trial-and-error approach), and each test usually took up to about a day (after waiting for full shearing stage to finish). For this reason, the total testing time period took approximately a full semester's work, not including the time spent on conclusions and producing the Matlab data. During testing, Dr. Schneider-Muntau and I analyzed and judged the results together, along with the help of other university faculty members. The official results were finalized after all testing is was completed.

Before commencing the triaxial laboratory testing, Stefan Tilg (UIBK geotechnical laboratory assistant) performed a couple of full triaxial tests in order for the to observe and take notes. He explained everything step by step, and for the most part, in German. Because of the language barrier, I had to learn everything mostly by sight and through trial-and-error, which was moreover truly beneficiary for me.

During the performance of triaxial experiments, I kept in mind that this test is a lengthy experience in which many precautions have to be taken. The triaxial test requires much patience and care while assembling the sample into the proctor, and lots of attention and preciseness while performing the loading with the triaxial database program.

The main challenges I encountered during the triaxial test assembly are the following:

1. Distributing a uniform water density within the soil specimen (while mixing)
2. Building a uniform soil sample
3. Separating the soil sample into three separate layers while compacting with a blow-hammer or stamp
4. Applying the rubber membrane to keep sample intact within the proctor
5. Keeping water and air out of the soil sample when using valves
6. Maintaining the cell pressure and pore water pressures at a stable interval for air to exit the soil sample and prior to loading
7. Keeping the loading stamp (located on top of the loading frame) in place while maneuvering the cell and pore water pressures

Solutions to reduce human error and to achieve a successful triaxial test experience:

1. Pour water in sections (while mixing) and not all together so that it is mixed uniformly throughout the sand
2. Ensure that proctor device is assembled tightly and correctly. Always double check that the amount of soil is adequate for the respective bulk density
3. Use a ruler to measure height of soil layer after its compaction. Make sure that the height is close to the specified height (proctor height 20.2 cm)
4. Always ensure that rubber membrane has been coated with powdered magnesium to reduce friction and sticking.
5. Double check the external pipes so that water flows continuously (without any air bubbles). Raise cell pressure slightly so that water flows in a slightly faster.
6. If both the cell and water pressures are not kept within a 0.2 bar interval, this could significantly add over-pressure to the sample and damage it by bulging. When performing this step, one has to make sure they are raised simultaneously at the given rates while observing the sample closely
7. While raising up the cells, the loading stamp sometimes comes off contact with the loading frame. To place it back into place, let go off of the pressures knobs and use both the fine and large hand wheels to raise the cell back in place with the stamp. Do this while observing that the loading force value does not exceed 0.01 kN. The 'path taker' has to be adjusted back to 0.00 mm at this point as well. When all of this has been checked, resume with raising the pressures.

Conclusion

The purpose of this entire research assignment was to closely analyze the Critical State Line of Soils with a focus on the fine sand material. Thanks to all of the help from the professors and mentors at the University of New Orleans and at the University of Innsbruck, all of the concepts, theories, and physical procedures were thoroughly explained so that I could master it and eventually produce my own analysis on the subject. After analyzing the CSL concept closely and performing several triaxial tests while facing numerous challenges, I can say that this assignment was very complex but extremely rewarding and entertaining. In theory and using different CSL book theory as guidance, the Critical State Line is studied and observed using clay samples (as studied in David Muir's book using Cambridge Clay). However, the study on fine sand is a growing study, which made it a fresh start for me, with lots of room for learning and creating unique hypotheses and conclusions.

The main points I gathered from the Critical State Line concept is that there is a focus on qualitative observation rather than the qualitative. For this reason, several tests needed to be performed in order to observe their behavior as loose and dense specimens. This concept is proven with the graphs that I obtained from the Matlab software, which reached a critical state line limit, or 'unexpected behavior'. Once these points are evaluated further and plotted in a regression curve (using all of the tests performed in the lab, both loose and dense), an ultimate critical state line can be observed as all of these points reach it. These results can then be put into records and compared to those demonstrated in David Muir Wood's book against the behavior of clay samples.

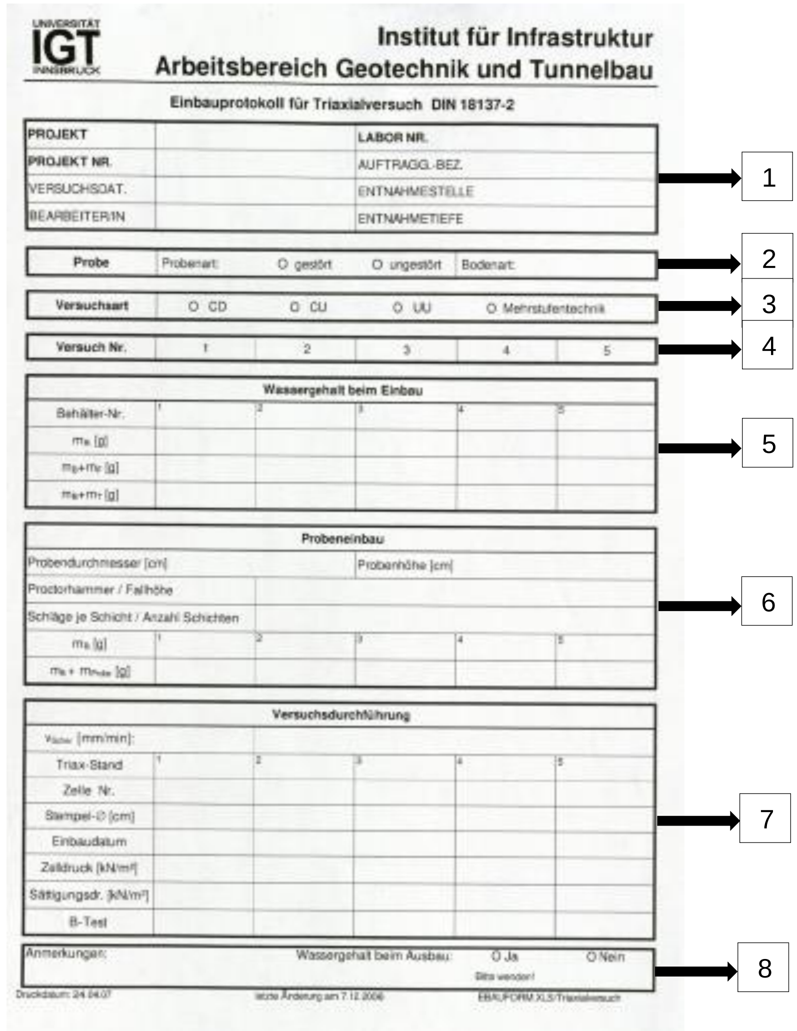
I can certainly say that I am not finished analyzing and understanding this topic, and as Dr. Barbara Schneider-Muntau stated, "It is a very complex and complicated concept indeed." The performance of physical triaxial tests and the analysis of graphs brought me a step closer to comprehending this phenomenon, and if I was to given the opportunity to pursue it further, I would accept it eagerly without hesitation. As a future goal, I would like to see how these testing procedures are used as a dependent value for the construction of structures, and observing the deformation of the sand material.

Acknowledgement

I would like to express my gratitude to the sponsors, professors, faculty, assistants, and mentors that helped me fulfill this research experience by providing either the financial support or the devoted time, help, and advice. A special thanks goes to all the faculty at the University of Innsbruck Geotechnical and Tunnel Engineering Unit: to Professor Barbara Schneider-Muntau for leading and mentoring me through the entire research process, to Stefan Tilg for his dedication and patience provided during the laboratory experiments, to Fabian Schranz and Iman Bathaeian for explaining and clarifying theoretical concepts and helping me utilize and navigate the Matlab software, to Marcus Maier for serving as a point of network to the UIBK geotechnical department and helping me join in this research assignment, and to Professor Dimitrios Kolymbas for encouraging me during the research process and providing useful advice based on his geotechnical engineering expertise. Also, a special thanks goes to all the faculty at the University of New Orleans College of Engineering: to Professor and Dean Emir J. Macari for serving as the professor of record and for being a home-university representative and supporter, and to Professor Malay G. Hajra for helping me with the application process prior to the commencement of the research project and for his ongoing support and mentorship. A special thanks also goes to the directors at the University of New Orleans AYA Innsbruck-UNO: to Dr. Irene Ziegler for being a present support from the beginning to the end of the application process and throughout my time in Austria, and to Professor Margaret Davidson for providing moral and financial support at all times, for helping me with the application process, for serving as a chief mentor during my stay in Austria, and for continuously overlooking my research progress. Last but not least, a special thanks goes to my principal sponsor: The Austrian Marshall Plan Foundation for giving me the chance to obtain fellowship with the Republic of Austria and for serving as the primary financial support for this research assignment. I heartily express my thanks to all other concerned persons who cooperated with me and helped me fulfill this research experience.

Appendix

Figure 26



UNIVERSITÄT IGT INNSBRUCK

Institut für Infrastruktur
Arbeitsbereich Geotechnik und Tunnelbau

Einbauprotokoll für Triaxialversuch DIN 18137-2

PROJEKT		LABOR NR.	
PROJEKT NR.		AUFTRAGG.-BEZ.	
VERSUCHSDAT.		ENTNAHMESTELLE	
BEARBEITER/IN		ENTNAHMETIEFE	

Probe	Probenart:	O gestört	O ungestört	Bodenart:

Versuchsart	O CD	O CU	O UU	O Mehrstufentechnik

Versuch Nr.	1	2	3	4	5

Wassergehalt beim Einbau

Behälter-Nr.	1	2	3	4	5
m_B [g]					
m_B+m_f [g]					
m_B+m_t [g]					

Probeneinbau

Probendurchmesser [cm]			Probenhöhe [cm]		
Proctorhammer / Fallhöhe					
Schläge je Schicht / Anzahl Schichten					
m_B [g]	1	2	3	4	5
$m_B + m_{Probe}$ [g]					

Versuchsdurchführung

$v_{Vorschub}$ [mm/min]:					
Triax-Stand	1	2	3	4	5
Zelle Nr.					
Stempel-∅ [cm]					
Einbaudatum					
Zelldruck [kN/m²]					
Sättigungsdr. [kN/m²]					
B-Test					

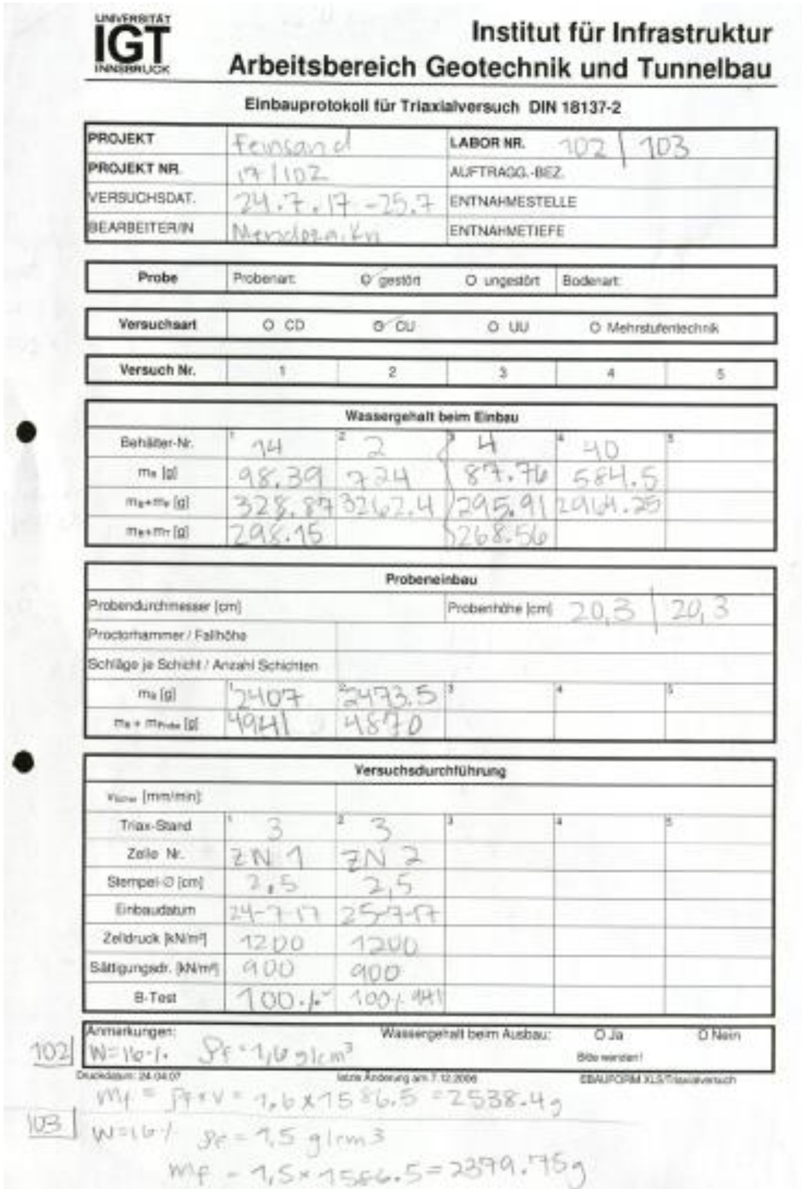
Anmerkungen: Wassergehalt beim Ausbau: O Ja O Nein
Bitte wenden!

Druckdatum: 24.04.07 letzte Änderung am 7.12.2006 EBAUFORM.XLS/Triaxialversuch

Figure 26 shows UIBK's laboratory protocol sheet used by the department of Geotechnical and Tunnel Engineering for the installation of a Triaxial Test. The procedure of each block will be described in numbered sections below:

Section 1: Project/experiment title, number, date, and name of student/lab technician.
Section 2: Sample type, whether disturbed or undisturbed, and soil type
Section 3: Type of test, whether consolidated drained, consolidated undrained, unconsolidated undrained, or multi-staged testing.
Section 4: Number of test trials
Section 5: Water content during installation
Section 6: Apparatus and installation values
Section 7: Testing operations
Section 8: Notes or comments

Figure 27



UNIVERSITÄT IGT INNSBRUCK

Institut für Infrastruktur
Arbeitsbereich Geotechnik und Tunnelbau

Einbauprotokoll für Triaxialversuch DIN 18137-2

PROJEKT	Feinsand	LABOR NR.	102 | 103
PROJEKT NR.	17/102	AUFTRAGG.-BEZ.	
VERSUCHSDAT.	24.7.17 - 25.7	ENTNAHMESTELLE	
BEARBEITER/IN	Mendoza, Kri	ENTNAHMETIEFE	

Probe	Probenart:	O gestört	O ungestört	Bodenart:

Versuchsart	O CD	O CU	O UU	O Mehrstufentechnik

Versuch Nr.	1	2	3	4	5

Wassergehalt beim Einbau

	1	2	3	4	5
Behälter-Nr.	14	2	4	40	
m_B [g]	98.39	724	87.76	584.5	
m_B+m_f [g]	328.87	3262.4	295.91	2964.25	
m_B+m_T [g]	298.15		268.56		

Probeneinbau

Probendurchmesser [cm]			Probenhöhe [cm]	20,3 | 20,3	
Proctorhammer / Fallhöhe					
Schläge je Schicht / Anzahl Schichten					
m_B [g]	2407	2473.5			
$m_B + m_{Probe}$ [g]	4941	4870			

Versuchsdurchführung

	1	2	3	4	5
v_{Presse} [mm/min]:					
Triax-Stand	3	3			
Zelle Nr.	ZN 1	ZN 2			
Stempel-Ø [cm]	2,5	2,5			
Einbaudatum	24-7-17	25-7-17			
Zelldruck [kN/m²]	1200	1200			
Sättigungsdr. [kN/m²]	900	900			
B-Test	100 %	100 % 441			

Anmerkungen: Wassergehalt beim Ausbau: O Ja O Nein Bitte wenden!

102 $W = 16\%$ $\rho_f = 1{,}6\ g/cm^3$

Druckdatum: 24.04.07 letzte Änderung am 7.12.2006 EBAUFORM.XLS/Triaxialversuch

$m_f = \rho_f \times V = 1{,}6 \times 1586.5 = 2538.4\ g$

103 $W = 16\%$ $\rho_f = 1{,}5\ g/cm^3$

$m_f = 1{,}5 \times 1586.5 = 2379.75\ g$

Figure 27 shows UIBK's laboratory protocol sheet filled out by the student-intern with the data from two triaxial test trials.

Figure 28

Consolidated-Undrained Triaxial Test

Preliminary Data

Description of soil ______________________ Specimen no. ________

Location ____

Tested by ______________________________ Date ________

Beginning of Test	
1. Moist unit weight of specimen (beginning of test)	
2. Moisture content (beginning of test)	
3. Initial length of specimen, L_0	
4. Initial diameter of specimen, D_0	
5. Initial area of specimen, $A_0 = \frac{\pi}{4}D_0^2$	
6. Initial volume of specimen, $V_0 = A_0L_0$	
After Consolidation of Saturated Specimen	
7. Cell consolidation pressure, σ_3	
8. Net drainage from specimen during consolidation, ΔV	
9. Volume of specimen after consolidation, $V_0 - \Delta V = V_c$	
10. Length of specimen after consolidation, $L_c = L_0\left(\frac{V_c}{V_0}\right)^{1/3}$	
11. Area of specimen after consolidation, $A_c = A_0\left(\frac{V_c}{V_0}\right)^{2/3}$	

291

Figure 28 shows the laboratory experiment data sheet used in a typical soil mechanics class at the University of New Orleans (Das 291).

Figure 29

Consolidated-Undrained Triaxial Test

Axial Stress-Strain Calculation

Proving ring calibration factor ______________________

Specimen Deformation ΔL (cm) (1)	Vertical Strain $\varepsilon = \frac{\Delta L}{L_c}$ (2)	Proving Ring Dial Reading (no. of small div.) (3)	Piston Load P (N) (4)	Corrected Area $A = \frac{A_c}{1-\varepsilon}$ (cm^2) (5)	Deviatory Stress $\Delta\sigma = \frac{P}{A}$ (kN/m^2) (6)	Excess Pore-Water Pressure Δu (kN/m^2) (7)	$\bar{A} = \frac{\Delta u}{\Delta\sigma}$ (8)

293

Figure 29 shows the data sheet used for the axial stress-strain calculations to produce the graphs for the analysis. These values are executed by the triaxial test numerical database program connected to the unit.

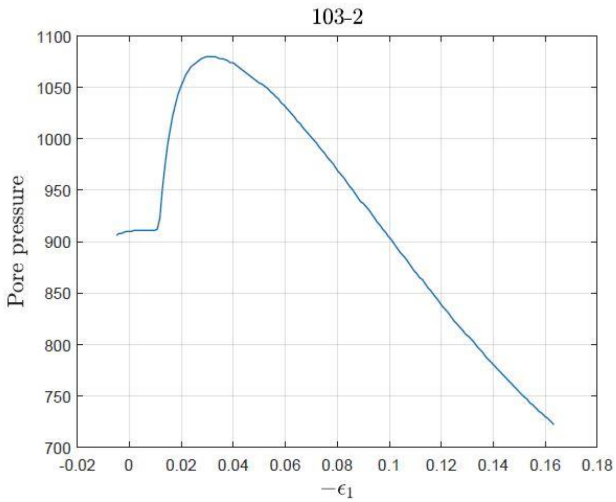
Figure 30: Data Produced by Triaxial Test Database Program

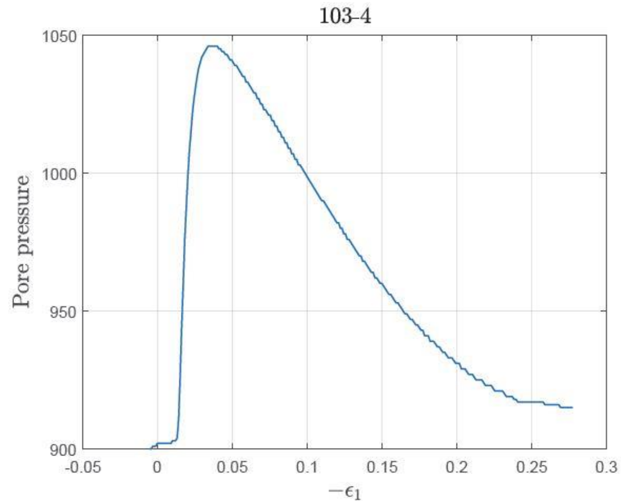
```
Konsolidieren
Zeit        Kraft     Weg    Zell   Poren    Diffd     Vol
             kN        mm     bar    bar      mbar    cm^3
0.20  0.42  0.00  9.12  8.96  -0.08 0.03
1292.94     0.60  -0.26 12.01 9.00  76.97 -35.21
Abscheren
Zeit        Kraft     Weg    Zell   Poren    Diffd     Vol
             kN        mm     bar    bar      mbar    cm^3
0.08  0.59  0.00  12.01 9.00  0.00  0.00
119.06      0.59  0.20  12.01 9.04  -0.01 0.00
131.36      0.59  0.41  12.01 9.04  -0.01 0.00
143.05      0.59  0.61  12.01 9.04  -0.00 0.00
154.34      0.59  0.81  12.01 9.04  -0.01 0.00
165.45      0.59  1.02  12.01 9.04  -0.02 0.01
176.77      0.72  1.22  12.01 9.08  -0.01 0.00
189.30      1.20  1.42  12.01 9.35  -0.01 0.00
201.22      1.49  1.62  12.01 9.63  -0.01 0.01
213.53      1.67  1.83  12.01 9.88  -0.01 0.00
225.45      1.78  2.03  12.01 10.06 -0.01 0.00
237.37      1.85  2.23  12.01 10.22 -0.01 0.01
249.31      1.89  2.44  12.01 10.35 -0.01 0.00
261.03      1.91  2.64  12.01 10.45 -0.01 0.00
272.75      1.92  2.85  12.01 10.54 -0.01 0.00
284.06      1.93  3.05  12.01 10.60 -0.01 0.00
295.80      1.93  3.25  12.01 10.67 -0.01 0.00
307.52      1.93  3.45  12.01 10.72 -0.01 0.01
319.23      1.92  3.66  12.01 10.76 -0.01 0.00
330.77      1.92  3.86  12.01 10.80 -0.01 0.01
342.48      1.91  4.06  12.01 10.84 -0.01 0.01
354.02      1.91  4.27  12.01 10.87 -0.01 0.01
365.53      1.90  4.47  12.01 10.89 -0.01 0.00
376.84      1.89  4.67  12.01 10.91 -0.01 0.00
```

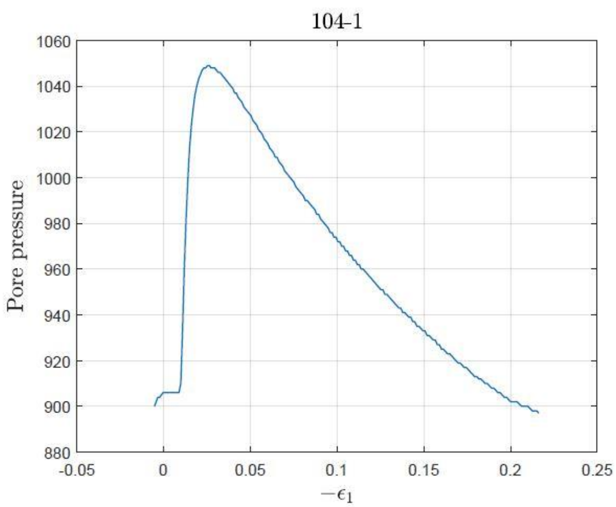
Figure 30 shows the values generated during the ‘Konsolidieren’ (consolidation) and ‘Abscheren’ (shear) during a triaxial test. The database records the ‘Zeit’ (time), ‘Kraft’ (force), ‘Weg’ (distance), ‘Poren bar’ (pore water pressure), ‘Diff bar’ (change in pressure), and ‘Vol’ (change in volume). These are inputted into the Matlab program to generate the graphical results.

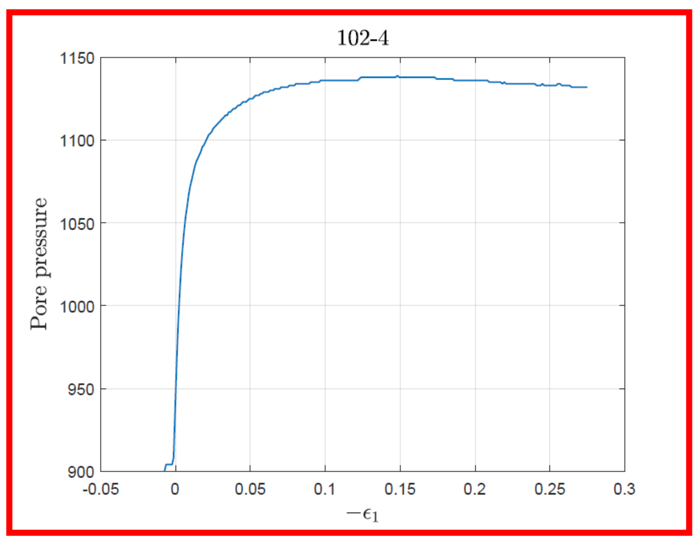
The following graphs were generated using Matlab from the data of various triaxial tests performed. The best results with the most precise data were chosen to represent the Critical State Line studied in this report, however, the inadequate graphs serve to hypothesize potential problems with the procedure of the experiment. These graphs are compared to the theoretical soil response values from David Muir Wood's book, as shown in **Figure 5**. The fourth figure with the red border identified to yield the best results.

Strain vs. pore water pressure graphs:



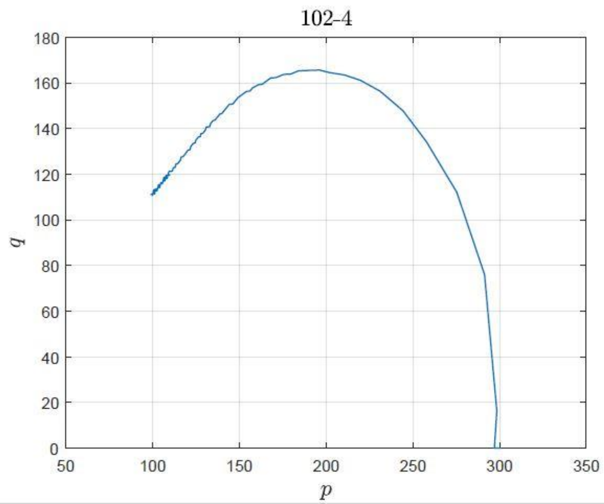


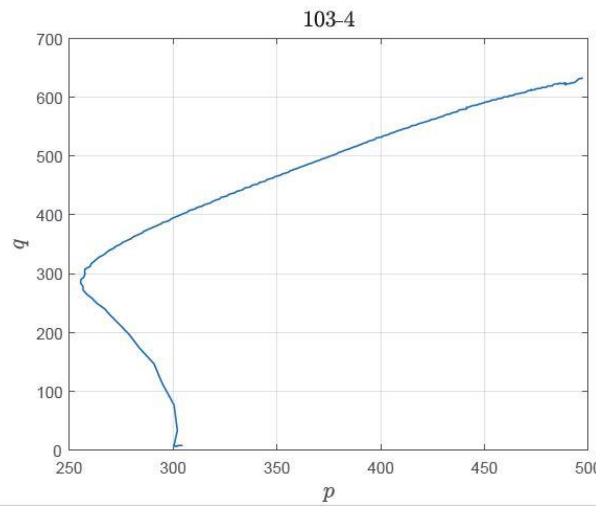


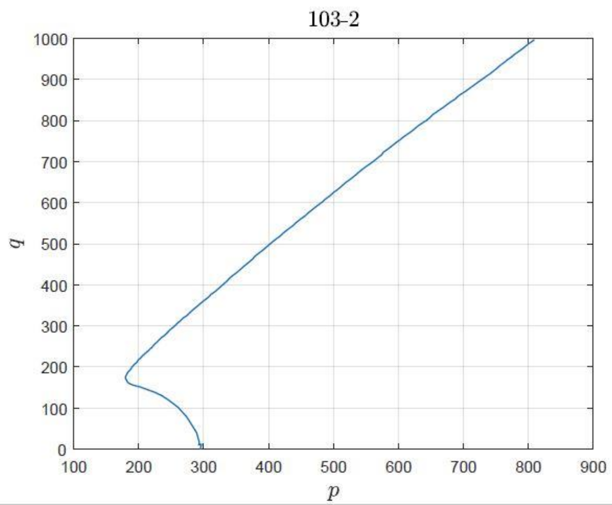


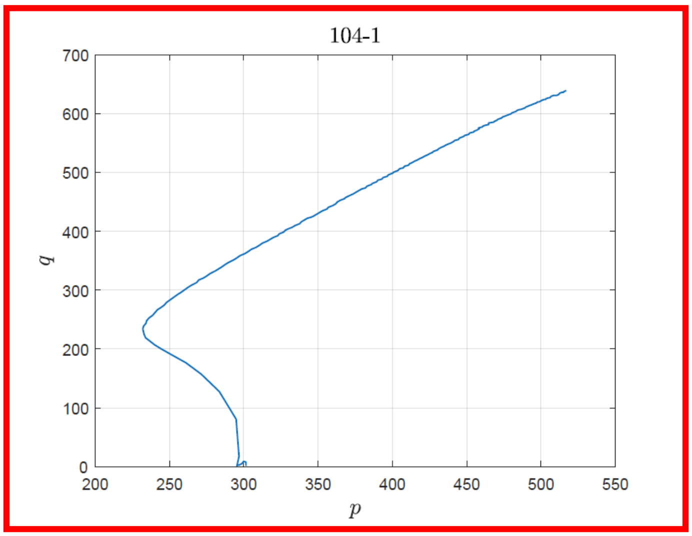
Graph 102-4 yields the best results and can be best compared to the theoretical CSL graph. Problems with the graphs 103-2, 103-4, and 104-1 may be due to the loading ram not applying constant loading during the test. Other issues may include: the loading being interrupted, or the loading Force (kN) not being inputted correctly in the database prior to commencing the test.

Deviatoric stress vs. effective stress:



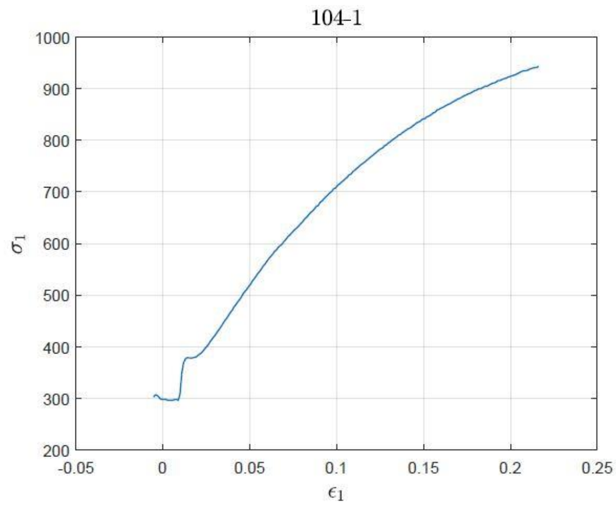


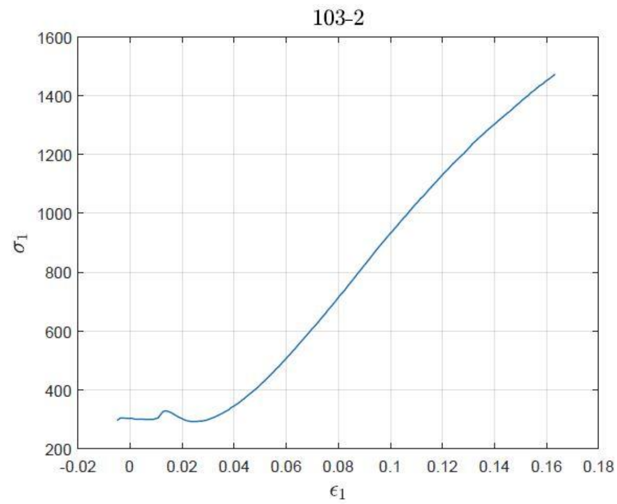


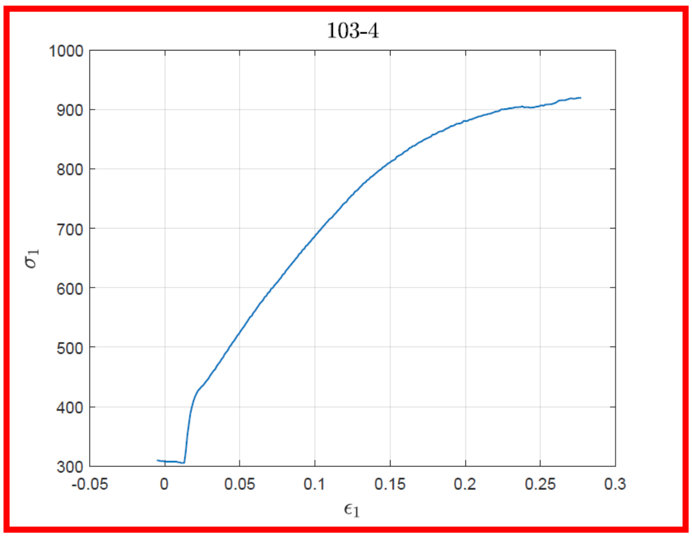


The graphs above all demonstrate the relationship between the deviatoric stress and the effective stress. Graph 102-4 yields the behavior of a loose sample, since the deviatoric stress turns in the negative direction. This sample is then classified as normally consolidated. Graphs 103-4, 103-2, and 104-1 show similar behavior initially, but as effective stress increases, the deviatoric stress turns in the positive direction. Thus, the sample is classified as dense and over consolidated.

Total stress vs. strain:

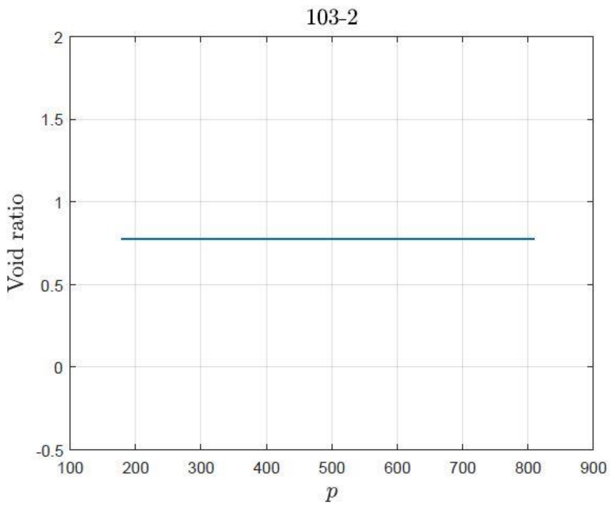


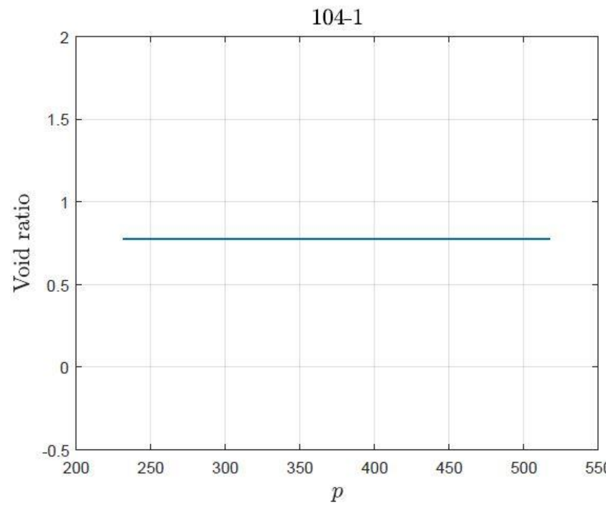


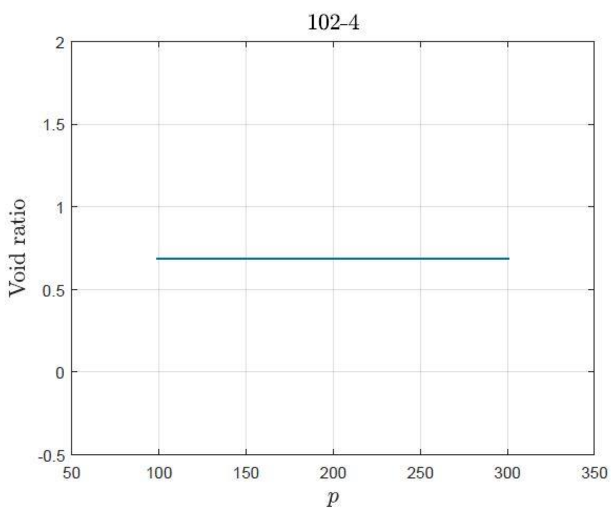


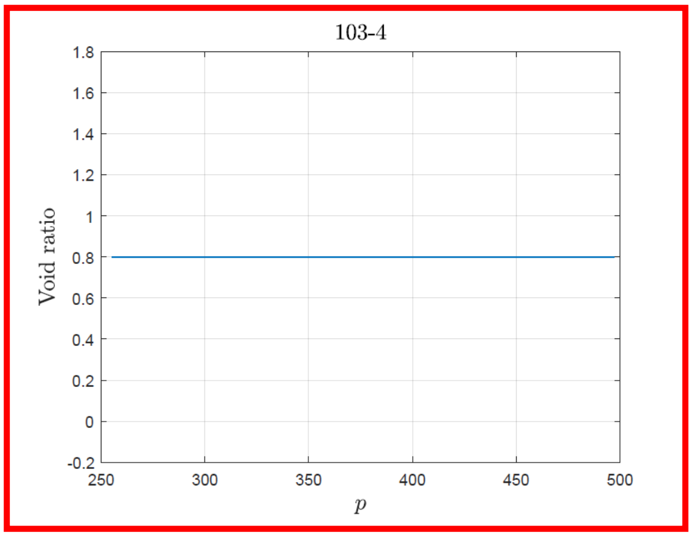
Graph 103-4 is used to show the stress train relationship of the soil during shearing. The curve seems to yield to a plateau state with no further increase in stress values, thus satisfying the critical state line concept. This behavior can also be seen in graph 104-1. This behavior was not established in graph 103-2. Problems reaching this behavior may have been due to loading not being applied adequately.

Effective pressure vs. Void Ratio









As expected, the void ratio remains unchanged while the deviatoric pressure increases.

References

Atkinson, John. *Foundations and Slopes: An Introduction to Applications of Critical State Soil Mechanics*. Wiley, 1981.

Das, Braja M. *Principles of Geotechnical Engineering*. Boston: PWS, 1997. Print.

Das, Braja M. *Soil Mechanics Laboratory Manual: 7th Revised Edition*. Oxford University Press Inc, 2009.

Kolymbas, Dimitrios. *Geotechnik Bodenmechanik, Grundbau Und Tunnelbau*. Springer Berlin, 2016.

Schofield, Andrew N., and Peter Wroth. *Critical State Soil Mechanics*. McGraw-Hill, 1968.

"Triaxial Automated System." *GDS Instruments*, 15 Oct. 2017, www.gdsinstruments.com/gds-products/triaxial-automated-system-load-frame-type.

Wood, David Muir. *Soil Behaviour and Critical State Soil Mechanics*. Cambridge Univ. Pr., 2007.