Undrained Triaxial Compression Tests

Laboratory Experiment # 10

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CE 4718   Intermediate Soil Mechanics -Group 2

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Purpose

The undrained triaxial strength tests are used to determine the shear strength of a soil sample that is not allowed to drain. The test will be completed on three unsaturated soil samples. The test results will be analyzed to determine the Mohr-Coulomb failure envelope, failure angle, shearing resistance, and Young's Modulus of Elasticity. This information can be used to predict how a soil would respond in field conditions, such as under an applied structural load or in the stability or instability of a slope.

Theory

The consolidated isotropic undrained triaxial test is the most common type of triaxial test. In this test, the saturated soil specimen is first consolidated by an all-around chamber fluid pressure, σ_3, which results in drainage. After the pore water pressure generated by the application of confining pressure is dissipated, the deviator stress, $\Delta\sigma_d$, on the specimen is increased to cause shear failure. During this phase of test, the drained line from the specimen is kept closed. Because drainage is not permitted, the pore water pressure, $\Delta u_{d,}$ will increase. During this test, simultaneous measurements of $\Delta\sigma_d$ and Δu_d are made. The increase in pore water pressure, $\Delta u_{d,}$ can be expressed in a non-dimensional form as

$$\bar{A} = \Delta u_d / \Delta\sigma_d \quad \text{where } \bar{A} = \text{Skempton's pore pressure parameter}$$

The general patterns of variation of $\Delta\sigma_d$ and Δu_d with axial strain for sand and clay soil are shown in figure below. In loose sand and normally consolidated clay, the pore water pressure increases with strain. In dense sand and overconsolidated clay, the pore water pressure increase with strain to a certain limit, beyond which it decreases and become negative (with respect to the atmospheric pressure). This decrease is because of a tendency of the soil to dilate.

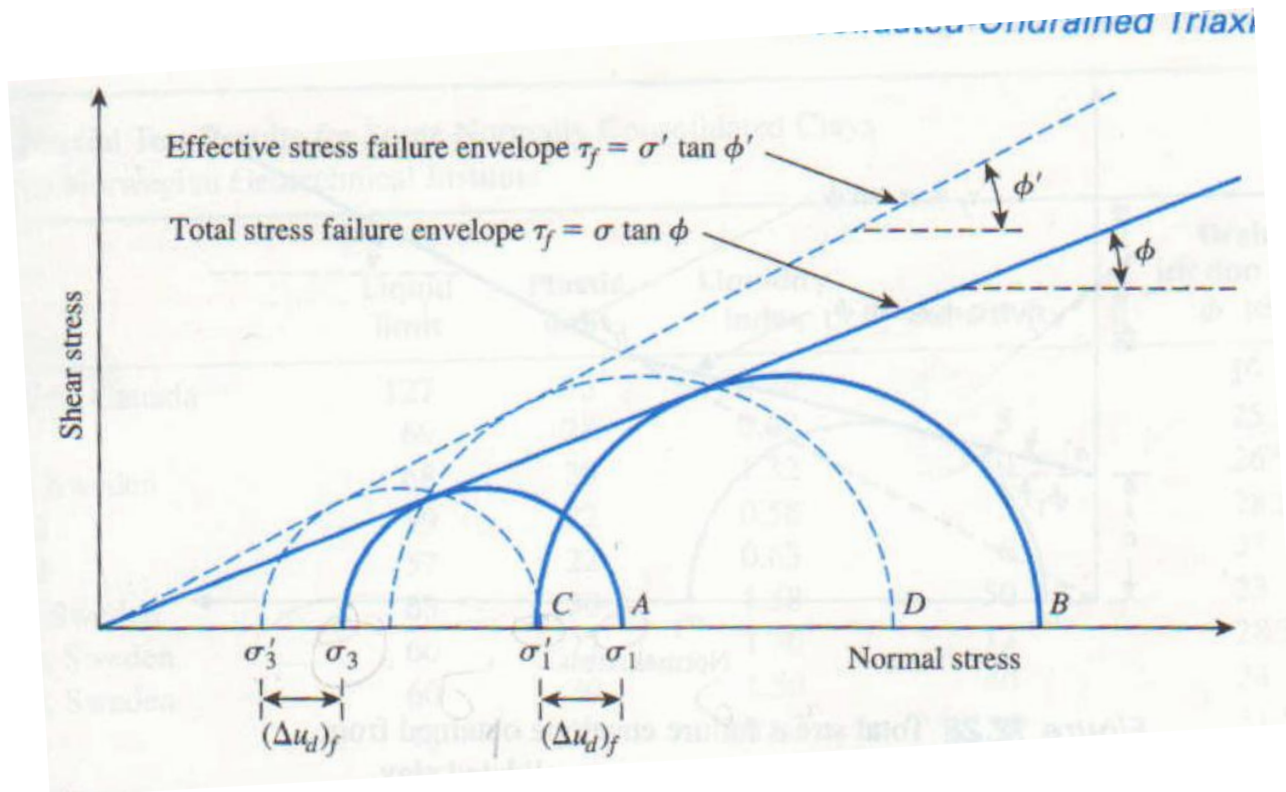
Unlike the consolidated-drained test, the total and effective principal stresses are not the same in the consolidated-undrained test. Because the pore water pressure at failure is measured in this test, the principle stresses may be analyzed as follows:

- Major principal stress at failure (total): $\sigma_{3\,+}(\Delta\sigma_d)_f = \sigma_1$
- Major principal stress at failure (effective) : $\sigma_{1\,-}(\Delta\sigma_u)_f = \sigma'_1$
- Minor principal stress at failure (total): σ_3
- Minor principal stress at failure (effective): $\sigma_{3\,-}(\Delta\sigma_u)_f = \sigma'_3$

In these equations, $(\Delta\sigma_u)_f$ = pore water pressure at failure. The preceding derivations show that

$$\sigma_1 - \sigma_3 = \sigma'_1 - \sigma'_3$$

Tests on several similar specimen with varying confining pressures may be conducted to determine the shear strength parameters. Figure below shows the total and effective stress Mohr's circles at failure obtained from consolidated –undrained triaxial test in sand and normal consolidated clay. Noted that A and B are two total stress Mohr's circles obtained from two test. C and D are the effective stress Mohr's circles corresponding to total stress circles A and B, respectively. The diameters of circles A and C are the same, similarly, the diameter of circles B and D are the same.



In figure, the total stress failure envelope can be obtained by drawing a line that touches all the total stress Mohr's circles. For sand and normally consolidated clays, this will be approximately a straight line passing through the origin and may be expressed by the equation: $\tau_f = \sigma \tan \Phi$ *(2.1)*
Where σ = *total stress*, ϕ = *the angle that the total stress failure envelope makes with the normal stress axis*, also known as ***the consolidated-undrained angle of shearing resistance***
Equation *(2.1)* is seldom used for practical considerations. For sand and normally consolidated clay, we can write:

$$\phi = \sin^{-1}\left(\frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}\right)$$

$$\phi' = \sin^{-1}\left(\frac{\sigma'_1 - \sigma'_3}{\sigma'_1 + \sigma'_3}\right)$$

$$= \sin^{-1}\left(\frac{[\sigma_1 - (\Delta ud)f] - [\sigma_3 - (\Delta ud)f]}{[\sigma_1 - (\Delta ud)f] + [\sigma_3 - (\Delta ud)f]}\right)$$

$$= \sin^{-1}\left(\frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3 - 2(\Delta ud)f}\right)$$

Again referring to the Figure, we see that the failure envelope that is tangent to all the effective stress Mohr's circles can be represented by the equation $\tau_f = \sigma' \tan \Phi'$, which is the same as that obtained from consolidated-drained tests.

In consolidated clays, the total stress failure envelope obtained from consolidated-undrained tests will take the shape shown in figure. The straight line a'b' is represented by the equation $\tau_f = c + \sigma \tan \Phi$ and the straight line b'c' follows the relationship given by $\tau_f = \sigma \tan$. The effective stress failure envelope drawn from the effective stress Mohr's circle will be similar to that so in figure.

Consolidated –drained test on clay soils take considerable time. For this reason, consolidated-undrained test can be conducted on such soils with pore pressure measurement to obtain the drained shear strength parameters. Because drainage is not allowed in these tests during the application of deviator stress, they can be performed quickly.

Skempton's pore water pressure parameter $\bar{A}$ was defined in Eq: $\bar{A} = \Delta u_d / \Delta\sigma_d$ at failure, the parameter $\bar{A}$ can be written as

$$\bar{A} = \bar{A}_f = (\Delta u_d)_f / (\Delta\sigma_d)_f$$

The general range of $\bar{A}_f$ values in most clay soils is as follows:

- Normally consolidated clays: 0.5 to 1
- Overconsolidated clays: -0.5 to 0

Soil Sample Description

Ottawa Sand

Material and Equipment Needed

- Right-circular cylindrical specimen of cohesive soil;
- load frame;
- pressure system and water source;
- triaxial cell;
- 2 0-rings;
- latex membrane;
- membrane stretcher;
- vacuum grease;
- deformation indicator graduated to 0.001 in.;
- load cell or proving ring;
- scale with precision of0.01 g;
- calipers;
- oven-safe moisture content container; and
- soil drying oven set at 110° ± 5° C

Procedure

1. Obtain a soil specimen from your instructor. Use calipers to measure the initial length (La) of the specimen. Measure the diameter near the top, middle, and bottom of the specimen, and calculate the average diameter (D_0) and average initial area (A_0).Also measure the moist mass of the specimen (M).

2. Apply a light coating of vacuum grease to the perimeter of the base and cap to help create a waterproof seal.

3. Place the soil specimen on the base, and place the cap on top of the specimen. Make sure that the piston hole in the cap faces up.

4. Place the membrane and two O-rings on the membrane stretcher, and apply light vacuum to the membrane stretcher tube to pull the membrane towards the inside wall of the membrane stretcher.

5. The following steps describe how to place the membrane on the soil specimen:

    a. Carefully lower the stretched membrane over the specimen without touching the specimen.
    b. Center the membrane on the specimen and release the vacuum to allow the membrane to constrict around the specimen.
    c. Gently pull the ends of the membrane over the base and cap so that the membrane surrounds the base, specimen, and cap without wrinkles.
    d. With the membrane stretcher still around the specimen, carefully roll the O-rings onto the membrane where the membrane contacts the base and cap. If the base and cap are machined with grooves, make sure that the O-rings are seated in the grooves.

6. The following steps describe how to assemble the triaxial cell:

    a. Place a light coating of vacuum grease on the O-rings in the pedestal and top.
    b. Place the cell wall on the pedestal, and make sure the pedestal and cell wall are properly seated against one another.
    c. Place the top on the cell wall, and make sure the cell wall and top are properly seated against one another.
    d. Slide the piston down into the hole in the cap. The tip of the piston should be far enough into the hole to prevent the specimen from tipping when the triaxial cell is moved, but should not be applying any load to the cap. Once in position, lock the piston in place by turning the locking screw in the top.
    e. Tighten each of the three cell bars a little bit at a time, alternating between bars to assure an intimate seal between the pedestal, cell wall, and top.

7. Open the vent valve in the top of the triaxial cell, and begin filling the triaxial cell with water from the pedestal valve. Shut off all valves to the triaxial cell when water emerges from the vent valve.

8. Position the triaxial cell in the load frame with the deformation indicator and load cell.

9. Apply the desired cell pressure σ_3 to the cell through the bottom valve. You will know the specimen is under pressure when the membrane appears to be in intimate contact with the specimen.

10. Release the piston by loosening the locking screw in the top of the triaxial cell, and zero the load cell. If a proving ring is used instead of a load cell, zero the dial gauge and record the proving ring constant K_p.

11. Zero the deformation indicator. If an analog dial gauge is used, record the dial gauge conversion factor K_L.

12. Manually advance the piston until the tip of the piston is seated against the cap. You will know it is seated when the load cell begins to indicate a slight load. Once the load cell indicates a slight load, stop advancing the piston.

13. Begin loading the specimen at a strain rate between 0.3-1.0%/min. ASTM D2850 suggests that initial readings be taken at 0.1 %, 0.2%, 0.3%, 0.4%, and 0.5%, 1.0%, 1.5%, 2.0%, 2.5%, and 3.0%. After that, readings should be taken at a strain interval of 1.0%. However, it may be necessary to take readings more frequently to accurately identify the peak applied load. Record your data on the Unconsolidated Undrained Triaxial Test Data Sheet, using additional sheets as needed. Load the specimen until ε_1 = 15%.

14. If your deformation indicator is a digital dial gauge, proximeter, or LVDT, your reading will be ΔL, and will be in units of length. If your deformation indicator is an analog dial gauge, your reading will be G_L, and will be in units of divisions. For analog dial gauges, ΔL is calculated as:

$$\Delta L = G_L K_L$$

15. If your load frame is configured with a load cell, your reading will be P, and will be in units of force. If your load frame is configured with a proving ring instead of a load cell, your reading will be G_P, and will be in units of divisions. For proving rings, P is calculated as:

$$P = G_P K_P$$

16. Plot Δσ versus ε_1. Identify the deviator stress at failure, $\Delta\sigma_f$, as either 1) the peak value of Δσ or 2) Δσ at ε_1 = 15%. Calculate σ_{1f} as follows:

$$\sigma_{1f} = \sigma_3 + \Delta\sigma_f$$

17. Place the specimen in a soil drying oven overnight and obtain the dry weight of the specimen, M_S, for weight-volume calculations.

18. Repeat Steps 1-17 for 3 or more additional specimens tested over a range of σ_3. Plot the Mohr circles for each specimen to define the Mohr-Coulomb failure envelope and S_u.

Laboratory Results:

Plots for deviator stress ($\Delta\sigma$) versus axial strain (ε_1) are shown below. These plots are used to calculate the deviator stress at failure (σ_{1f}) for the soil. This is identified as the maximum axial stress endured by the sample, or the stress pertaining to a strain of 15%. Since all samples appeared to fail at a strain lower than 15%, the maximum stresses are used. These stresses are used to calculate the principle stresses as follows:

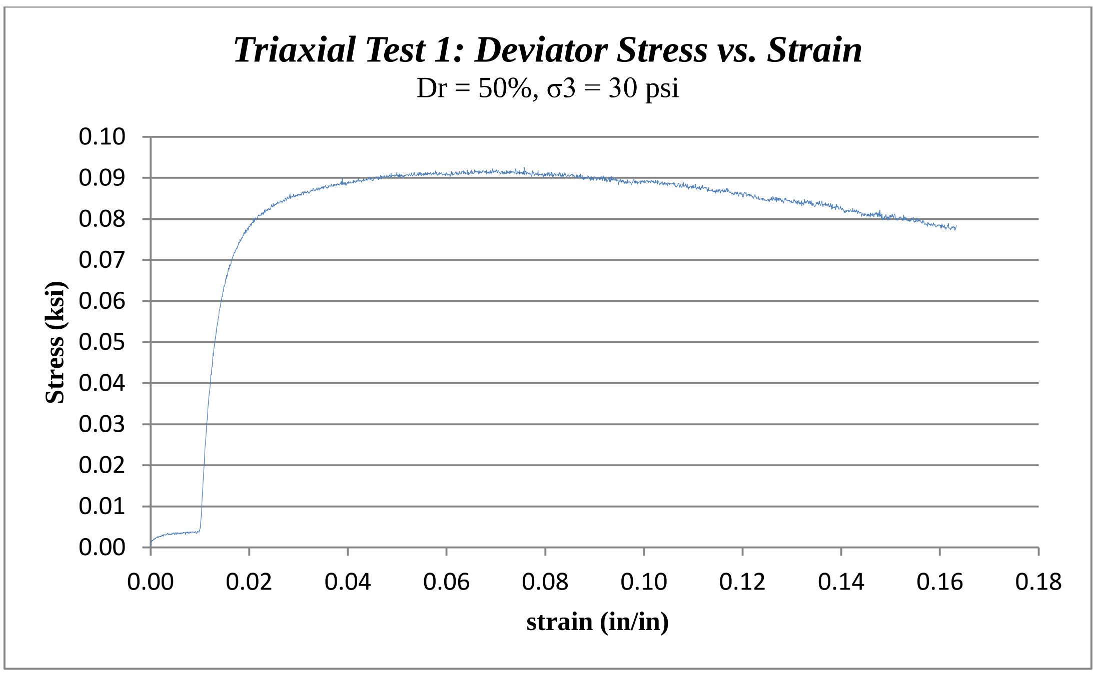
$$\sigma_{1f} = \sigma_3 + \Delta\sigma_f$$

$$Where\ \Delta\sigma_f = deviator\ stress\ at\ failure$$

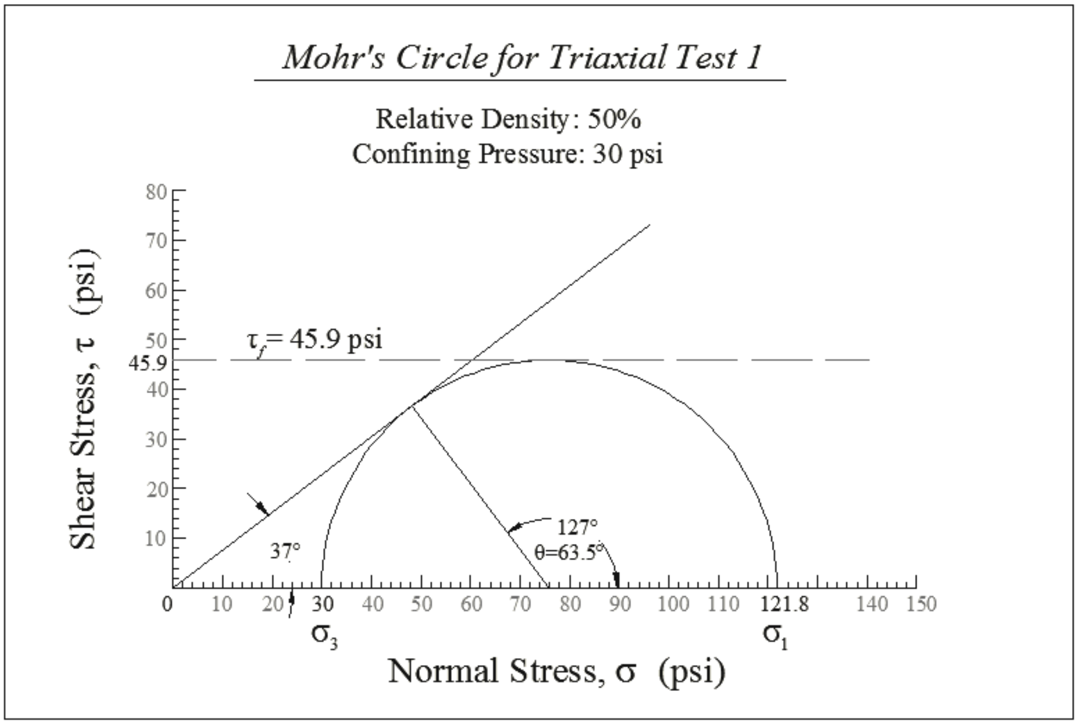
$$\sigma_3 = cell\ confining\ pressure$$

With that information, a Mohr's Circle can be constructed and the Mohr-Coulomb failure envelope may be established. These graphical procedures give the friction angle of the soil, as well as the maximum shear stress for each. These strength characteristics define the Mohr Coulomb failure envelope. Since the Ottawa sand used for our sample has little to no cohesion, it has zero shear strength when under no normal stress, thus the envelope crosses the origin, and extends linearly. The laboratory results are given below:

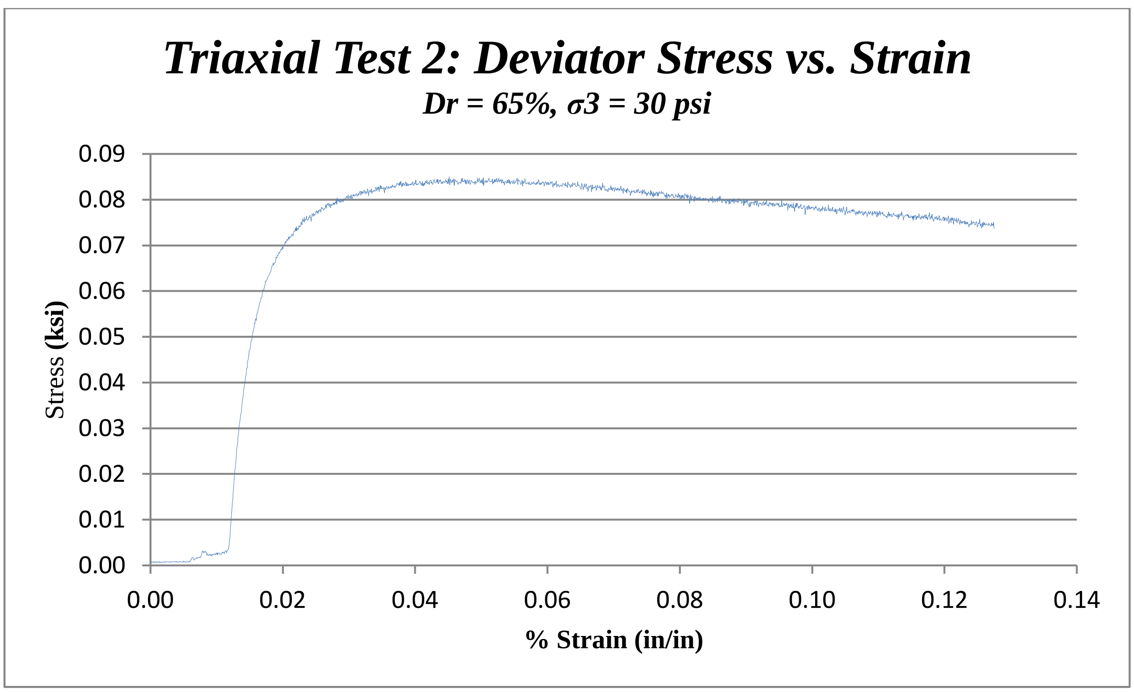
- *Test 1 Data:*



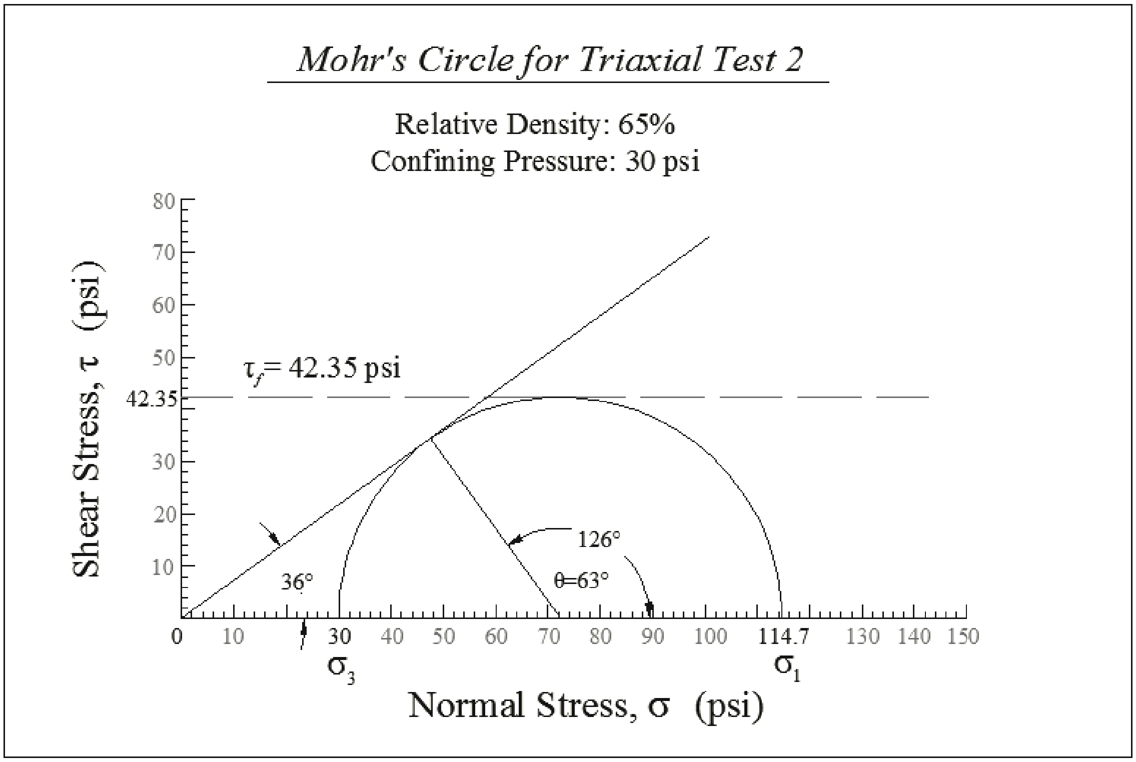
$$Young's\ \ Modulus\ E = 6.1\ ksi$$



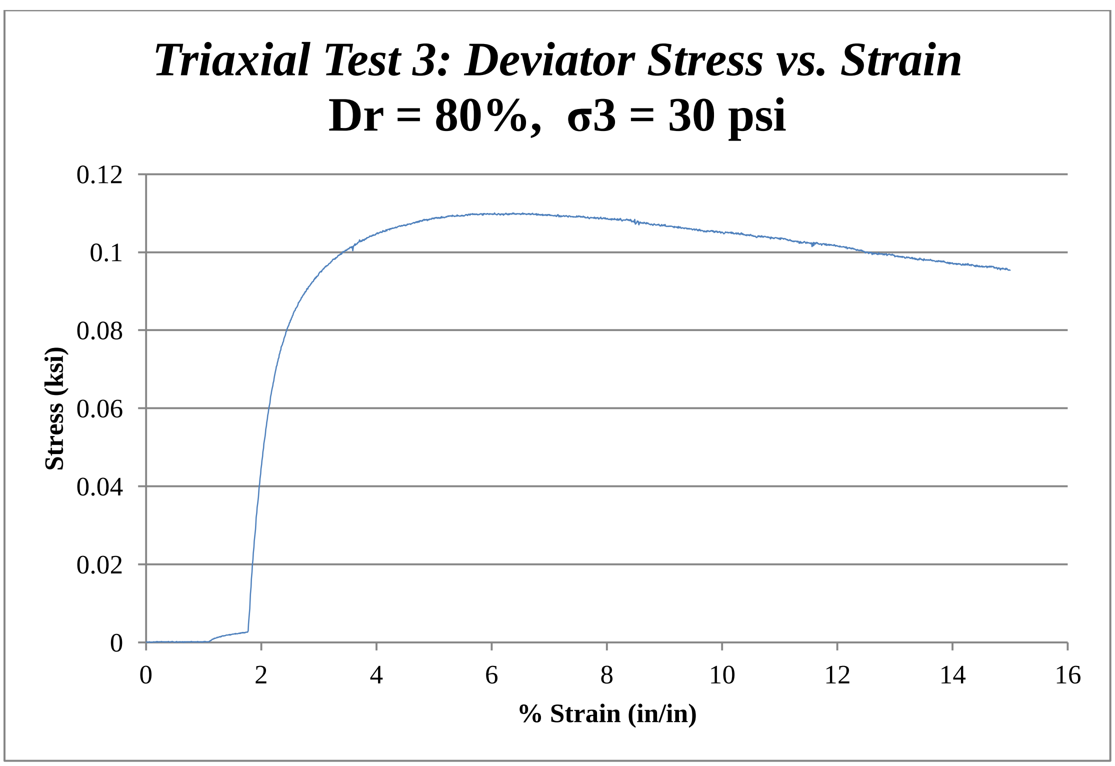
- *Test 2 Data*



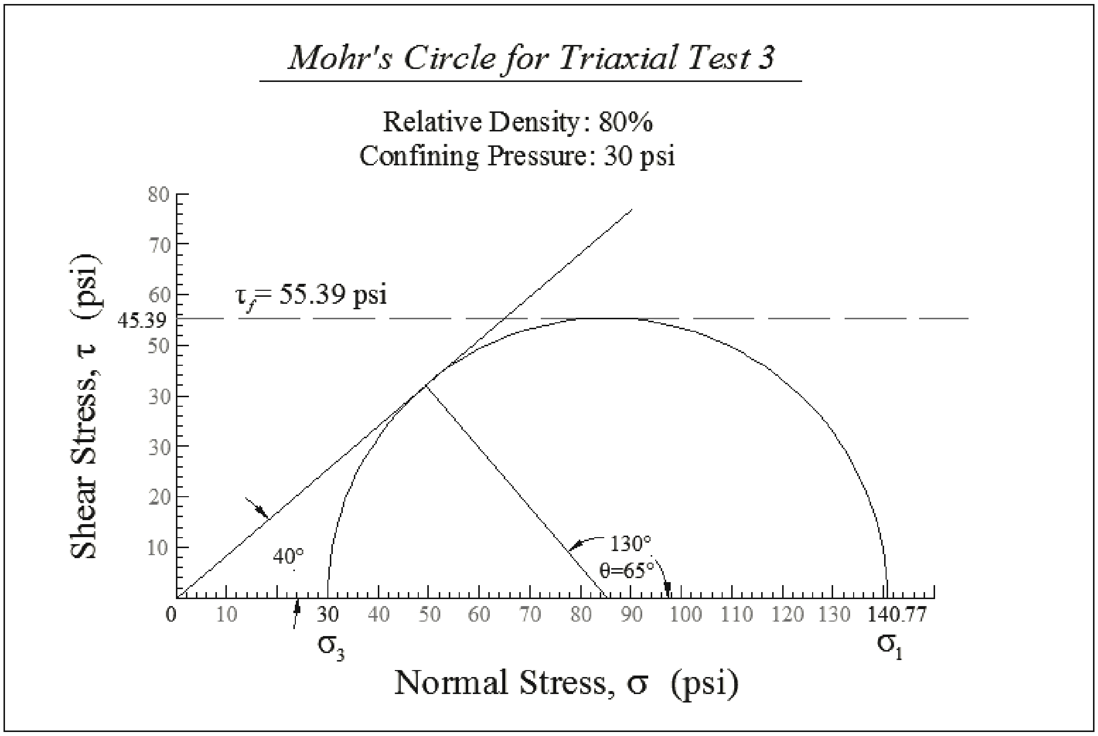
$$Young's\ \ Modulus\ E = 12.9\ ksi$$



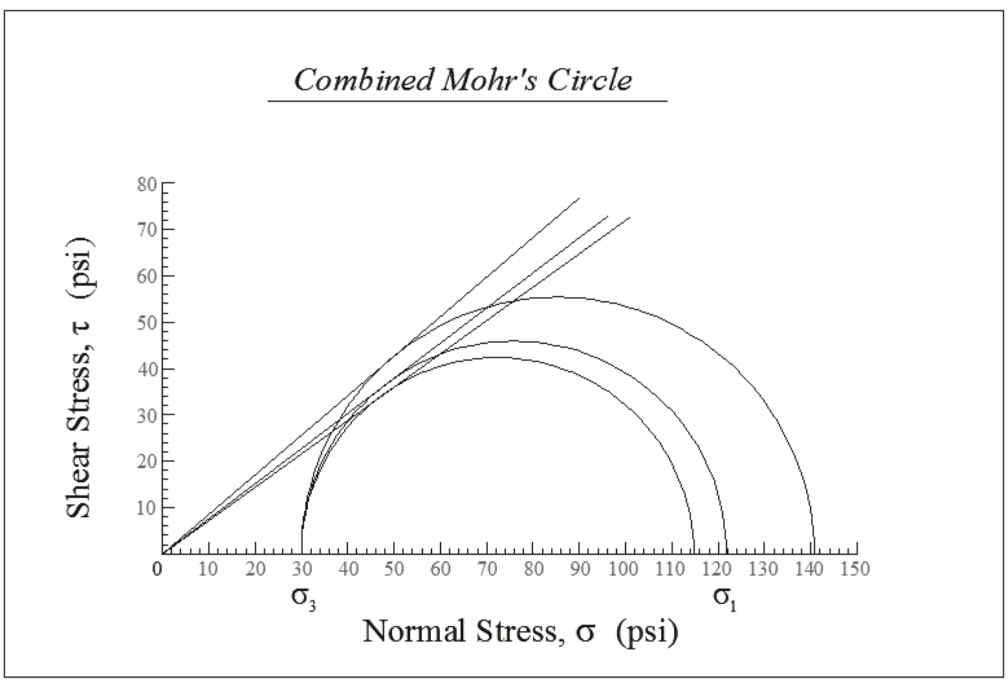
- *Test 3 Data:*



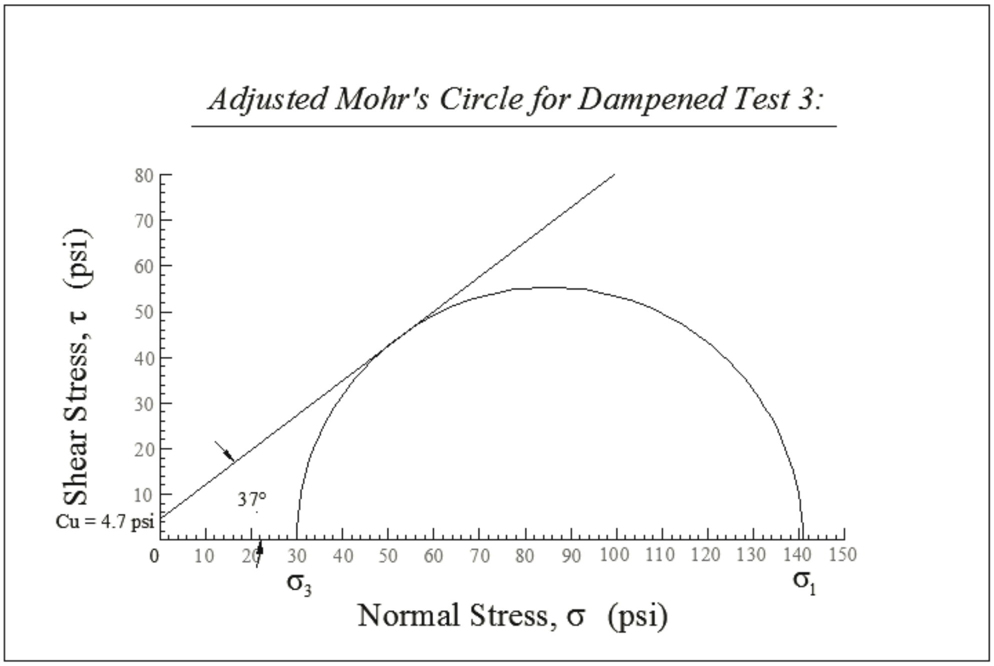
$$Young's\ \ Modulus\ E = \ 0.15\ ksi$$



To compare the results, an overlay image is used combining the results of the three tests. Test no. 3 exhibited the highest strength, but is also an outlier because it became moist during the test.



Interestingly, if the angle of the tangent line is adjusted so as to match the $36° - 37°$ range of the first two tests, the envelope does not pass through the origin. The ordinate at which the normal stress is zero is defined as the undrained shear strength of the soil. Because of surface tension, the sand may have obtained some small amount cohesion. By scale, the graph below shows that the Ottawa sand, when moistened, may have had undrained shear strength of about 5 psi.



Discussion of Error:

Apart from issues with calibration and accurate reading of some measurement tools, laboratory procedures were executed very well during this laboratory. Tests 1 and 2 were successful and provided data with good correlation, however, test 3 is a definite outlier due to the improper seal around the specimen. This allowed the sample to become partially saturated during the test, which appears to have given it a higher shear stress at failure, and a steeper friction angle. This is to be expected due to the effects of surface tension from the presence of water.

A second error observed may have to do with improver calculation of relative density for the first or second sample. As the tests were performed on progressively denser samples, the shear stress at failure should increase progressively also. Interestingly, the initial test, compacted to a relative density of 50% showed more strength than the second sample, which was compacted at 65%. The only logical explanation for this behavior is that an error was made in the initial calculation of relative density. If there was a mathematical error, an improper weight of sand would be given, and an inaccurate relative density would be obtained.

Conclusion:

The tests were completed on progressively denser soil samples. The undrained shear strength for a dry compacted soil specimen may range to over 50 psi. The first two tests had values of 45.9 psi ad 42.35psi. While the third value is 55.39 psi, due partial saturation of the sample, the results may not be valid. The angle of failure was approximately 36.5º. Had the third sample not been contaminated, it would be expected that it would have an angle of failure near this value as well, instead had a slightly higher value of 40º. The modulus of elasticity was showing signs of steadily increasing with an increase in sample density, however the third test again must be removed from analysis as it was partially saturated and the strain rate of an unsaturated soil is faster than that of a partially saturated sample (air permeability is higher than water permeability). These results will provide an estimate of how the same soil sample will react in field conditions, however being in ideal lab conditions the values will differ slightly.