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Title: Test Procedures for Low-Confining Stress, Multistage Triaxial Testing of Compacted Cohesive Soils

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The use of the triaxial test to characterize the strength of soils for civil engineering applications is widespread. These tests are typically conducted with confining stresses in excess of 5 psi. To characterize the strength of a soil located in the upper layers of the subgrade of an aggregate surfaced road it is necessary to conduct triaxial tests with low confining stresses (5 psi or less).

The development of a method for conducting multistage, consolidated undrained (CU) tests at low confining stresses (0.5 to 5.0 psi), with back pressure saturation, is presented. Aspects of the test procedure that require special attention are described and recommendations are made including:

1. Compaction of the sample in an atmosphere of carbon dioxide reduces the time and pressure required to complete back pressure saturation.

2. Seepage force related pore pressures develop during sample flooding. Zeroing of the effective stress transducer should be completed prior to sample flooding so that it is certain that zero effective stress conditions are present.
3. Back pressure saturation is simplified by the use of a slave regulator (air loaded pressure regulator) that maintains a nearly constant pressure differential between the cell pressure and the back pressure.
4. The stress path method of interpretation is an essential part of multistage triaxial testing. This method simplifies the decision of when to stop each shear stage and the determination of shear strength parameters.
5. The use of a computer data acquisition system that processes data in real time and visually presents test progress simplifies the completion of multistage triaxial tests.

Test Procedures for Low-Confining Stress, Multistage Triaxial Testing of Compacted Cohesive Soils

by

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TEST PROCEDURES FOR LOW-CONFINING STRESS, MULTISTAGE TRIAXIAL TESTING OF COMPACTED COHESIVE SOILS

1 INTRODUCTION

The use of the triaxial test to characterize the strength of soils for civil engineering applications is widespread. Typically, these tests are conducted at confining stresses in excess of 5 psi. However, to characterize the strength of a soil located in the upper layers of the subgrade of an aggregate surfaced forest road these typically used confining stresses are too high. Thus, it is necessary to conduct triaxial tests with low confining stresses (less than 5 psi) to properly examine the strengths of these subgrade soils.

This desire to conduct low confining stress triaxial tests generates a need to carefully examine traditional test methods and equipment to insure that they are appropriate. Errors that can be tolerated and perhaps not even detected at higher confining stresses may become significant in low confining stress tests. The conduct of these tests can be aided by the use of electronic devices to measure deformation, loads, and pressures. Computer data acquisition further aides testing by providing potential for more rapid and accurate recording of data.

This work emerged as a part of a larger project investigating the behavior of subgrades in aggregate surfaced roads. The desired results were to develop a system for conducting triaxial tests with the following features:

1. The ability to conduct single- or multistage, consolidated-undrained (CU) tests. [Since the components to be measured are the same, consolidated-drained (CD) tests can also be conducted with slight modifications to the procedure.]
2. The ability to handle low confining stresses (0.5 to 5.0 psi) and back pressure saturation.
3. A computer data acquisition system that will process data in real-time and visually present the progress of the consolidation and shear phases of the test.

The discussion that follows describes the resulting test system, its development, and presents examples of data that have been obtained from multistaged, low confining stress, consolidated-undrained tests on a compacted forest soil.

2 LITERATURE REVIEW

The triaxial test has been extensively used to quantify the strength of soils. Considerable literature exists discussing test equipment, methodology, and limitations, as well as the application of the triaxial test to specific problems. Of specific interest to this discussion is testing of compacted cohesive soils, back pressure saturation, multistage testing, instrumentation, and data acquisition.

2.1 Triaxial Testing of Compacted Cohesive Soils

Seed et al. (1960) described a series of tests conducted to investigate the shear strength of compacted clays. The tests were conducted on both partially saturated and saturated samples. The research investigated the effects of different types of loading and the strength characteristics of the compacted cohesive soils including the influence of compaction method and conditions, pore pressures and the age of the sample. Tests were conducted at confining stresses of 15 psi to 60 psi.

They presented a variety of results including:

a) For a given soil, within the range of water contents and densities tested, the effective angle of friction for constant composition, φ_c, remains essentially constant. The effective cohesion for constant composition, c_c, is low, and it appears that c_c decreases as the water content at failure or the void ratio at failure increases. [φ_c and c_c are terms used by Seed et al. to refer to strength parameters obtained under conditions that produce the same void ratio at failure.]

b) It appears that differences in structure and compaction method may cause significant differences in stress-strain characteristics, pore water pressures and effective stresses in compacted clays. However, they cause only small variations in maximum principal stress ratios and little change in effective stress strength parameters.

c) Wide variations in the effect of the rate of loading on strength (total stress) may be expected for different types of compacted clays and for the same clay prepared under a variety of conditions. The effect of the rate of loading on strength, in terms of effective stresses, was not investigated.

d) Where long term effects and compositional changes are controlled, effective stress soil strength is independent of variations in the soil structure.

Lovell and Johnson (1979, 1981) conducted a series of consolidated undrained tests with pore pressure measurements on compacted highly plastic clay. Confining stresses of 10, 20, 30, and 40 psi were used. The purpose of this investigation was to determine the effect of compaction conditions on effective stress friction angle, Skempton's A-parameter at failure, volumetric strain upon saturation, and undrained shear strength. The following conclusions were reached based on this study:

a) Skempton's A-parameter at failure (A_f) is largely a function of the final void ratio attained with the selected consolidation pressure, and is probably related to the degree of overconsolidation produced by compaction. [Degree of overconsolidation is terminology used by Lovell and Johnson. Because compacted soils are not actually overconsolidated, alternative terminology, such as, degree of dilative behavior is probably more appropriate.]

b) The effective stress friction angle, φ', is essentially constant over the range of compaction conditions investigated.

c) The effective stress strength intercept, c', is largely a function of the final void ratio attained with a selected consolidation pressure, and is probably related to overconsolidation ratio and the maximum past stress created in the soil by compaction. [Overconsolidation ratio and maximum past stress is the terminology used by Lovell and Johnson. Alternative terminology is probably more appropriate because compacted soils are not actually overconsolidated.]

Low-confining stress triaxial testing is of special concern to this discussion. Unfortunately, the aspects of the triaxial testing that are unique to low-confining stress testing (less than 5 psi) are not well documented in the literature. Further, it is unclear which elements of triaxial testing (5-30 psi confining stress) discussed in conjunction with traditional civil engineering problems are pertinent to this type of testing.

2.2 Back Pressure Saturation

If it is desired to conduct tests on saturated soils, full saturation of the specimen is important to eliminate errors in the measurement of volume change and pore pressure. Back pressure is a commonly used technique to achieve saturation (Bishop and Henkel, 1962; Baldi et al, 1988; Bishop et al, 1960; Lowe and Johnson, 1960; Black and Lee, 1973; Poulos, 1981; Rad and Clough, 1984; Germaine and Ladd, 1988).

In general, the procedure that has been adopted is to begin by flushing or flooding the triaxial system to remove any air bubbles. The actual back pressure saturation is accomplished by increasing both the cell and pore pressure simultaneously in a series of increments of 5-10 psi, allowing time for pore pressure equalization at each increment. Lovell and Johnson (1979) report incrementing back pressure 10 psi every two hours while maintaining a 10 psi differential between cell and back pressure for specimens to be consolidated at more than 20 psi. For 10 psi consolidation pressures, back pressure was incremented 7 psi every two hours while maintaining a 3 psi differential between cell and back pressure.

The literature is unclear with respect to selection of the size of the increment. However, it is apparent the increment should be selected to minimize the development of excess pore pressures prior to equalization and to insure that the sample doesn't experience excessive consolidation as a result of differences between the cell and the pore pressure and delays in the flow of water into the sample.

Saturation occurs by the movement of water into the sample to replace volume formerly occupied by gases that were compressed at elevated pressures and gases that have gone into solution in the pore water. The rate at which back pressure saturation proceeds is a function of many things including: soil type, initial degree of saturation, and the amount of back pressure applied to the sample (Black and Lee, 1973).

The applied back pressure gradually reduces the size of air bubbles by compressing them (according to Boyle's Law) and by dissolving them in the pore water (according to Henry's Law).

The ability of a gas to dissolve in water is described in part by Henry's constant, the higher the value the higher the solubility of the gas in water. If the pore air in the sample can be replaced with a gas of higher solubility, the pressure and the time required for back pressure saturation can be reduced. Mulilis et al. (1975) described a method for achieving saturation at low back pressures by preparing samples in an atmosphere of carbon dioxide (CO_2). Henry's Constant for carbon dioxide is much larger than for air, thus, a much larger amount of carbon dioxide can be dissolved in water, compared to air, at lower pressures. They reported achieving saturation, in sands, at back pressures less than 20 psi compared to back pressures of 60 to 200 psi that have been reported elsewhere (Lowe and Johnson, 1960; Black and Lee, 1973).

2.3 Multistage Testing

In traditional triaxial testing, several samples, each of which has undergone a consolidation and shear phase at a given state of stress, are required to produce a failure envelope. Often, it may be difficult or impossible to obtain or make a homogeneous set of specimens upon which to conduct these tests. Multistage testing uses one sample, consolidated and sheared several times, to obtain the information required to produce a failure envelope.

Soranzo (1988) describes a procedure for conducting isotropically consolidated, undrained multistage triaxial compression tests. Sample preparation and saturation stages are conducted in the same manner as for traditional, isotropically consolidated, undrained triaxial compression tests. During the shear stage, the sample is subjected to strain, producing a significant shear stress. The deviator stress is then released and the sample is subjected to an increase in confining or consolidation stress prior to the next shear stage. Soranzo suggests using a double or triple set of consolidation, and shear stages for this type of test, if peak shear stress is not expected for axial strains of at least 8%. It is not clear why this level of axial strain is specified. After each consolidation stage, the new sample height and volume must be determined and used for subsequent shear stages.

Soranzo suggests that the first shear stage should be conducted to at least 5% strain and says that the method is not generally applicable for soils that reach failure at very small axial strain. This suggestion appears to be related to difficulties in determining when failure has occurred for each stage, although no particulars were presented.

Kenny and Watson (1961) investigated the use of the multistage triaxial test for determining effective stress strength parameters in saturated natural and remolded soils. For undrained tests, axial strain was applied at a constant rate of approximately 4 percent per hour. After the completion of any undrained test, the load was removed and constant pore pressure was allowed to be reached before any further testing.

Kenny and Watson define failure as occurring when the stress path is tangent to the maximum effective stress shear strength envelope. For a multistage test, they found that it was not critical to know exactly where this point lies, but instead it was more important to know that the test had been carried far enough to pass the tangency point and fully mobilize c' and φ'. They found, for most of the undrained tests conducted, that failure occurred at axial strains less than 15 percent and generally less than 10 percent. Consolidation pressures used for the multistage tests were generally 10, 30, and 60 psi.

They concluded that, for undrained tests, it is possible to fully mobilize c' and φ' at least two and possibly three times on the same sample, with different confining stresses, if the test equipment is capable of strains of 25 percent or more. They report that multistage and conventional undrained tests result in similar values for c' and φ' for all the soils investigated, however the shape of the stress paths were not generally similar between the two test types. This implies that the induced pore pressures are different between the two tests.

Schoenemann and Pyles (1988) point out that the stress paths imposed on a specimen in multistage testing can be significantly influenced by the test procedure. They found, in tests where the deviator stress was completely removed between stages, a substantial positive excess pore pressure was generated, creating effective confining stresses considerably below those felt by the specimen during the preceding consolidation and shear phase. Specimens also experience considerable axial rebound when the deviator stress is removed, so that strain at the beginning of subsequent shear phases was 1 to 3% less than at the end of the previous shear phase. Axial rebound occurred while the drainage lines were closed (implying constant volume) so that radial compression must also be occurring.

Schoenemann and Pyles suggest a modified test procedure to reduce the impact of these problems. In this procedure, the specimen is anisotropically consolidated using a stress-controlled load frame, and is sheared using a strain-controlled load frame. An important element of the procedure was to lock the piston between the end of shear and the beginning of a new consolidation phase. These test modifications prevent axial extension, resulting in an ending strain value that matches the strain experience by the specimen. In addition, stress path loops are also reduced.

2.4 Instrumentation and Data Acquisition

One of the most complete discussions of triaxial test equipment and methodologies was presented by Bishop and Henkel (1962). Although many changes have occurred to the equipment used to conduct triaxial tests, the descriptions of test equipment and procedures are still relevant.

The use of load cells and various electronic transducers to make the measurements required during triaxial testing of soils is becoming routine. Coupling of this electronic equipment with computer controlled data acquisition allows for rapid collection of data without the human errors that may be present with manual collection and recording of data. There is considerable information in the literature about instrumentation and data acquisition, including the work discussed below.

Tatsuoka (1988), as a part of his discussion of triaxial test systems for cohesionless soils, discusses instrumentation that is also suitable for use in the testing of cohesive soils. He recommends the use of a triaxial apparatus equipped with a high capacity (HC) and a low capacity (LC) liquid-liquid differential pressure transducers (DPT) as illustrated in Figure 1.

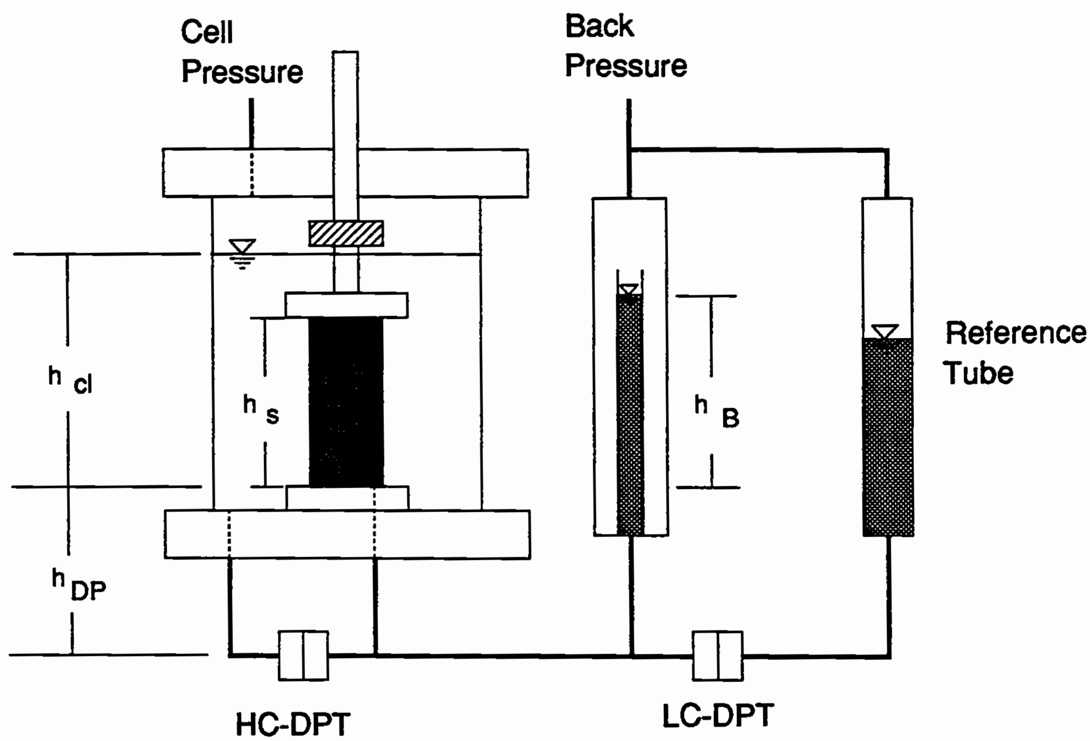

Figure 1. Triaxial Apparatus Using an HC-DPT and an LC-DPT. After Tatsuoka (1988).

The HC-DPT is used to directly measure the effective confining stress (σ'_3). In this configuration, the pressure on the high-pressure side of the HC-DPT is equal to the cell pressure plus the height of the water column above the transducer

$$p_h = \sigma_c + (h_{cl} + h_{DP})\gamma_w$$

and the pressure on the low-pressure side of the HC-DPT is equal to the back pressure plus the height of the water column above the transducer

$$u = \sigma_{BP} + (h_B + h_{DP})\gamma_w$$

The differential pressure (DP) measured by the HC-DPT is the difference between the high and low side pressures or

$$DP = p_h - u$$

which is simply the effective confining stress. This method is considered superior, especially for tests at high back pressure, because the resolution of DP is independent of the back pressure and the accuracy of σ'_c decreases when p_h and u are measured separately.

The LC-DPT is used to measure volume change by monitoring changes in the height of a water column and is appropriate for a back pressure saturated sample. Two cautions are expressed with respect to this method. First, evaporation of water from the burette may induce error, especially in long-duration tests. The rate of evaporation is reduced by the application of a back pressure. This problem may also effectively be eliminated by the use of a reference tube parallel to the burette so that the effects of evaporation are canceled out. Second, a hysteresis effect may occur due to friction between the water and the tube wall. This effect is eliminated by reducing the tube length between the burette and the LC-DPT or by increasing the tube diameter.

The use of internal load cells for the measurement of axial loads in triaxial testing is also discussed by Tatsuoka. He suggests the use of load cells that satisfy the following: (1) high linearity, (2) small hysteresis,

(3) small zero drift, (4) high resolution, (5) low compliance, (6) temperature insensitive, (7) sealed against pressurized water, (8) compact, (9) insensitive to hydrostatic pressure, and (10) simple to calibrate.

Schoenemann and Pyles (1986, 1988) recommend the use of a continuous reading load cell for low stress testing where piston friction may significant. The continuous reading feature of a load cell makes it possible to monitor stress changes and thus stress path throughout a test, even if it is necessary to move the cell from a consolidation bench to a loading frame. This type of load cell also makes it possible to determine the stress path between the end of a stage and the end of consolidation for the next stage in multistage triaxial tests.

3 TEST SYSTEM

The test system has three major components: the triaxial cell and load frame, the control board, and the data acquisition system.

3.1 Triaxial Cell and Load Frame

Strain controlled loading was applied using a ELE load frame (maximum load 10 kN). Strain rates are adjustable over the range 0.00064 mm/minute to 1.50 mm/minute by changing the gear configuration on the load frame.

The triaxial cell used was manufactured by Soil Engineering Equipment Company. The interior of the cell is approximately 5.4 inches in diameter and is adjustable in height depending on the length of the support bars and of the lucite cylinder. For this application, a height of approximately 1.2 feet was used. In this configuration, the cell will accommodate samples up to about 6 inches tall. The size of the end caps controls the sample diameter that can be tested, for this application 2.9 inch diameter end caps were used.

The triaxial cell (Figure 2) is equipped with three valves that open to the interior of the cell, two on the cell top and one on the cell bottom; and two valves that open to the sample through the end caps, one on the sample top and one on the sample bottom. To minimize pressure leakage around the load piston, the cell is equipped with a small chamber surrounding the load piston above the main cell that can be pressurized to the same pressure as the cell. The load piston is also sealed using a quad-ring at the top of the leakage chamber.

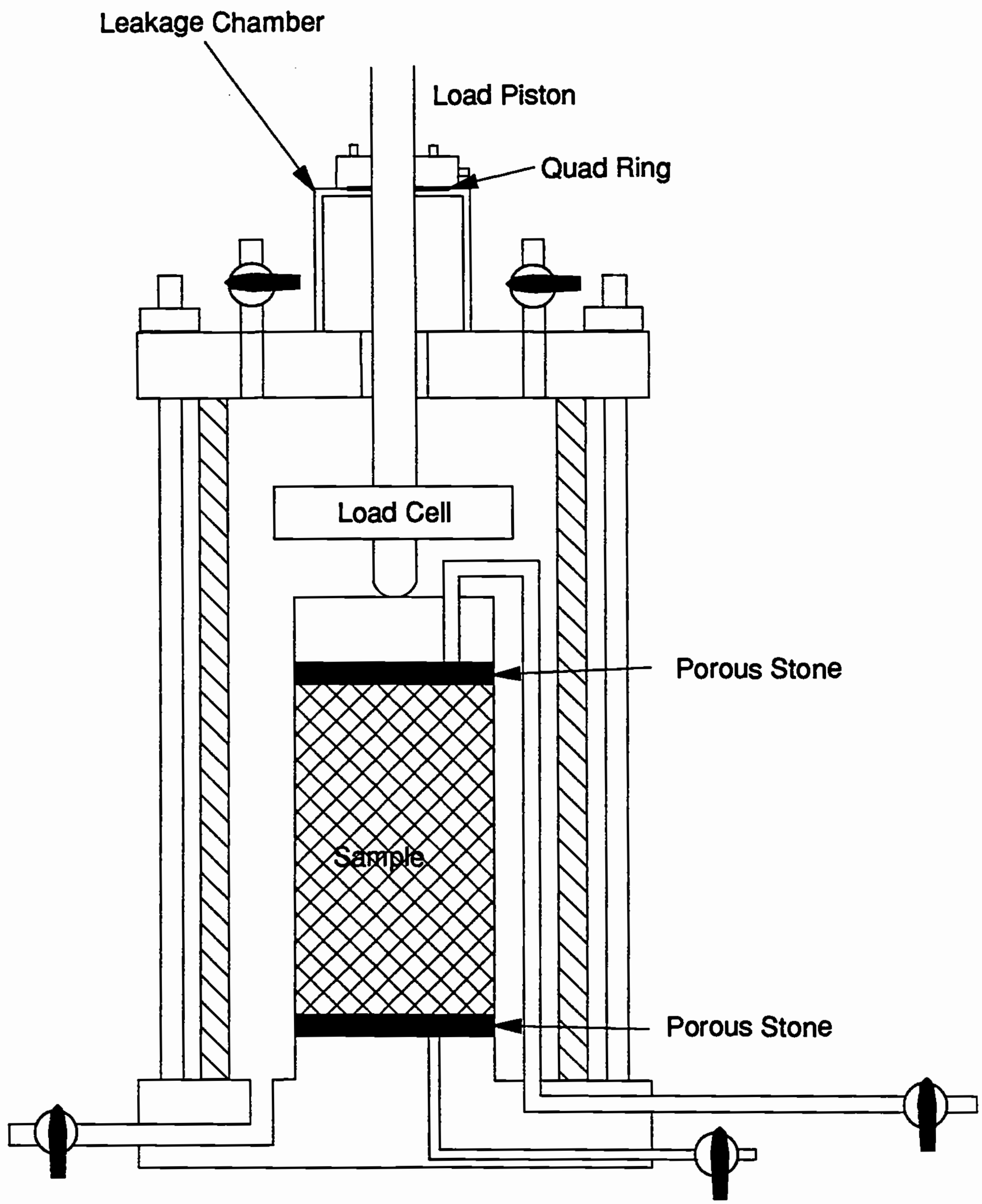

Figure 2. Triaxial Cell.

The cell was modified to use a continuously reading internal electronic load cell (Schoenemann and Pyles, 1988). The electronic load cell uses strain gauges to measure the deviator load applied to the sample. Because the load cell is located inside the cell, the effects of piston friction are eliminated from the load reading. However, because of its design and location, the load cell interprets changes in cell pressure as changes in load, thus, the load reading must be adjusted when cell pressure is changed during testing.

Pressure is applied to the water filled cell by the application of air pressure to the top of a tube, partially filled with water, called the air-water interface tube (AWIT). The bottom of this tube is connected to the cell bottom. The top of the AWIT is connected to pressure with a tee so that the same pressure can also be applied to the leakage chamber surrounding the load piston at the top of the cell, as illustrated in Figure 3.

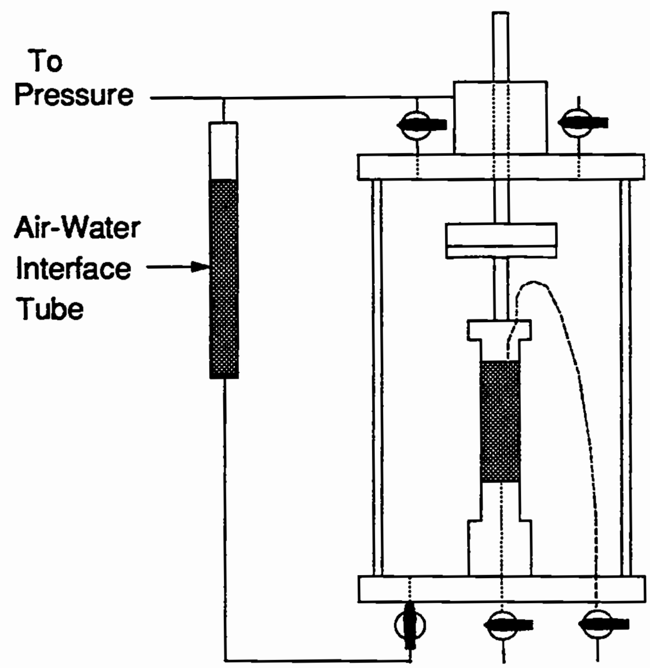

Figure 3. Triaxial Cell and Air-Water Interface Tube.

3.2 Triaxial Control Board

Pressure levels are controlled by a series of regulators that are a part of the triaxial control board (Figure 4). Air from the compressed air source is pre-regulated (Bellofram Type 10-B, 2-120 psi regulator) to a constant value above any pressures that will be required during testing, typically 65-70 psi.

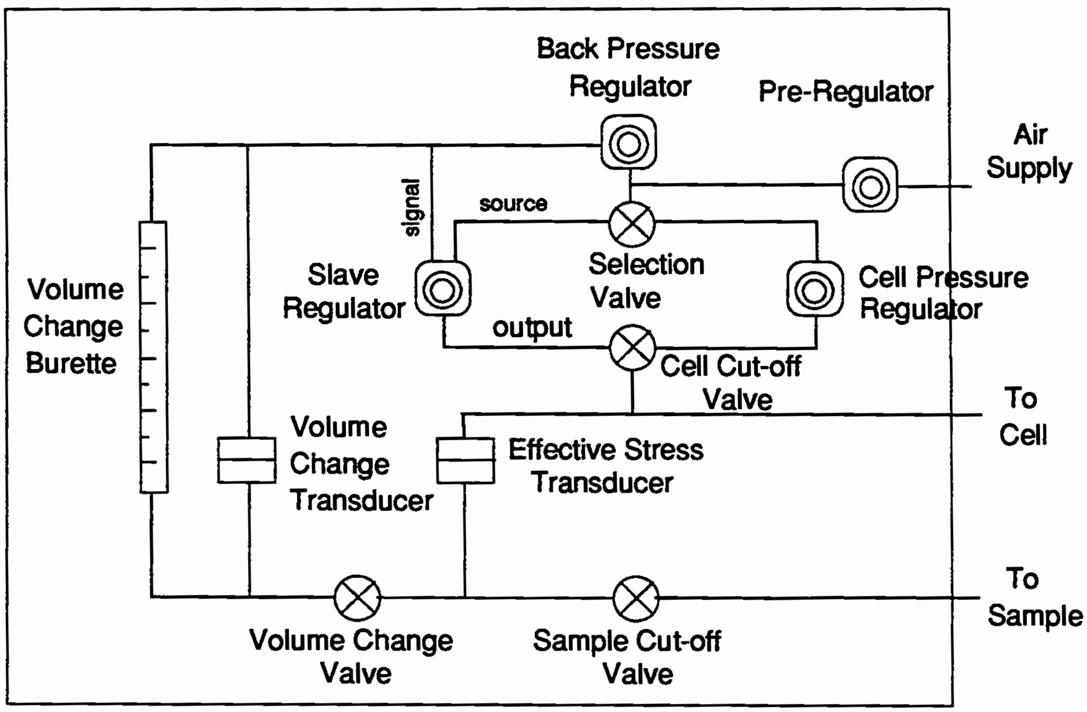

Figure 4. Triaxial Control Board.

By changing the position of the selection valve and the cell cut-off valve, two configurations are possible after the pre-regulation of the air pressure. In the first configuration, the pre-regulated air goes to a back pressure regulator (Bellofram Type 10-B, 2-60 psi regulator) and to a cell pressure regulator (Bellofram Type 10, 2-120 psi regulator). Cell pressure and back pressure are independent in this configuration. To increase the cell pressure and the back pressure at the same rate requires the adjustment of two regulators simultaneously.

In the second configuration, the pre-regulated air goes to a back pressure regulator, that controls the back pressure, and to a slave regulator (Fairchild Model 15 Air Loaded Pressure Regulator) that controls the cell pressure relative to the back pressure. In this configuration, changes in the back pressure result in like changes to the cell pressure without the adjustment of any other regulator. The operation of the slave regulator generates a small pressure bias, such that the cell pressure will always be slightly larger than the back pressure. The magnitude of the pressure bias increases linearly as the back pressure increases. In this configuration, the cell pressure is increased by adjustment of the slave regulator. This configuration was used for all tests described in this report, and is illustrated in Figure 5.

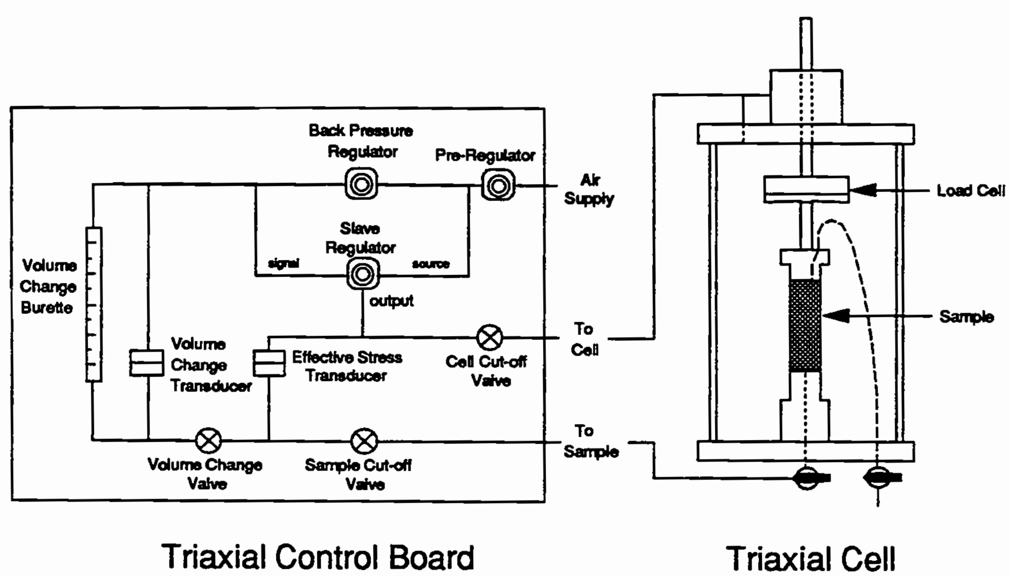

Figure 5. Test Configuration: Cell and Control Board.

3.2.1 Effective Stress Measurement

The effective stress in the sample is measured using a differential pore pressure transducer (Validyne model DP215 with a ±12.5 psi diaphragm). The high side of the effective stress transducer (EST) is open to the cell pressure line (measuring σ_3) and the low side of the EST is attached to the sample bottom (measuring u, when the sample cut-off valve is open). The EST reads the difference between these two pressures

$$\sigma'_3 = \sigma_3 - u$$

resulting in a direct measurement of the effective stress on the sample.

3.2.2 Measurement of Water Movement During Saturation

During saturation, the amount of water that moves into the sample is determined using a differential pore pressure transducer (Validyne model DP215 with a ±8.2″ H_2O diaphragm installed) and a water column of known diameter. The high side of the volume change transducer (VCT) is attached beneath a water column pressurized by the back pressure (or atmospheric pressure when back pressure is not being applied), thus, it reads the pressure applied to the water column plus the height of the water column. The low side of the VCT is attached to the top of the pressurized water column, thus it reads the pressure applied to the top of the column. The VCT reads the difference between these pressures, or the height of the water column. The volume of water that flows in or out of the sample is determined knowing the change in the height of the water column and its diameter.

3.2.3 Measurement of Volume Change During Consolidation

During consolidation, because the sample is saturated, the flow of water out of the sample must be accompanied by changes in the sample volume. Thus, the configuration described in Section 3.2.2 is also used to indirectly measure the volume change during the consolidation portion of these tests. The change in the volume of water in the volume change burette equals the change in the volume of the sample.

3.2.4 Measurement of Changes in Sample Length

Changes in sample length were measured using a linear variable distance transducer (LVDT) (Schevitz Engineering Type E1000, range ±1.000"). The LVDT was mounted outside the cell and measured sample length change indirectly based on the position of the bottom of the cell relative to the reaction bar of the load frame. This provides correct readings of the sample length change because, as the sample shortens, the position of the load piston is fixed on the reaction bar and the bottom of the sample (and the cell) moves upward.

3.3 Data Acquisition System

The data acquisition system consists of a hardware and a software component as described in the sections that follow.

3.3.1 Hardware

The data acquisition system consists of a Hewlett-Packard (HP) 9816 microcomputer, and an HP model 3421A data acquisition unit. Signal conditioning was performed by a Validyne model MC1 signal conditioner with the following boards: CD148 Carrier Demodulator (for LVDT output); CD18 Dual Output Carrier Demodulator (for EST output); CD19A Carrier Demodulator (for VCT output); and, SG71 Strain Gage Amplifier (for load cell output). This equipment was selected because of availability and the ease of programing data acquisition with this system, however, there are numerous other hardware configurations capable of doing data acquisition.

3.3.2 Software

The software developed as a part of this project was designed to facilitate the conduct of and data acquisition for multistaged, consolidated undrained triaxial tests. The program collects data, processes data and graphically monitors test progress (consolidation and shear) in real time. Calibration factors can be changed interactively, to allow for changes necessary after recalibration or due to hardware changes. The program does not control application of back pressures, consolidation pressures and sample loading, this must be done by the operator when directed to by the software.

3.3.2.1 Saturation Phase

The saturation phase is typically carried out in two stages: sample flooding and back pressure saturation. During sample flooding, water flows through the sample under a low head with the goal of replacing as much of the pore gas with water, as possible, prior to back pressure saturation, and to flood the test apparatus from the bottom cap to the top cap.

Flooding is continued at least until water flow in and out of the sample are approximately equal. Sample flooding is monitored manually and thus, is not a part of the triaxial software.

During back pressure saturation, pore pressure is increased, in small increments, to drive the pore gases into solution. Cell pressure and back pressure are increased simultaneously in order to maintain a nearly constant effective stress on the sample. All pressure changes are made manually by the operator, while the data acquisition system monitors the elapsed time and the change in water level in the volume change burette. Changes in the water level in the volume change burette indicate the movement of water into or out of the sample. The volume of water moving in and out of the sample is automatically calculated based on changes in the water level in the volume change burette. Typically, during back pressure saturation, the water level drops in the volume change burette, indicating that water is moving into the sample. Data are collected automatically every 3 minutes during back pressure saturation. The time between data points can be changed interactively at any time. Extra data points may be recorded at anytime, without changing the increment, by pressing a single key.

Back pressure is incremented when pore gases are no longer going into solution at the current pressure. The progress of saturation is checked by monitoring the changes in Skempton's B pore pressure parameter. Skempton's B is determined by measuring, under undrained conditions, the pore pressure response to a change in the cell pressure.

Skempton's B is determined using an operator selected subroutine. This routine instructs the operator when to make changes to system pressures and valve configurations, collects data, and calculates B.

3.3.2.2 Consolidation Phase

The consolidation phase monitors the level of water in the volume change burette at instants of time. Changes in water level are the result of movement of water into and out of the sample caused by an increase in the cell pressure. Because the sample is saturated, this movement of water represents a change in volume. The change in volume, as well as the elapsed time, are monitored by the data acquisition system. During the consolidation phase, the rate of data collection automatically varies. Initially, six data points are collected at approximately one second intervals, allowing the operator to start data collection for the consolidation phase, then open the sample to drainage. After these initial points, the time increment between data points doubles until a maximum increment of 30 minutes is reached. The time between data points can be changed interactively at any time. Extra data points may as be recorded as desired, using a single key. Volume change versus $\sqrt{time}$ is continuously plotted by the computer so that the progress of consolidation can be visually monitored.

3.3.2.3 Shear Phase

The shear phase monitors the vertical sample deformation, deviator load, minor principal effective stress, and elapsed time while the sample is subjected to strain controlled loading (or stress controlled loading, if desired). During the shear phase, data are not recorded until changes are measured in the load cell reading. This allows the operator to begin the data acquisition software, then begin strain controlled loading without collecting extraneous data points. Data points are recorded every 3 minutes or when specified changes occur in stress or strain, whichever occurs first. The default time interval can be changed, or extra data points can be recorded at any time.

During shear, it is assumed, that the sample volume is constant since drainage is not allowed and, that the sample deforms as a right circular cylinder. Raw data are recorded so that it is possible to recalculate results using different models for sample deformation. The stress-strain curve and the stress path are continuously plotted by the computer so that the progress of the shear phase can be monitored. Changes in the sample area and the artificial loads on the internal load cell resulting from changes in the cell pressure are considered in computations for the stress-strain and the stress path plots. The real-time plotting of the stress-strain curve and the stress path assists the operator in deciding when to stop shear. In multistage tests, these plots show previous stages as well as the current stage.

4 DEVELOPMENT OF TEST PROCEDURES

No standard procedures exist for low confining pressure, consolidated-undrained, multistage triaxial testing of compacted soils. The section that follows describes the development of procedures for this type of triaxial test. The procedures presented can be modified to accommodate other, similar, types of tests.

4.1 Sample Preparation

Selection of a sample preparation method was controlled to a large degree by the type of soil to be tested. In addition, it was desired to prepare a sample consistent with what might be expected to be found in the subgrade of a forest road. It was decided that this could most closely be represented by the AASHTO standard compaction test method (AASHTO T99). However, samples prepared by this method are too large, so that the sample must either be trimmed or molded to the desired size. Trimming the soils to be tested (MH) was not deemed possible, so it became necessary to mold the samples to the desired size without trimming.

Samples were compacted in a mold specially designed to produce samples 2.8" in diameter and 5.6" in height. To facilitate removal of the compacted sample, the mold was split and machined with a slight diameter taper over its length (0.005" in diameter over 5.60" in length). Removal of the sample from the mold without damage was an on-going problem. The sample tended to stick to the mold or to split near the mold seams when the mold halves were separated. It was not possible to extrude the sample from the mold without producing excessive shortening of the sample because of adhesion of the sample to the mold. Initially, the sample removal problem was reduced by lubricating the mold and by placing acetate strips over the mold seams during compaction. This reduced, but did not eliminate sample removal problems.

Construction of a new, Teflon coated, split mold virtually eliminated the sample damage problem. The Teflon coating allowed the sample to be partially extruded (just enough to cause movement of the sample in the mold, typically 110 to 150 pounds force required) before the mold was split open and the sample removed.

The compaction hammer used was a scaled down version of the AASHTO standard compaction hammer. The diameter of the hammer was selected to provide similar coverage to that accomplished by the AASHTO standard compaction hammer in the standard mold. The hammer weighs 1 lb, has a drop of 1 foot, and has Teflon shrink wrap applied to the outside sleeve to minimize damage to the Teflon coated mold during compaction. The compactive effort can be varied by changing the number of lifts and the number of blows per layer. During compaction a soil lip tends to form along the edge of the mold. It is recommended that this lip be trimmed and the surface of the lift be scarified, before placement of the next layer, to minimize the layer effect in the sample.

The soil was processed by forcing the soil through a No. 4 sieve to break up clay lumps and to facilitate removal of rocks and organic debris. After sieving, water was added to the soil to bring it to the desired moisture content. The prepared soil was then placed in a covered container in a humid environment and allowed to sit for at least three days prior to compaction to allow the added moisture time to equilibriate in the soil prior to sample compaction.

To reduce the required back pressure and the time required to reach saturated conditions, just prior to compaction, the soil storage container was filled with carbon dioxide and the soil was stirred in an attempt to replace as much pore gas as possible with carbon dioxide. Compaction was carried out with the mold placed in a container being continuously filled with carbon dioxide. During compaction, carbon dioxide was also was allowed to flow directly into the mold. Since carbon dioxide is heavier than air, it was assumed that the sealed container (bottom and sides) was filled with carbon dioxide, and that any losses caused by mixing with air at the top of the container were replaced by the carbon dioxide flowing into the container.

Since compaction was conducted in an atmosphere of carbon dioxide, it is reasonable to assume that a significant amount of the pore gases are carbon dioxide instead of air, and thus lower back pressures will be required to reach saturation. It is not essential that all of the pore gases be carbon dioxide for there to be a reduction in back pressures. Saturation typically could be accomplished with back pressures less than 30 psi.

After compaction, the top of the sample was trimmed and the sample was removed from the mold. To determine the "as-compacted" water content, samples were taken from the trimmings and from the remaining uncompacted soil. After measuring its dimensions, the sample was enclosed in an impermeable membrane and loaded into the triaxial cell.

4.2 Saturation

The percolation method of sample saturation, in which water is forced through the sample by applying a vacuum to the top of and pressure to the bottom of the sample, was deemed unacceptable for these tests. Not only does this method not achieve 100% saturation in fine grained soils, but it puts undesirable seepage forces on the sample that can cause consolidation of the sample (Lowe and Johnson, 1960). This may cause considerable error in low consolidation pressure testing.

The back pressure method achieves saturation by slowly building up pore pressures, while at the same time increasing the pressure on the sample by a like amount. The net effect of these pressure changes is no change in the effective stress felt by the sample. Saturation occurs in two ways, the increased pressures compress the pore gases according to Boyle's law (at constant temperature, the volume of a fixed mass of gas is inversely proportional to the pressure, Brescia et Al., 1974) and cause additional amounts of pore gases to go into solution in the pore water according to Henry's Law of Solubility.

To keep the soil skeleton from being compressed during the back pressure saturation process, pressures must be built up slowly and water allowed to flow into the sample to replace the volume formerly occupied by the gases that have compressed and/or gone into solution. Saturation was conducted in two stages: sample flooding followed by back pressure saturation.

4.2.1 Sample Flooding

After the cell was filled with water, the sample was flooded by allowing water to flow, under a small head, from the bottom to the top of the sample. The goal of flooding is to replace as much of the pore gases as possible with water before beginning back pressure saturation, and more importantly, to flood the system from the bottom cap to the top cap, including the space between the membrane and the sample. Flooding was allowed to proceed, at least, until the rate of flow of water into and out of the sample was approximately equal. Depending on the timing of tests, some samples were allowed to flood overnight.

The rate at which flooding occurred varied considerably from sample to sample. It was not always clear whether the flow of water was primarily through the sample or between the sample and the membrane. During some tests, the rapid progress of the wetting front indicated that, at least initially, water was moving between the sample and the membrane instead of just through the sample.

Small pore pressures develop during flooding. The magnitude of these pore pressures are important in low confining pressure tests and must be accounted for in setting the zero value for the effective stress transducer (EST) used to measure the minor principal effective stress on the sample. During initial testing, the effective stress reading was zeroed after flooding, thus the value considered to be zero effective stress was actually a small negative effective stress. The development of pore pressures in the sample as a result of flooding is illustrated by considering the effective stress at key times as shown in Figure 6.

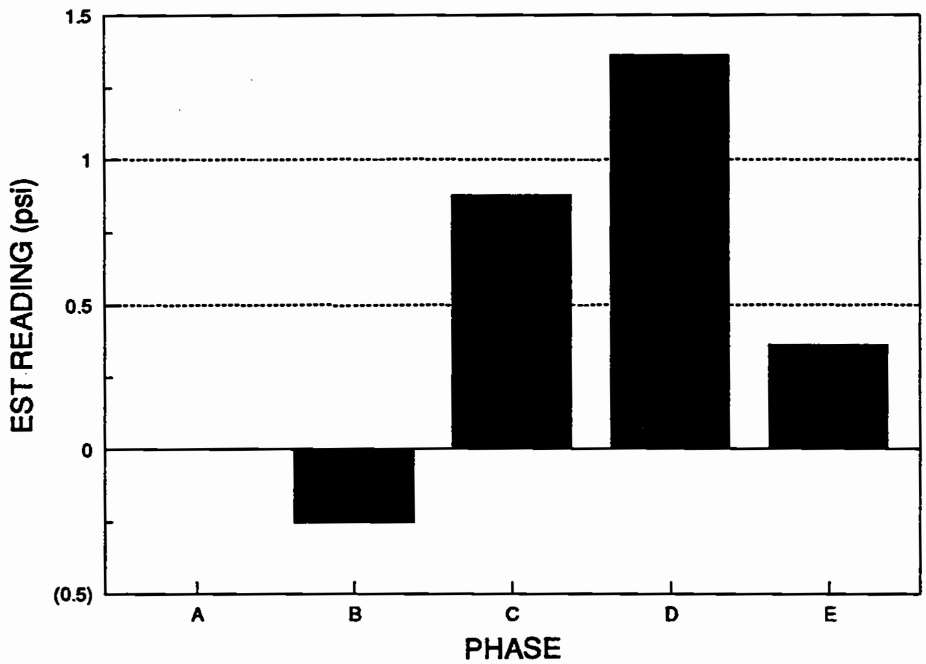

Figure 6. Effective Stress Conditions During Testing

Phase A shows the zero effective stress condition prior to the start of flooding. The effective stress reading was zeroed without a sample in the cell and with both sides of the EST open to the same pressure. Phase B shows the effective stress condition immediately after flooding. This small negative effective stress indicates the existence of positive pore pressures caused by the small head of water used to accomplish flooding. When flooding is stopped, the effective stress rises rapidly as illustrated in Figure 7. Phase C shows the effective stress condition after the sample was allowed to equilibriate with drainage valves closed for 22 hours after flooding. Figure 8 illustrates the effective stress change over time for the full 22 hours of this test (Figure 7 shows only the first 150 minutes of this test). This positive effective stress is a result of the development of negative pore pressures (capillary tension) as the sample tries to reach equilibrium.

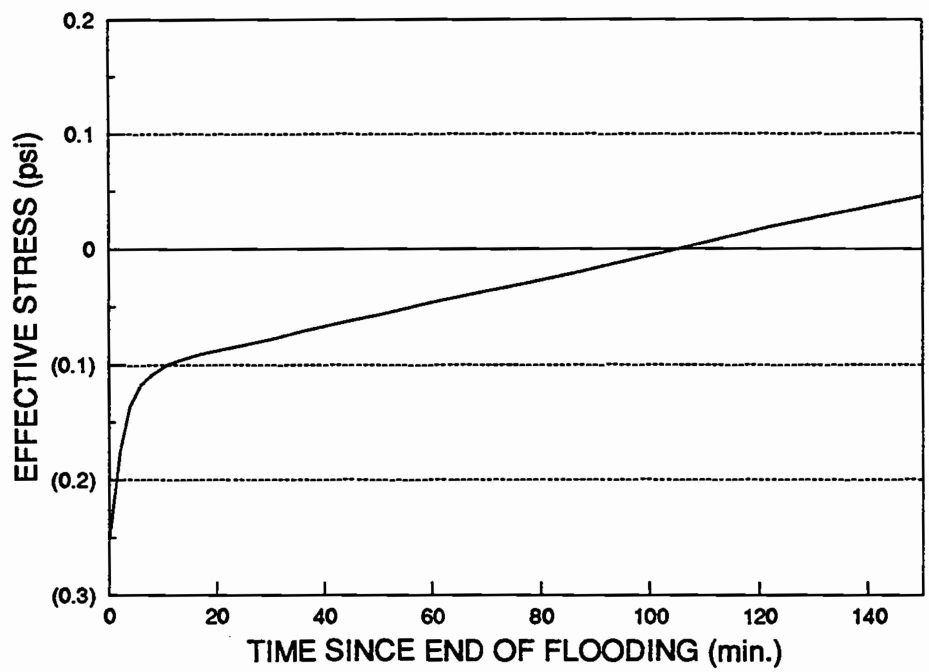

Figure 7. Effective Stress Change After Flooding - First 150 Minutes

The pore pressures developed during flooding are small, but they cannot be ignored during low-confining pressure test. The original test procedure zeroed the effective stress reading after flooding, just prior to the start of saturation, assuming that the effective stress was zero. If the sample was not being tested at low confining pressures this assumption might not be a problem, however for low confining pressure tests this assumption has a significant impact on test results.

Consider again Figure 6. If effective stress is zeroed after the completion of flooding, "zero" would fall somewhere between the end of flooding (phase B) and equilibrium (phase C) values. Because, as Figure

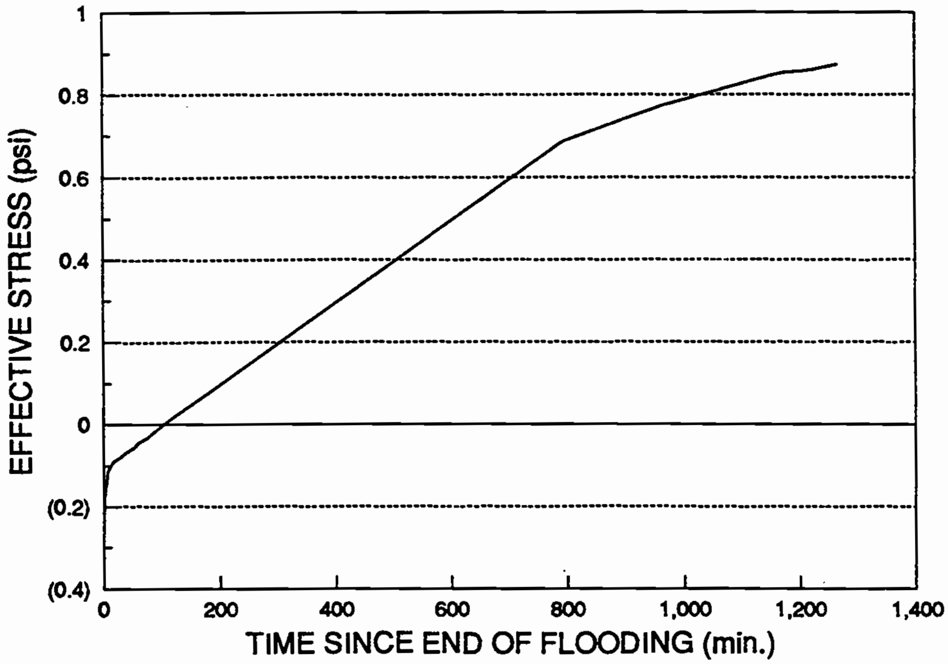

Figure 8. Effective Stress Change After Flooding - Full 22 Hours

7 indicates, effective stress increases rapidly after flooding, "zero" is likely to be somewhere near the equilibrium. The first consolidation phase will cause the effective stress to rise, and then the shear phase will cause it to drop again, as seen in Figure 6, phase D and E. The result is that the sample may appear to develop negative effective stresses during shear at the lowest confining pressure. Although the error caused by zeroing effective stress after flooding is systematic, it is not possible to determine its magnitude. Thus, tests in which the effective stress was zeroed after flooding must be considered suspect.

The zeroing problem described above became apparent in two ways. First, it was always difficult to keep effective stress reading at zero between flooding and the beginning of back pressure saturation. This is apparently the result of the flooded sample trying to come into equilibrium resulting in changes to the effective stress over time (Figure 7).

Secondly, tests in which the first stage was consolidated at less than about 1 psi, were apparently developing negative effective stresses during shear. Figure 9 illustrates the stress path for a test in which effective stress was zeroed after flooding. The development of apparently negative effective stress is easily seen in the first stage because the stress path crosses above the line where $q = p'$.

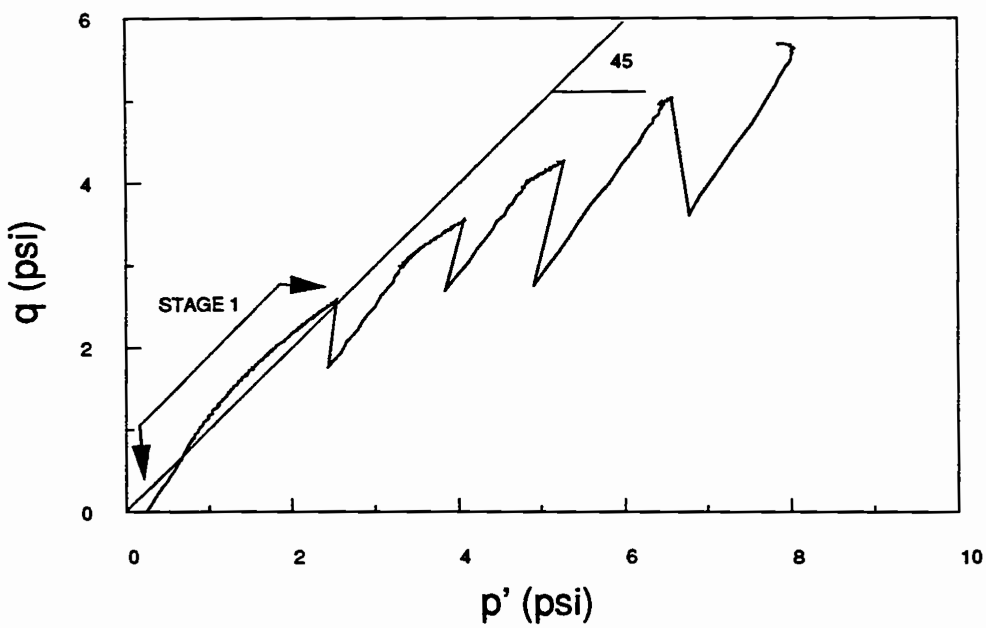

Figure 9. Results Showing Apparent Negative Effective Stresses

Initially, it was unclear whether this was a real phenomenon or a compliance error. After further investigation, it was determined that this was not real, but instead a function of when effective stress was zeroed. The resulting error is systematic, with an unknown magnitude, as previously discussed. The apparent impact of zeroing effective stress at the wrong time is to cause a shift of the stress path along the p′ axis. The magnitude of the shift cannot be determined, but, as a minimum, it must be sufficient to move all of the stress path below the $q = p'$ line.

Figure 10 illustrates the minimum stress path shift that would be require to adjust these data. In Figure 11, the K_f line is added for the unadjusted and the adjusted stress paths. The slope of the K_f line remains constant, but the q-axis intercept drops for the shifted stress path. Because of the relationship between the K_f line and the Mohr-Coulomb failure envelope (discussed in section 4.4.3), the friction angle, ϕ', remains constant and the cohesion, c', drops for the shifted line. Thus, it is expected that improperly zeroed tests will overstate the magnitude of c'.

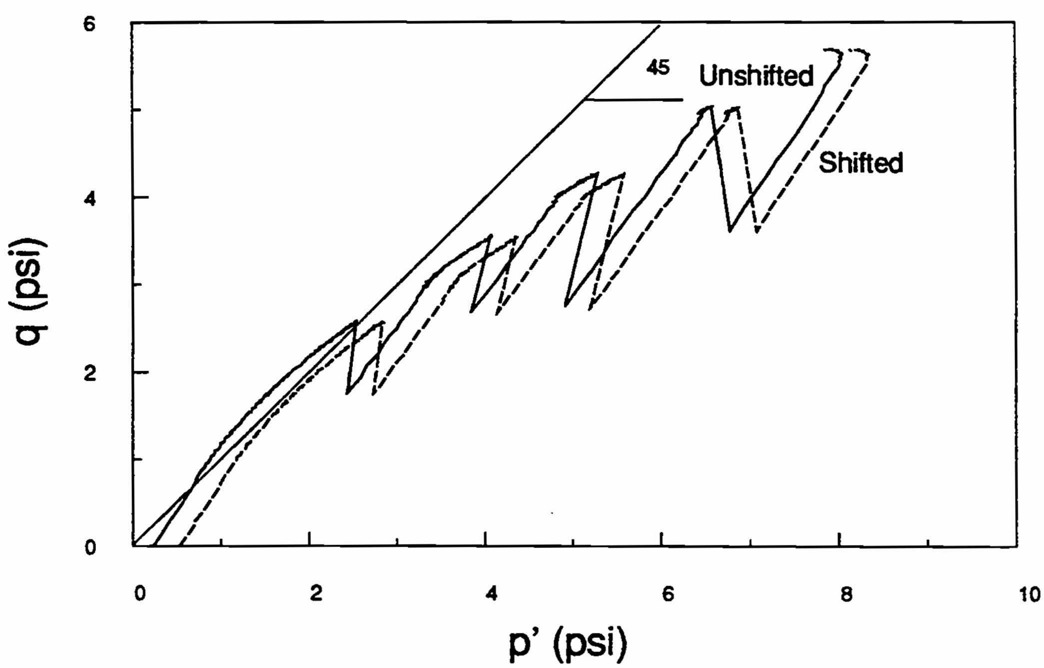

Figure 10. Shifted Stress Path

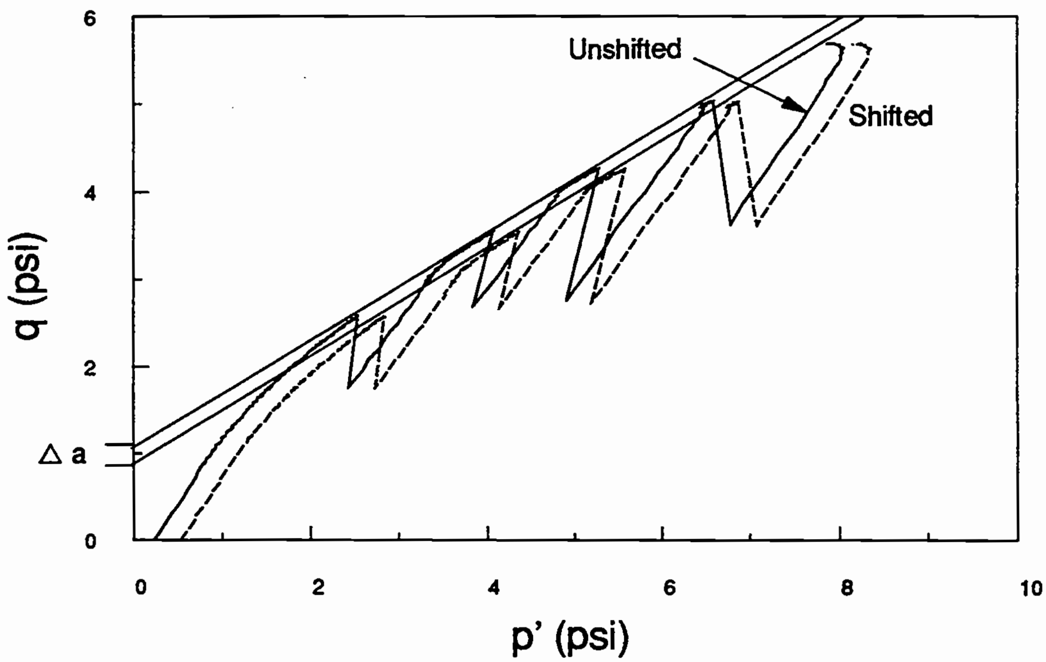

Figure 11. K_f Lines for Original and Shifted Stress Paths

This expectation is supported by tests conducted during the development of this test procedure. The average strength parameters of two properly zeroed tests were compared to the average strength parameters of five improperly zeroed tests with approximately 5% lower dry density. Since the samples were prepared from the same soil in the same area of the compaction curve (above optimum moisture content) the friction angles should be approximately equal and the denser samples would be expect to have a higher cohesion. The average strength parameters of the properly and the improperly zeroed tests were found to be almost identical, thus there is an indication of an overstated value for the cohesion as obtained by the improperly zeroed test.

This problem was eliminated by a change in procedures, that shifted zeroing of the effective stress reading from after flooding, to before the sample was placed in the cell. The recommended procedure for zeroing effective stress is to fill the cell with water and place it in its testing position. Open the appropriate valves so that the EST experiences the same pressure on both sides (no added pressure to system), then zero the effective stress reading. The height of the cell relative to the EST must remain constant throughout the test.

4.2.2 Back pressure saturation

The samples were saturated by the application of a back pressure. This was accomplished by the simultaneous increase of the cell pressure and the back pressure (pore pressure). This simultaneous increase was facilitated by the use of a slave regulator that allowed the cell pressure to increase with increases in the back pressure by adjusting only the back pressure regulator. The use of the slave regulator results in a small pressure bias, such that the cell pressure is always slightly higher than the back pressure. The magnitude of the bias increases as the back pressure increases, however the bias never exceeded 0.75 psi. In the range of back pressures utilized, the bias is linearly related to the back pressure magnitude, as shown in Figure 12.

The existence of the bias limits the minimum confining pressure that can be utilized. The magnitude of the bias is a function of the back pressure, but its effect can be reduced by changing the position of the triaxial cell relative to the triaxial control board. If the cell is lowered relative to the control board, the pressure head from the fluid in the volume change

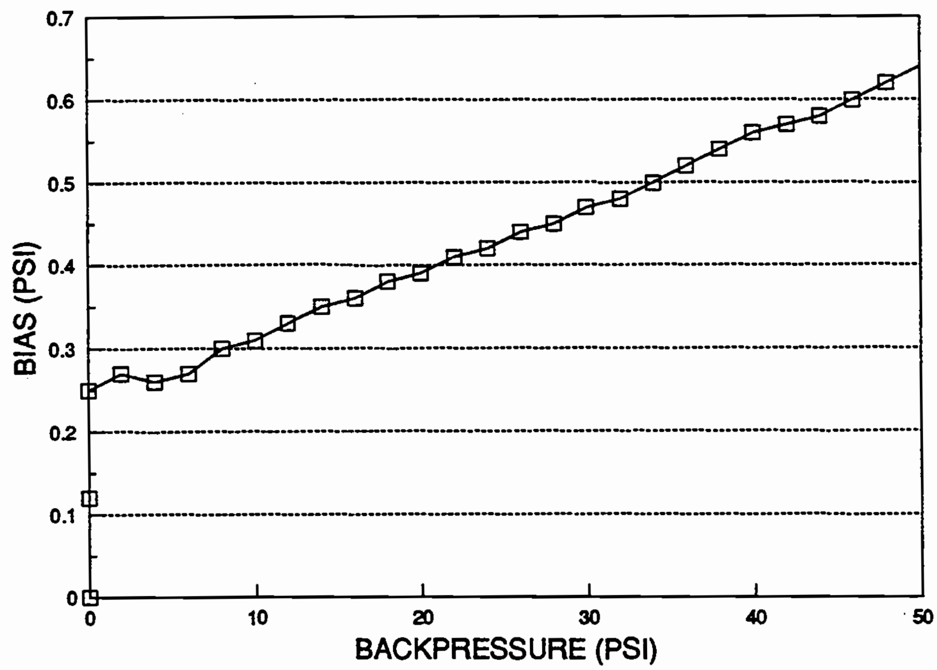

Figure 12. Bias Versus Back Pressure.

burette is increased and, therefore, the pore pressure is increased (and effective stress is decreased). The increased pore pressure allows the slave regulator to have some pressure on it to maintain zero effective stress.

The pressure bias from the slave is positive in the cell, so that as back pressure increases it also increases. Since the slave regulator now has pressure on it, it is possible to back off the slave to maintain the desired zero effective stress. In theory, it is possible, by this method, to configure the system so that the pressure bias can be eliminated. Note in Figure 12, that the vertical position of the pressure bias curve is dependent on the position of the triaxial cell relative to the triaxial board as described above. If the cell is lowered enough, the bias will appear to be negative over a range of back pressures. It is important, if this method is used to reduce the pressure bias magnitude, that the position of the triaxial cell not be lowered relative to the triaxial control board after the effective stress reading is zeroed.

When the back pressure is incremented, the sample immediately feels the effects of the increase in cell pressure, however the pore pressure increase is not felt immediately throughout the sample. This delay results because the transmission of the pore pressure increase requires the movement of water into the sample to replace the pore gases that go into solution. The rate at which the pore pressure increase is transmitted is dependent on the permeability of the soil and the amount and type of gases present in the pores. Because of this delayed increase in pore pressures throughout the sample, the rate of back pressure application was carefully controlled to minimize the increase of effective stress in the sample.

Factors that can play a roll in the selection of the back pressure increment include:

1. The rate at which the pore gases go into solution in response to increases in back pressure.
2. Rate at which water can flow into the sample to replace volume which was previously occupied by pore gases that have compressed or gone into solution.
3. Desire to minimize volume changes that might be occurring as a result of small differences between the pore pressure and the cell pressure because we don't have the ability to easily measure these volume changes.

4. The desire to achieve saturation in a reasonable period of time.

Bishop and Henkel (1962) suggest that standard practice is to increase back pressure in 5 to 10 psi increments, however, this recommendation is made assuming tests are to be conducted with confining pressures in the typical civil engineering range of 5 to 30 psi. The literature gives no guidance in selecting the increment for tests that will be conducted outside this range. It was decided that a smaller increment is desirable for low-confining pressure tests, but that the increment must be sufficiently large so that saturation can be achieved in a reasonable period of time.

In practice, the back pressure increment was limited to 3 psi, and the next increment was not applied until water flow into the sample ceased (evidenced by constant or nearly constant volume change transducer readings over a period of time) and pore gas was no longer going into solution. The determination whether pore gas is still going into solution is made by closing the sample to drainage and watching the effective stress transducer reading. If pore gases are still going into solution, the pore pressure will drop because no water can move into the sample and thus, the effective stress will rise.

It is not possible during saturation to compute sample volume changes. The saturation process must be conducted at a constant effective stress to minimize consolidation effects, and the constant effective stress must be given an opportunity to equilibriate prior to incrementing pore pressure. If this is done, the measured water inflow is primarily due to increasing saturation, rather than to changes in sample volume.

4.2.3 Verification of Saturation

Saturation of the sample was verified by checking Skempton's B-pore pressure parameter. This is accomplished by preventing sample drainage and applying an additional cell pressure to the sample. The pore pressures before and after the increased cell pressure are measured to determine the change in pore pressure (Δu) for the change in cell pressure ($\Delta\sigma_3$). The pore pressure parameter B is computed using:

$$B = \frac{\Delta u}{\Delta \sigma_3}$$

In a fully saturated soil, the change in pore pressure (Δu) should equal the change in cell pressure ($\Delta\sigma_c = \Delta\sigma_3$). Theoretically, this means, in a saturated soil, that $B = 1$. However, in relatively stiff soils, it is possible to have a fully saturated soil when B is less than 1 (Holtz and Kovacs, 1981; Black and Lee, 1973). This can be explained by considering the relationship between B and the compressibility of the soil structure and the voids (Skempton, 1954).

$$B = \frac{\Delta u}{\Delta \sigma_3} = \frac{1}{1 + \frac{n C_v}{C_c}}$$

where:

C_v = compressibility of the voids

C_c = compressibility of the soil structure

n = porosity

In a saturated soil, C_v equals the compressibility of water. As a soil becomes more rigid, C_c becomes smaller, thus the ratio $\frac{C_v}{C_c}$ becomes larger, resulting in a decrease in B. For compacted silts and clays, the theoretical B-value is 0.9988 for 100 percent saturation, and 0.930 for 99 percent saturation (Holtz and Kovacs, 1981). For the soil investigated as part of this project, the sample was considered fully saturated when B was 0.97 or greater.

4.3 Consolidation

Consolidation of the sample was accomplished by increasing the cell pressure while holding the back pressure constant and allowing sample drainage. The cell pressure was increased relative to the back pressure by use of the slave regulator. During the first consolidation stage, consolidation is isotrophic because the load piston is not applying any load to the top cap and thus the sample feels an all around consolidation pressure equal to the difference between the cell pressure and the back pressure. During subsequent consolidation phases, at least two options exist for the sequencing of the change from the axial load application of the shear phase to the confining stress change of the consolidation phase. The first option is to remove the deviator stress at the end of the shear phase before beginning the next consolidation phase. The second option is to maintain the deviator stress at the end of each shearing phase through the consolidation phase that follows.

A typical procedure for the first option is to, after the end of the shear phase, with drainage closed, remove the deviator stress, while maintaining a constant confining pressure. Then allow time for the sample pore pressures to equalize before increasing the confining pressure and opening drainage. Schoenemann and Pyles (1988) reported that removal of the deviator stress results in the development of substantial positive excess pore pressures that drive the effective confining stress well below values experienced by the sample during the previous consolidation and shear phases. Considerable axial rebound is also experienced upon removal of the deviator stress. This axial rebound occurs while the drainage lines are closed (constant volume), so that radial compression must also be occurring.

A typical procedure for the second option is to, after the end of the shear phase, close drainage and lock the load piston into place. The load piston is kept locked in place until consolidation begins. Stress relaxation will occur during the time that the load piston is locked. Consolidation may be anisotrophic by applying an axial load as well as increasing the cell pressure, then unlocking the load piston and allowing drainage simultaneously. Isotropic consolidation without elongation, that could be damaging to the sample, would require full relaxation of the deviator stress from the previous stage. It is doubtful that full stress relaxation can be achieved.

Schoenemann and Pyles (1988) report that there are significant differences between the stress paths for isotropically consolidated undrained (ICU) tests (deviator stress removed) and anisotropically consolidated undrained (ACU) tests (deviator stress maintained). The ICU tests experience large loops in their stress paths, while the ACU tests do not. Even though the stress paths are different, the parameters obtained from the K_f lines were not statistically different. They concluded, at least for the soils tested, that the ICU and the ACU procedures are equivalent. Since the soils being tested as a part of this project are the same as those tested by Schoenemann and Pyles, their results maybe applied in the selection of consolidation procedures.

The consolidation procedure that was adopted is a modified version of the ACU test described by Schoenemann and Pyles (1988). During the first consolidation phase, the sample is consolidated isotropically by being subjected to an increased cell pressure with the drainage lines open. During subsequent consolidation phases, the sample length is maintained after the completion of the shear phase and consolidation is accomplished by an increase in the cell pressure. No additional axial load was applied. As a result, the sample experienced uncontrolled anisotrophic consolidation, with the anisotrophy being an artifact of the previous deviator stress and stress relaxation experienced in the sample.

The volume change during consolidation was monitored by measuring the flow of water from the sample with the aid of a pore pressure transducer, as previously described. A plot of volume change versus $\sqrt{time}$ was continuously plotted by the computer, so that the progress of consolidation could be visually monitored. In addition, this information was used to evaluate the coefficient of consolidation, C_v, by Taylor's square root of time fitting method for the purpose of selecting the shear rate to be used. Taylor's method was selected over Casagrande's logarithm of time fitting method for several reasons, including: low consolidation pressures are being used, thus small system fluctuations can have major impact on the value evaluated for C_v; because of the way the system must be operated and because of potentially rapid consolidation it might not be possible to collect all of the necessary data at the beginning of the test; and it was desired to standardize on one method that is appropriate for all test conditions.

The use of low consolidation pressures makes it difficult to determine whether the volume changes measured are the result of consolidation or of system fluctuations. It was not uncommon, during the first consolidation phase to see volume changes less than 0.1 in^3. Figure 13 illustrates the consolidation curve for the first stage of a test, in which the confining stress was 1.18 psi. The normal procedure for the first consolidation phase is to apply the desired consolidation pressure with drainage closed, begin consolidation data collection, then open the drainage so that consolidation can begin. Initially, volume change readings are read rapidly, and the first several data points are recorded before or during the opening of drainage, thus the large early volume change fluctuations, seen in Figure 13 and Figure 14, are an artifact of the test procedure. Because of the low consolidation pressures, it is not clear whether the subsequent, smaller, fluctuations are the result of real changes to the sample, or are the result of undefined system fluctuations.

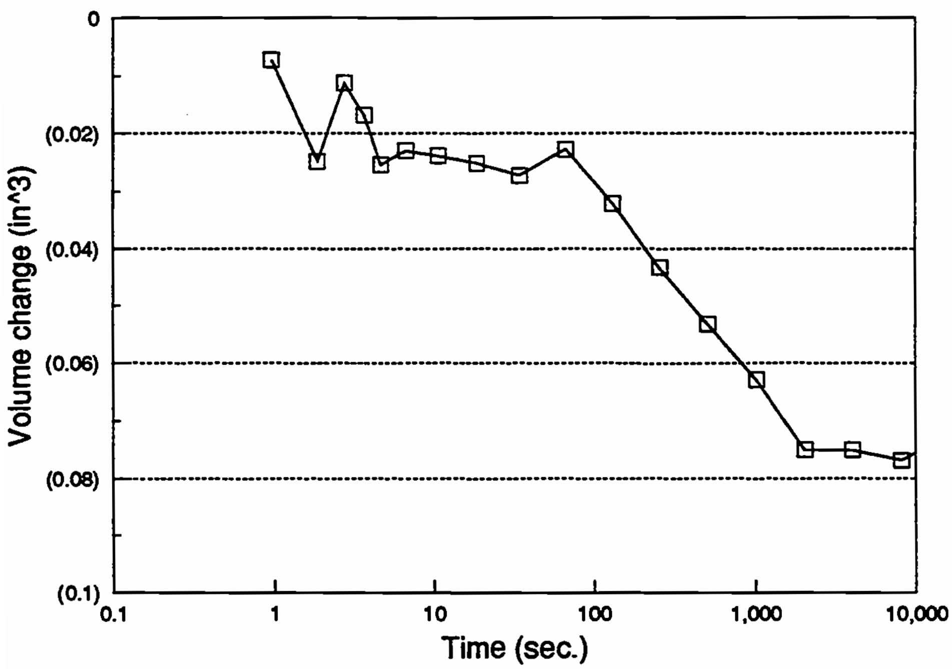

Figure 13. Consolidation Phase - Logarithm of Time

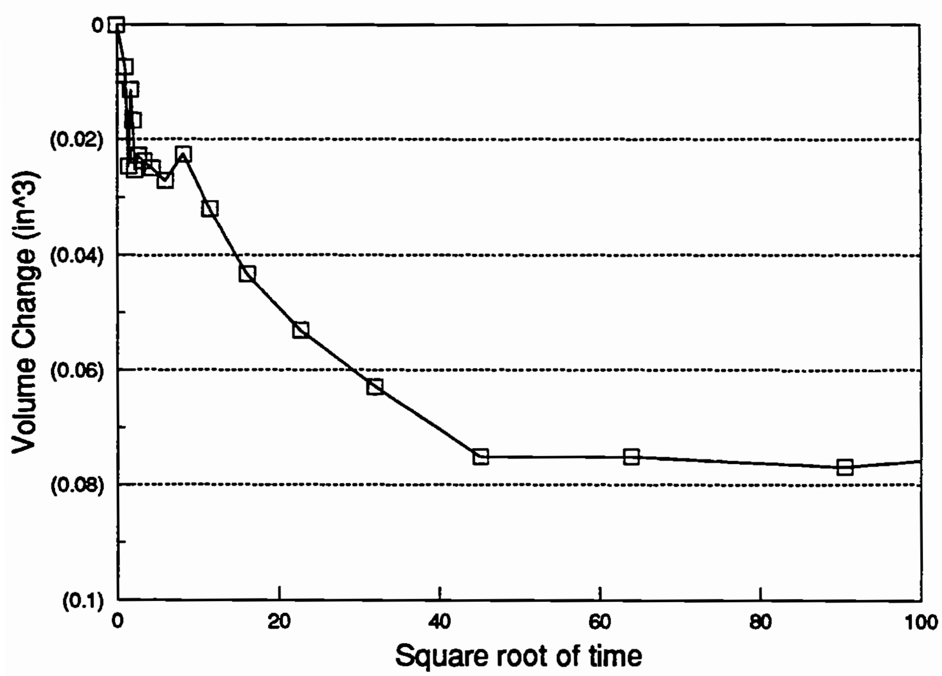

Figure 14. Consolidation Phase - Square Root of Time

Casagrande's method relies heavily on the information in the early portion of the log time-deformation curve for the determination of C_v. It is apparent, from Figure 13, that the system fluctuations occurring in this part of the test make Casagrande's method difficult to use. These same fluctuations are apparent in the early portion of the square root of time-deformation curve used by Taylor's method (Figure 14), but have less impact on the determination of C_v.

The samples were typically consolidated at confining pressures between 0.5 psi and 6.0 psi. These values were picked so that the resulting strength envelope is representative of the conditions experienced in the upper levels of forest road subgrades.

4.4 Shear

After each consolidation phase, the drainage line was closed to drainage and the sample was subjected to axial compression at a constant rate of strain. Stress controlled loading could also be utilized, but a different loading system would be necessary. The rate of loading may affect the shear strength magnitude, increased loading rates have been reported to cause increased strength and extremely slow loading rates may experience creep movements that result in lower measured strengths (Saada and Townsend, 1981). In saturated, undrained tests, loading rates must be sufficiently slow to allow pore pressures to equalize throughout the sample. The rate of strain was selected, with consideration of the coefficient of consolidation obtained from the first shear phase, to insure that pore pressure equalization in the sample could be realized. The same strain rate was used for each shear phase of the multistaged test because it was found that the required rate did not change significantly with subsequent stages. In addition, strain rate changes on the load frame were made by changing the gear configuration so that small rate changes were not possible.

During shear the following measurements were made:

1. Effective minor principal stress (σ'_3). This was directly measured using a differential pore pressure transducer as was previously discussed.
2. Axial load. The deviator load was measure using an internal load cell located in the triaxial cell.
3. Sample length change. Changes in sample length were measured using a LVDT mounted outside the triaxial cell. The LVDT actually measured the movement of the bottom of the triaxial cell relative to the reaction beam of the load frame, however, this is equivalent to the change in the sample length because strain loading in the load frame occurs by movement of the triaxial cell upwards towards the reaction beam.

4.4.1 Area correction

The deviator stress on a sample during undrained shear is computed by dividing the applied load, as measured by the load cell, by the area of the sample. Since shear is undrained, the volume of the sample remains constant, while its length decreases and its cross-sectional area changes. To reflect this, it is necessary to compute an adjusted area of the sample before computing the shear stress. The changes in sample cross-sectional area do not occur uniformly throughout the sample, thus, it is necessary to compute a corrected area based on the initial area, the measured axial deformation, and an assumed deformation pattern. Germaine and Ladd (1988) suggest three deformation patterns that can be used to calculate the adjusted area, they are: the right cylinder correction, the parabolic correction, and the bulging correction.

The right cylinder correction assumes that the sample deforms as a right cylinder and can be expressed as:

$$A_c = A_o\left(\frac{1-\varepsilon_v}{1-\varepsilon_a}\right)$$

where

A_c = corrected area
A_o = area at zero shear
ε_v = volumetric strain (=0, for undrained test)
ε_a = axial strain

The right cylinder correction is the most commonly used assumption and it is representative of the ideal case for a sample with frictionless ends. La Rochelle et al. (1988) recommend the use of this correction prior to the development of a shear plane in the sample.

The parabolic correction was developed specifically for use in undrained conditions. This correction assumes that the sample deforms in a barrel like manner, with the area being computed at the largest mid-plane section. The parabolic correction can be expressed as:

$$A_c = A_o\left(-\frac{1}{4}+\frac{\sqrt{25-20\varepsilon_a-5\varepsilon_a^2}}{4(1-\varepsilon_a)}\right)^2$$

The bulging correction assumes that the strains are concentrated at the central portion of the sample, and can be expressed as:

$$A_c = A_o\left(\frac{1-\varepsilon_v}{1-a\varepsilon_a}\right)$$

where

a =experimental constant, normally 1-2

The samples being tested typically failed under small strains, with minimal bulging, and without the development of a visible shear plane, so the right cylinder correction was applied to determine the corrected area.

4.4.2 Failure Criteria

Seed et Al. (1960) described three possible failure criteria that may be used to specify the strength of a soil.

a) The maximum deviator stress that the sample can sustain.

$$(\sigma_1 - \sigma_3)_{max}$$

b) The maximum effective principal stress ratio.

$$\left(\frac{\sigma'_1}{\sigma'_3}\right)_{max}$$

c) The deviator stress at some limiting value of axial strain.

$$(\sigma_1 - \sigma_3)\ \textit{at some strain}$$

The maximum deviator stress failure criterion was considered unsuitable for use because it is difficult to determine when the maximum is reached since the deviator stress versus strain curve tends to rise then flatten out without reaching a definite peak. This lack of a definite peak is problematic when multistage testing is used, because it is important to carry each stage far enough to ensure that failure has occurred, but not to carry it so far that there is excessive damage to the sample. Figures 15, 16, and 17 illustrate data from tests on a highly plastic silt that was compacted below, near, and above optimum moisture content (standard AASHTO), respectively. These figures clearly illustrate the difficulty in determining where the maximum deviator stress occurs in each stage, and thus when failure has occurred by this criteria. Note that the sudden drops in deviator stress present in the first stage of the below optimum test are the result of the load piston slipping and adjusting itself on the load bolt.

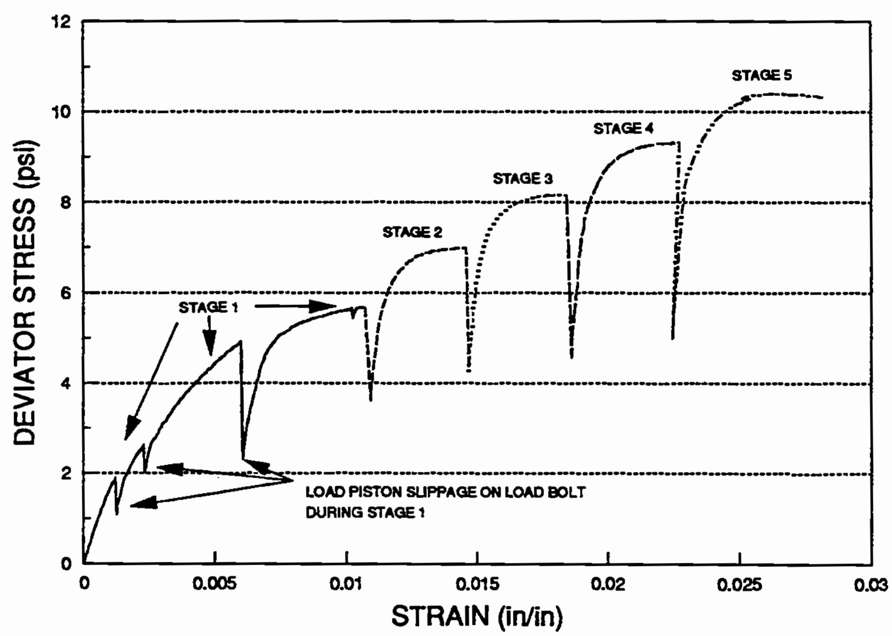

Figure 15. Deviator Stress versus Strain - Sample Compacted Below Optimum Moisture Content.

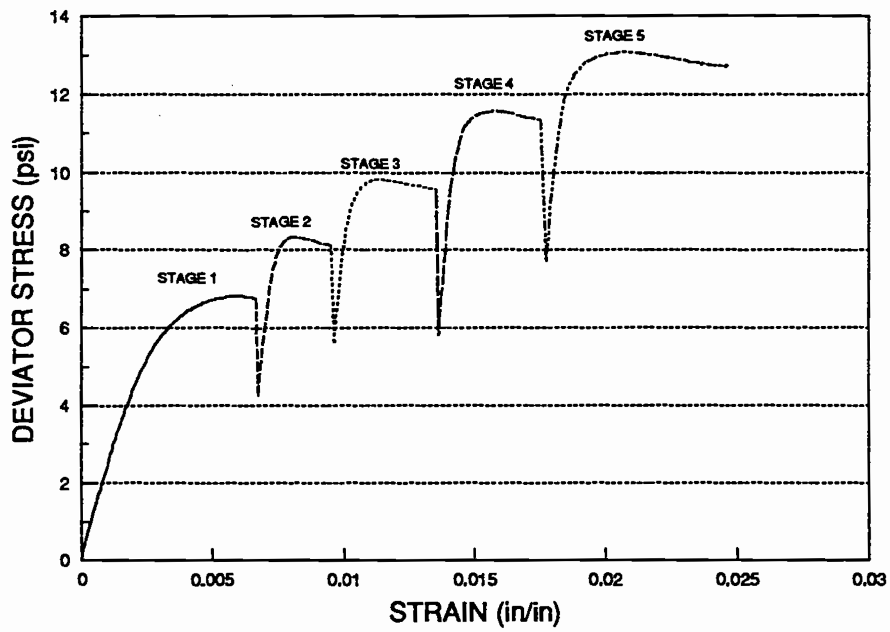

Figure 16. Deviator Stress versus Strain - Sample Compacted Near Optimum Moisture Content.

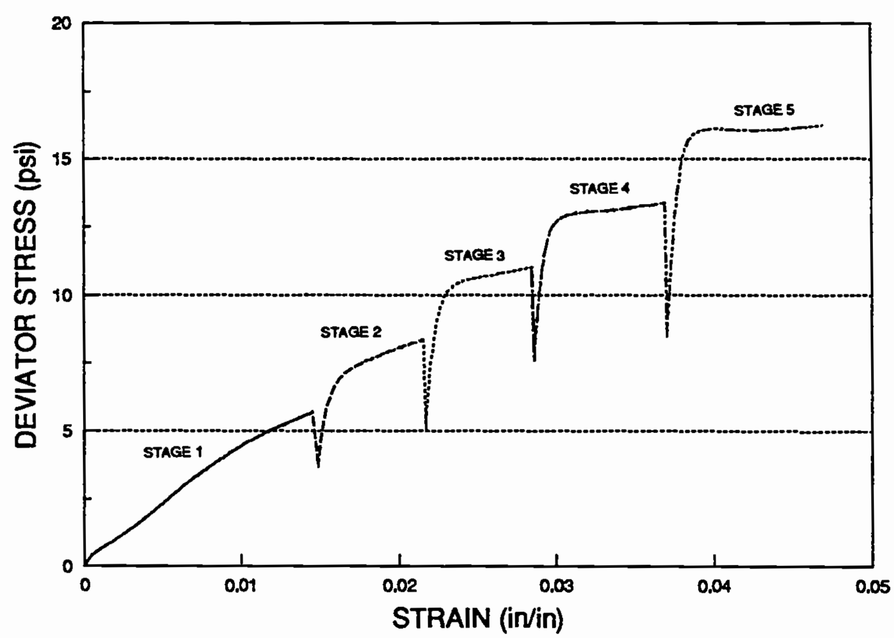

Figure 17. Deviator Stress versus Strain - Sample Compacted Above Optimum Moisture Content.

Seed et al. also indicated that, since effective stresses are so important to the strength of a soil, the maximum effective principal stress ratio (EPSR) is, from a fundamental standpoint, possibly the most appropriate definition of strength. However, it proves to be an unsuitable criterion for failure for multistage tests. During the first stage of these tests a maximum effective principal stress ratio is clearly evident, because the ratio rises sharply to some point then drops off sharply. In subsequent stages, the EPSR does not rise as sharply, and then instead of dropping off, it tends to level off making it difficult to identify failure by this criterion. Figures 18, 19, and 20 illustrate EPSR versus strain for the same tests previously illustrated. From these figures, it can be seen that the EPSR reaches a definite peak in the first stage, but not in subsequent stages.

Also evident during the first stage is a considerable amount of noise in the EPSR. Because of the small values of effective stress, the normal measurement noise is magnified by the calculation of EPSR in the first stage. Very small fluctuations in effective stress result in relatively large fluctuations in EPSR.

The small values of effective stress also result in relatively large values for EPSR during the first stage. It is expected that an improved matching of the range of the EST and the load cell would work to reduce this noise. However, because of the range of stresses encountered in multistage tests, noise in the first stage will continue to be a problem. It is clear that the maximum effective principle stress ratio is not a good method for defining failure in multistage tests.

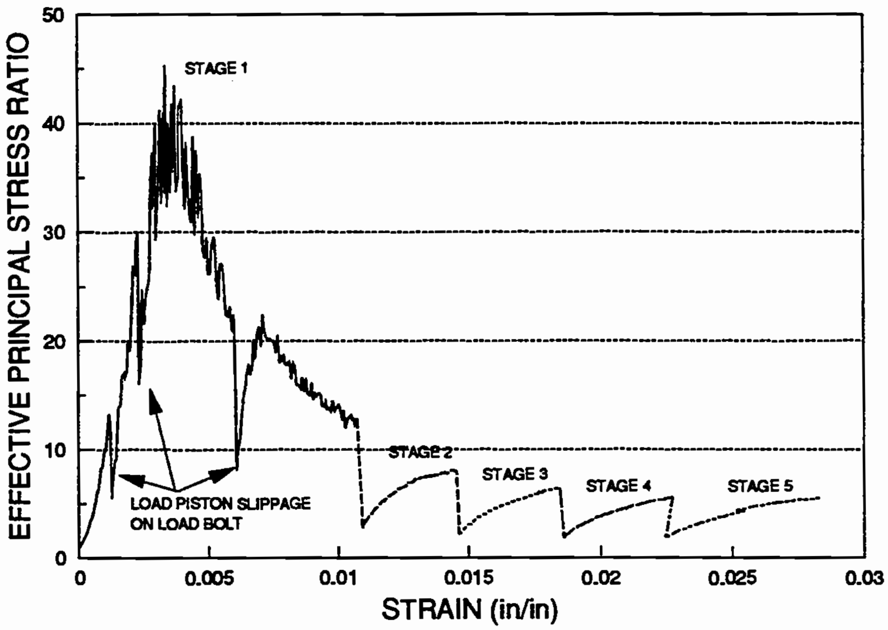

Figure 18. Effective Principal Stress Ratio versus Strain Sample Compacted Below Optimum Moisture Content.

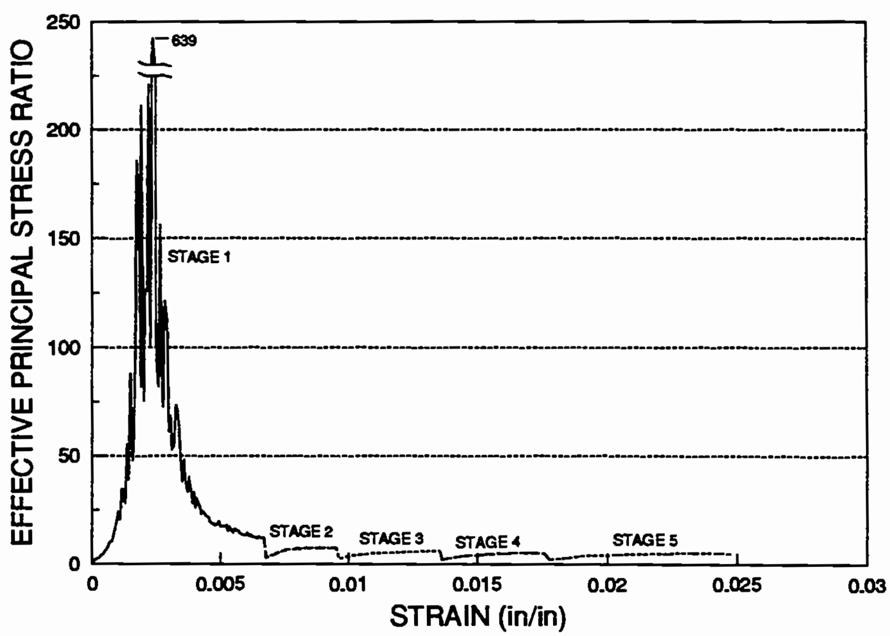

Figure 19. Effective Principal Stress Ratio versus Strain Sample Compacted Near Optimum Moisture Content.

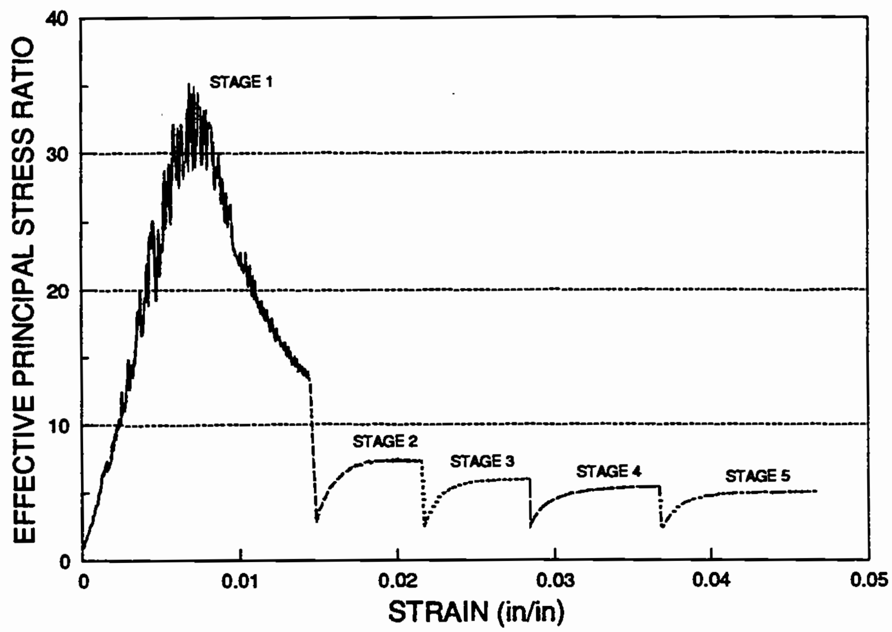

Figure 20. Effective Principal Stress Ratio versus Strain Sample Compacted Above Optimum Moisture Content.

The limiting strain failure criterion was also considered unacceptable for this type of testing. Typical limiting strain criteria are significantly higher than the strains that were required to produce failure by the maximum effective principal stress ratio criterion or the maximum deviator stress criterion (Figure 15-20). Typically, this failure criterion is used when no clear evidence of failure can be seen in the test results, or when the use of the soil must be limited to low strains. Additionally, since multistage testing is being used, it is also desired to keep strain to a minimum for each stage so that sample damage is minimized.

Kenny and Watson (1961) presented an alternative failure criterion that is more suitable for use in multistage tests. They defined failure as occurring when the stress path is tangent to the maximum effective shear strength envelope. For a multistage test, they indicated that it is not critical to identify the exact point where this occurs, only to insure that this point was passed before continuing on to the next stage.

Figures 21, 22, and 23 illustrate the stress paths for the same three tests previously illustrated. The stress paths clearly move toward a line (K_f line), and continued straining of the sample results in the stress path either moving along this line or remaining essentially stationary. The early stages typically have a stress path the moves upward and to the right along an essentially straight line, indicating dilative behavior. Later stages typically have stress paths that, after a period of moving upward and to the right will turn back and travel down the K_f line, indicating compressive behavior. The middle stages are typically transitional between the stress path travelling up or down the K_f line, indicating a shift from dilative to compressive behavior.

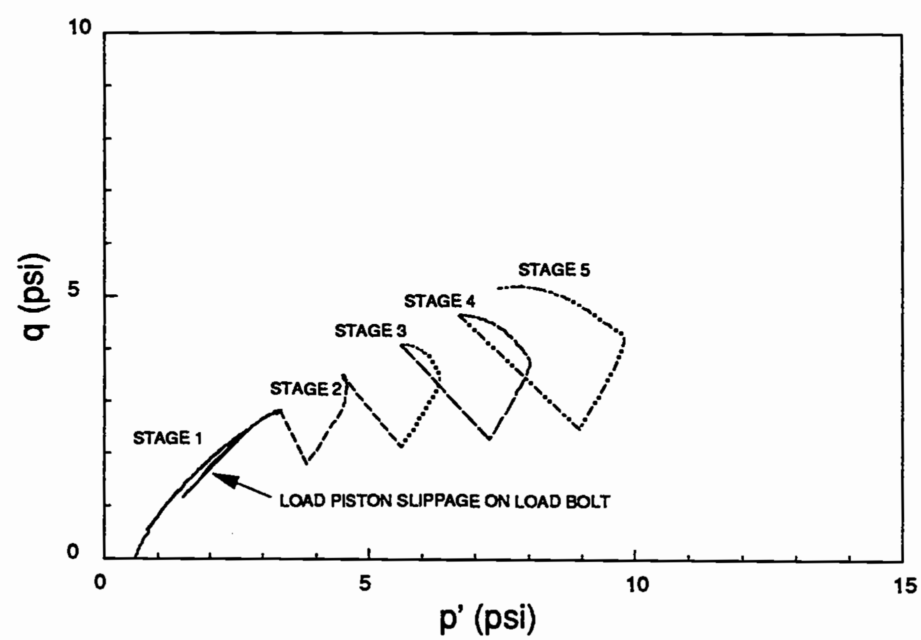

Figure 21. Stress Path - Sample Compacted Below Optimum Moisture Content.

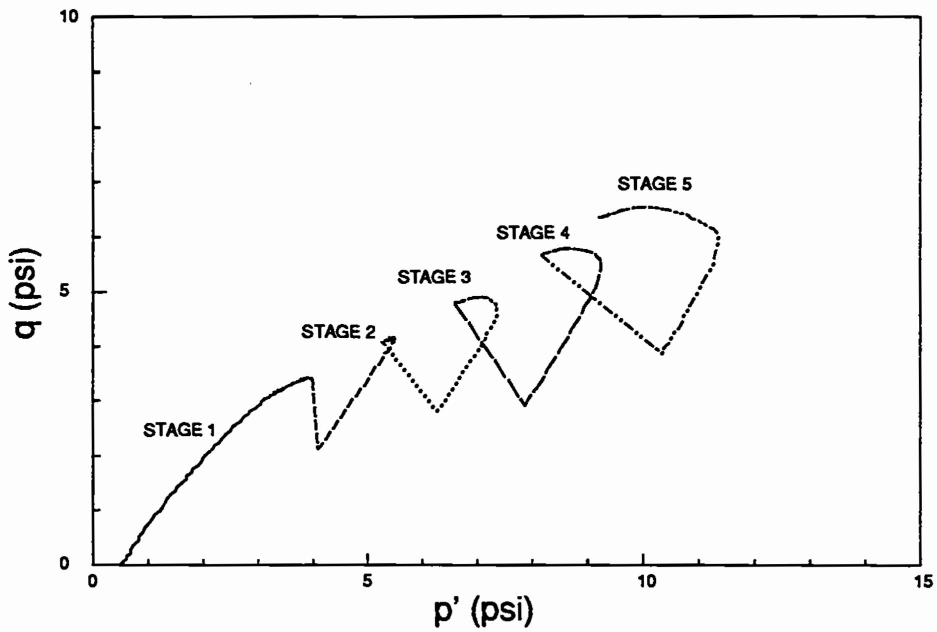

Figure 22. Stress Path - Sample Compacted Near Optimum Moisture Content.

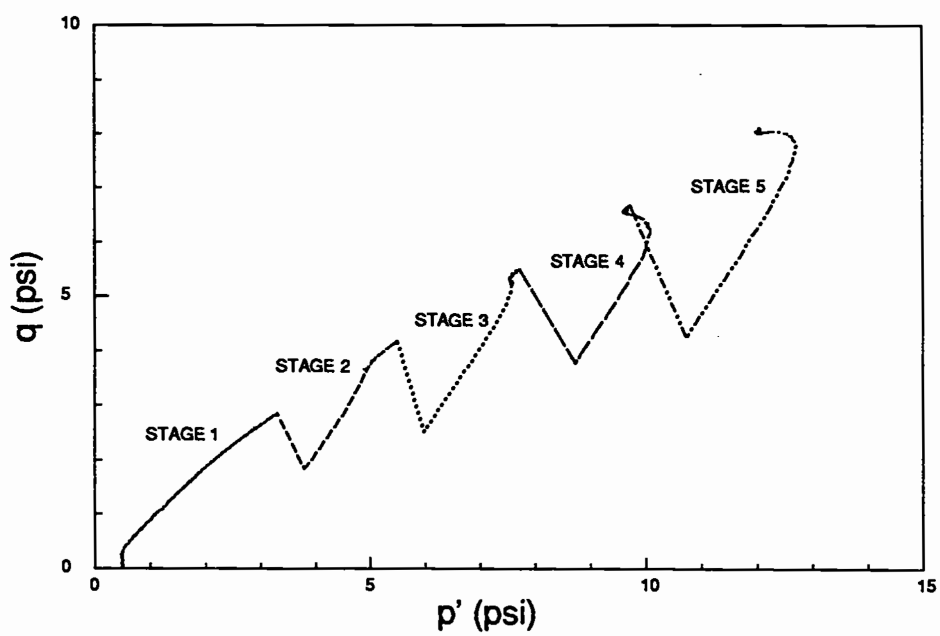

Figure 23. Stress Path - Sample Compacted Above Optimum Moisture Content.

For the purposes of the multiple stage tests conducted as a part of this work, Kenny and Watson's failure criterion along with the appearance of the stress-strain curve were used to determine when to stop a stage. During the first stage, it was always difficult to make the decision on where to stop. At the confining pressures utilized, failure typically occurred at less than two percent axial strain. At these low failure strains, it was possible to run three or more stages for each sample making it less important to run the first stage to exactly the right strain level.

4.4.3 Determination of Failure Envelope

The Mohr-Coulomb failure envelope is easily determined using the K_f line located by the stress paths. The equation of the K_f line may be described by:

$$q_f = a + p_f \tan\psi$$

where $a = q -$ axis intercept

$\psi =$ angle between K_f line and horizontal

The relationship between the K_f line and the Mohr-Coulomb failure envelope is described by (Holtz and Kovacs, 1981):

$$\sin\phi = \tan\psi$$

and

$$c = \frac{a}{\cos\phi}$$

Thus, the Mohr-Coulomb failure envelope is easily determined from the K_f line. For the three tests illustrated in Figure 24, 25, and 26; the shear strength parameters are found to be as shown in Table 1. More specific information on the soils tested and the actual test conditions is presented in Appendix C.

Table 1. Strength Parameters for Three Compaction Moisture Conditions

TEST[1]	a (psi)[2]	ψ	c' (psi)	φ'
3% Below Optimum	1.15	27.7	1.35	31.6
Near Optimum	1.28	28.4	1.52	32.7
3% Above Optimum	0.89	30.9	1.11	36.8

[1]Compaction moisture conditions are referenced to optimum moisture content (OMC) as determined by AASHTO T99. Because the samples were compacted in a different mold, the OMC for the test conditions is not the same.

[2]Pounds per square inch.

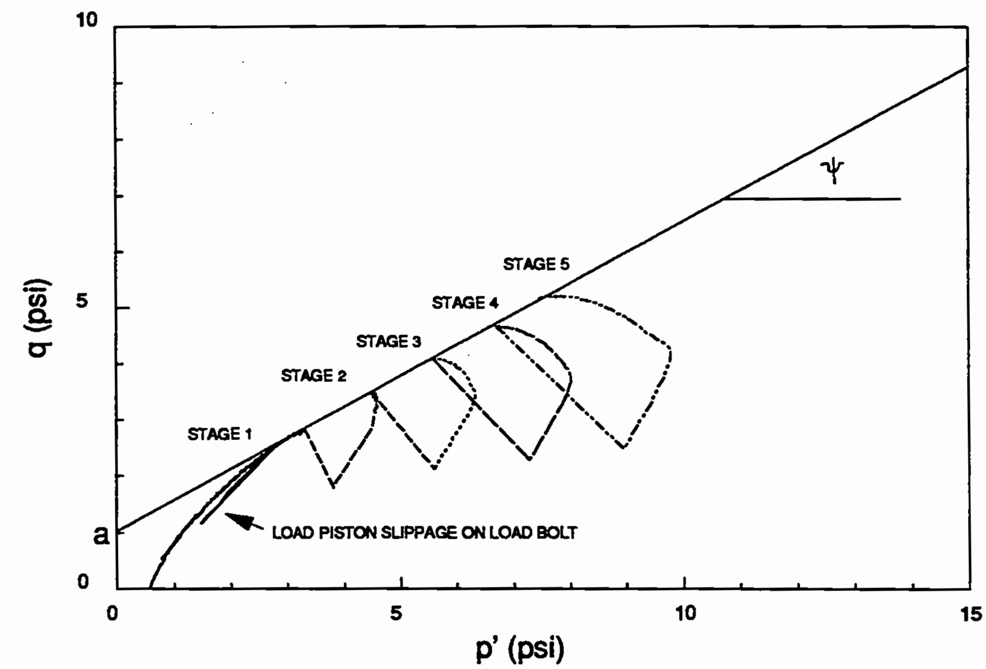

Figure 24. Failure Envelope - Sample Compacted Below Optimum Moisture Content.

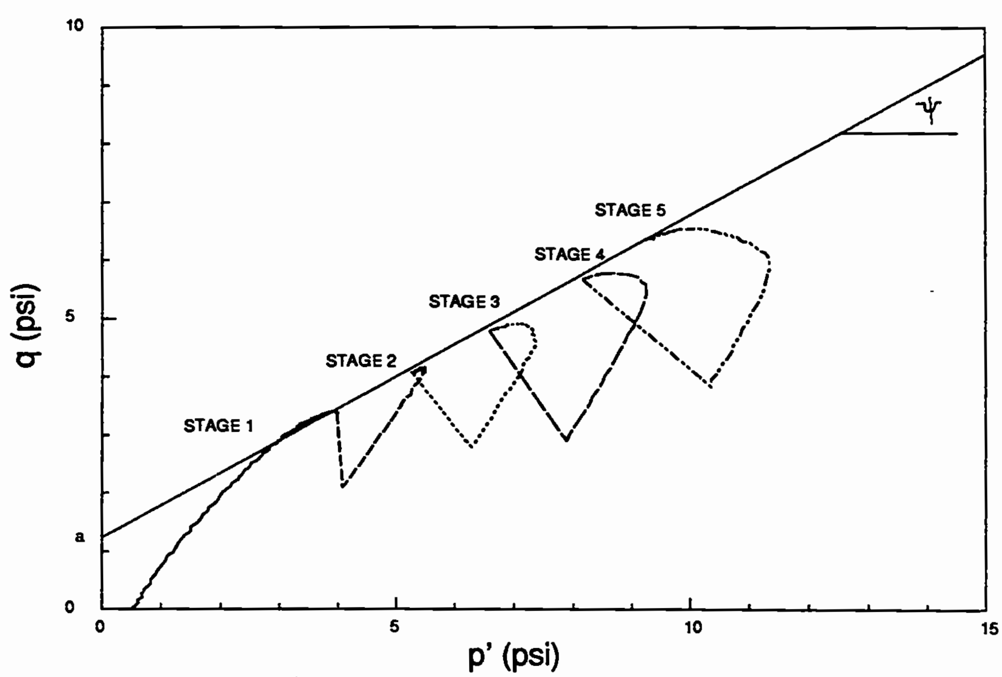

Figure 25. Failure Envelope - Sample Compacted Near Optimum Moisture Content.

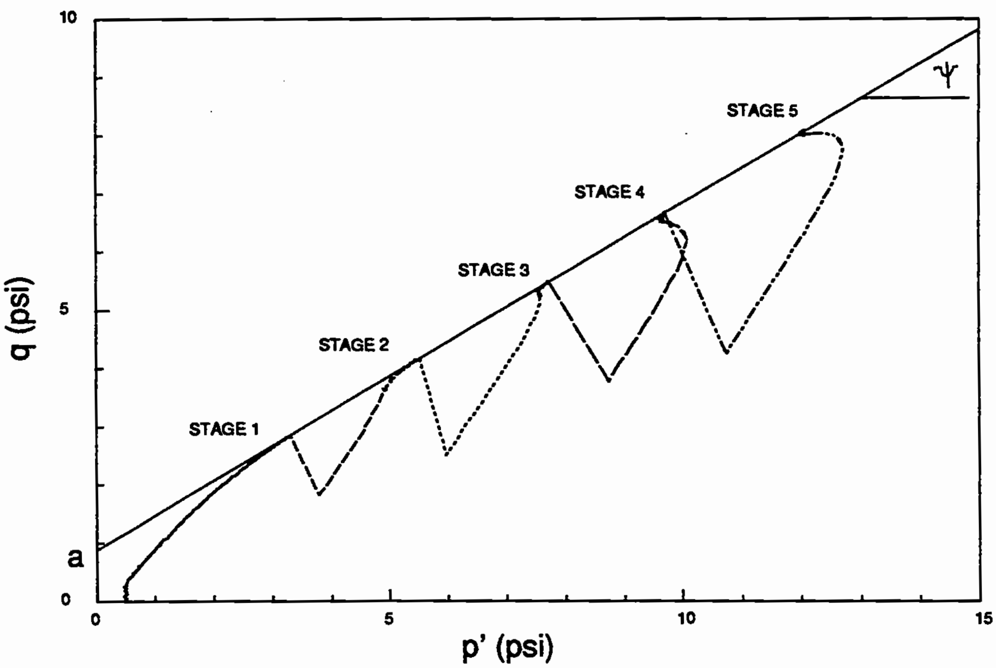

Figure 26. Failure Envelope - Sample Compacted Above Optimum Moisture Content.

5 CONCLUSIONS

Soils located in the upper layers of the subgrade for aggregate surfaced roads typically experience confining pressures below 5 psi. Traditional triaxial tests for civil engineering applications typically are conducted using confining stresses greater than 5 psi to determine the shear strength parameters of a soil. Thus, if the shear strength parameters for aggregate road subgrades are desired, traditional tests procedures must be modified to reflect the lower confining stresses experienced by these soils.

Unfortunately, it is not simply a matter of reducing the confining stresses while leaving all else the same. Low-confining stress tests require special attention to detail, because errors that are insignificant in traditional tests may be very significant to these tests. Arrangement of test apparatus, configuration during zeroing, magnitude of pressure application, failure criteria, and other seemingly small details can be very important to the test results. The following recommendations are made about test procedures.

1. The effective stress transducer should be zeroed before the sample is placed in the cell following the procedure previously described.
2. The vertical position of the triaxial cell relative to the triaxial control board must be maintained throughout the test.

3. The use of a slave regulator to increase cell pressure and back pressure simultaneously during back pressure saturation eliminates the need for two valves to be incremented simultaneously.
4. The pressure bias caused by the use of a slave regulator, during back pressure saturation, may be reduced or eliminated by changing the position of the triaxial cell relative to the triaxial control board. (This must be done prior to zeroing the effective stress transducer.)
5. Taylor's curve fitting method is the most suitable for determining the coefficient of consolidation. This is because it is difficult to determine whether measured changes in sample volume are the result of consolidation or system fluctuations.
6. For multistage tests, failure should be defined as occurring when the stress path becomes tangent to the K_f line. Alternative methods, such as the maximum effective principal stress ratio may be more appropriate for single stage tests. However, if low confining stresses are used, measurement noise will be magnified by the calculation of effective principal stress ratio.
7. Compaction in an atmosphere of carbon dioxide will reduce the time and pressure required for back pressure saturation of the sample.
8. Compaction in a Teflon coated, split mold reduces sample damage associated with removal from the mold.

The development of test procedures for low-confining pressure, multistage triaxial testing of compacted cohesive soils has been discussed in some detail. Although, the procedure was only validated with tests on a single soil type compacted under various conditions, the procedures should be appropriate for other moderately plastic soils. Because of the sensitivity of low-confining stress triaxial tests to small changes in equipment configuration and test procedures any changes to procedure should be carefully considered.

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APPENDICES

APPENDIX A: METHOD FOR CHECKING SKEMPTON'S B PORE PRESSURE PARAMETER

The progress of back pressure saturation can be monitored by determining the magnitude of Skempton's B pore pressure parameter. According to Skempton (1954), the pore pressure response Δu to a change in the total stress during undrained loading may be described by,

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

where:

Δu = pore presssure change

$\Delta\sigma_1$ = major principal stress

$\Delta\sigma_3$ = minor principal stress

B, A = Skempton's pore pressure coefficients

If $\Delta\sigma_1 = \Delta\sigma_3$, as is the case when cell pressure is increased, then Skempton's equation becomes:

$$\Delta u = B(\Delta\sigma_3)$$

which rearranges to:

$$B = \frac{\Delta u}{\Delta\sigma_3}$$

Thus, determination of B requires the measurement of pore pressure, u, in the undrained mode before and after the application of a known deviator stress. The test apparatus being used operates in a differential mode so that pore pressures and deviator stresses cannot be measured directly. To determine B, measurements must be made for three system configurations/conditions: Equilibrium, Set-up Increment, and Response to Increment. Equilibrium describes the conditions in the sample and the cell prior to incrementing the cell pressure and measures the initial effective stress in the sample. Set-up Increment cuts the sample off from the pressure board and then applies an increased cell pressure. Response to Increment closes the sample to drainage, but maintains a connection with the EST so that the effective stress after the cell pressure increment can be measured. The determination of B can be easily made using these three measurements and the equations discussed below. The equipment configuration is shown in Figure 27 and the valve configuration as well as the pressures seen by the EST are summarized in Table 2.

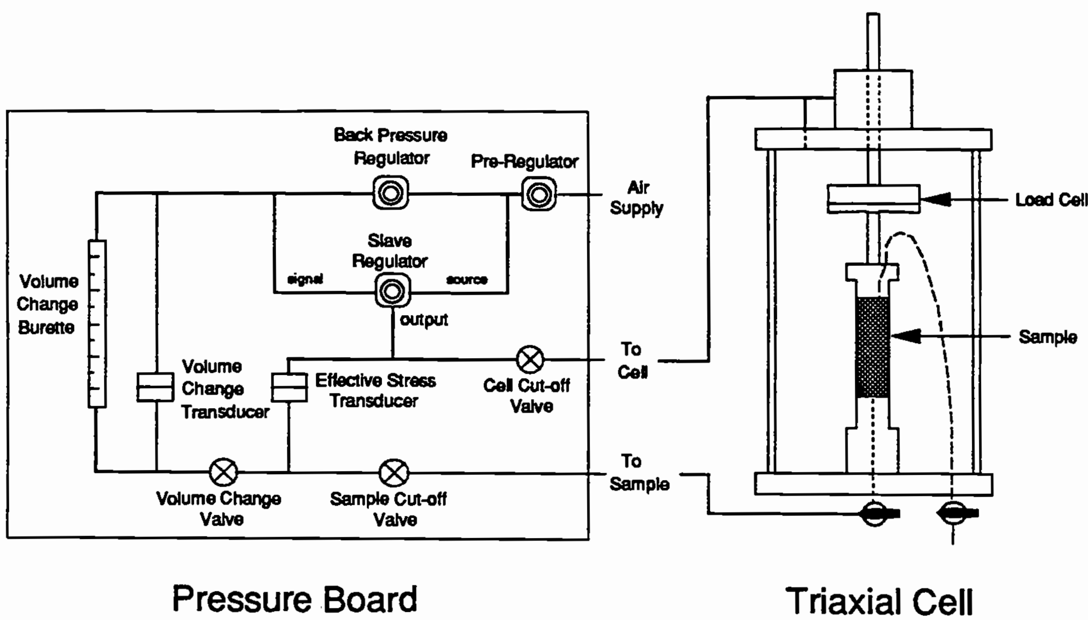

Figure 27. Configuration for Skempton's B Check

Table 2. Valve Configuration for B Check

	Equilibrium	Set-up Increment	Response to Increment
Valve Configuration	VC valve open. SC valve open.	VC valve open. SC valve closed.	VC valve A closed. SC valve open.
Top	Back pressure plus slave (currently 0) or the initial cell pressure, σ_c.	Back pressure plus slave (currently $\Delta\sigma_c$) or the cell pressure, $\sigma_c + \Delta\sigma_c$	Back pressure plus slave (currently $\Delta\sigma_c$) or the cell pressure, $\sigma_c + \Delta\sigma_c$
Bottom	Back pressure, which is the same as the initial sample pore pressure, u_o, if the sample is in equilibrium.	Back pressure, which is the same as the sample pore pressure before increment, u_o	Sample pore pressure after increment, $u_o + \Delta u$
Read	Initial effective st ress,$\sigma'_{3(o)} = \sigma_c - u_o$	$\sigma'_{3e} = \sigma_c - u_o + \Delta\sigma_c$	Effective stress after increment, $\sigma'_{3inc} = (\sigma_c + \Delta\sigma_c) - (u_o + \Delta u)$

As described previously, the test configuration used during saturation is a situation where $\Delta\sigma_3 = \Delta\sigma_1 = \Delta\sigma_c$, so B may be calculated using:

$$B = \frac{\Delta u}{\Delta\sigma_c}$$

Δu can be calculated using:

$$\begin{aligned}\Delta u &= \sigma_{3e} - \sigma_{3inc}\\ &= (\sigma_c - u_o + \Delta\sigma_c) - [(\sigma_c + \Delta\sigma_c) - (u_o + \Delta u)]\\ &= \Delta u\end{aligned}$$

and $\Delta\sigma_c$ can be calculated using:

$$\begin{aligned}\Delta\sigma_c &= \sigma_{3e} - \sigma_{3(0)} \\ &= (\sigma_c - u_o + \Delta\sigma_c) - (\sigma_c - u_o) \\ &= \Delta\sigma_c\end{aligned}$$

Resulting in an equation for B as follows:

$$\begin{aligned}B &= \frac{\sigma_{3e} - \sigma_{3inc}}{\sigma_{3e} - \sigma_{3(o)}} \\ &= 1 - \frac{\sigma_{3inc} - \sigma_{3(o)}}{\sigma_{3e} - \sigma_{3(o)}}\end{aligned}$$

APPENDIX B: DATA ACQUISITION SOFTWARE

The primary purpose of this document is to discuss the development of test procedures for low-confining stress testing of compacted cohesive soils. An important part of this procedure is the data acquisition software that was developed to conduct these tests. Portions of this software are unique to the computer system and the test apparatus in use and will not be discussed further in this document. Other portions are applicable regardless of the hardware in use. The purpose of this appendix is to briefly discuss the major components of the software, with particular emphasis on details that might easily be overlooked and on special features that facilitate test operations. Because software operations are indirectly influenced by hardware operations, some discussion of procedures that are not directly software related are included for clarity.

Development of data acquisition control software that ties together all components of a multistage triaxial test is no small task. Before undertaking the development of such software, the benefits of an integrated package, with real time data reduction and a graphics interface, over using a modular package that requires the user to tie the data together outside the program should be considered. The development of an integrated package was undertaken for this project because large numbers of similar tests were expected to be conducted during and after the project and it was desired to have real time interpretation of when failure conditions had been reached.

The data acquisition software that was developed contains four major phases: test initialization and control, saturation, consolidation, and shear; and numerous subroutines that are shared among the major phases. In the discussion that follows, each of the major phases is considered individually.

Test Initialization and Control Phase

The primary purposes of this phase are to control housekeeping activities such as initialization of variables and file handling; and to serve as an interface between the other phases.

Key Features

1. Initialization of variables. At the start of execution variables are initialized and the operator is queried for user defined inputs. System measured values for load, deformation, volume change, and effective stress are not zeroed during this phase.
2. Calculation of initial volume.
3. File handling. Initialization of data files and periodic saving of data is controlled by this phase. Automatic periodic saving of data is an important feature because, in the event of a power interruption, some portion of the test is still usable and it may be possible to restart the test.
4. Calibration factors. Default values for the calibration factors are contained in a data file that does not have to be re-entered every test. These factors are also interactively changeable by the operator.

5. Interface between other phases. Switching between phases is accomplished by the use of softkeys.

Special Considerations

Empty/Refill the Volume Change Burette (VCB).

At times during a test, it may be necessary to add or remove water from the volume change tube. Prior to the start of the test this process is very simple, but during the test this process is complicated by two important facts:

1. The system is under pressure. Because the system is under pressure, the reservoir to or from which water is added must also be under pressure or the water will rapidly leave the pressurized system. A careful sequencing of valve operations (hardware dependent) must be completed to move water in or out of the VCB under pressure.
2. Removal or addition of water to the VCB makes the original initial value inappropriate for all future determinations of volume change. To deal with this a new initial value for the volume change reading must be determined. The new initial value is determined by subtracting the operator caused volume change (removal of fluid is considered a positive volume change) from the original initial value. This operation can also conducted from the saturation and consolidation phases.

Saturation Phase

The purpose of this phase is to establish initial values for deformation, volume change, load, and effective stress; and to monitor the progress of back pressure saturation. This phase must be run or the initial values listed above will not be established.

Measured Values

During the saturation phase the following values are measured and recorded:

1. Elapsed time from the beginning of the test.
2. Amount of water moving in or out of the sample.
3. Effective stress.

Key Features

1. The time between recorded data points is user controllable and may be easily changed at anytime during the phase.
2. The progress of back pressure saturation can be checked at any time by requesting a check of Skempton's B pore pressure parameter. The software directs the operator through the process of valve changes and pressure increments and then calculates B. A more complete discussion of the method used is contained in Section 4.2.3 and in Appendix A.

Special Considerations

1. Zeroing. The timing and procedure for zeroing the test system is very important. It is not essential that the sample deformation LVDT, volume change transducer (VCT), and the load cell reading be physically zeroed at the beginning of the test, however the initial value must be known so that changes to the reading can be calculated.

   Zeroing of the effective stress transducer (EST) is much more critical then for the LVDT, VCT, and the load cell. Problems encountered because of improper zeroing of the EST are discussed in Section 4.2.1. Because small pore pressures develop during flooding, the EST must be zeroed prior to the start of flooding. To insure that a true zero effective stress condition exists during zeroing, the following procedure was used:

   System Setup: Triaxial cell in test position
   No sample in cell
   Cell filled with water
   No external pressure applied
   Both sides of the EST open to the same pressure

   For ease of use, the EST reading was physically zeroed in this configuration. Thus, subsequent readings directly give the effective stress after application of the calibration factor.

The following procedure was used to "zero" the LVDT, VCT, and load cell readings:

System Setup: Triaxial cell in test position
Sample loaded, cell filled with water
No external pressure applied
Load piston locked in contact with load bolt
LVDT in place
Volume change tube approximately 3/4 filled with water

The initial or "zero" value is then read for the LVDT, VCT, and the load cell. The initial reading may be made before or after flooding. This reading is subtracted from the reading at any subsequent time and the result is multiplied by the appropriate calibration factor to obtain the deformation, volume change or load at that time. The initial values may be changed during the test to reflect changes to the system such as, addition or removal of water from the volume change tube by the operator or artificial loads on the load cell caused by cell pressure changes.

2. Reduction of the effects Slave Regulator Bias. The use of a slave regulator to simultaneously increase cell pressure and back pressure results in a small pressure bias. The output from the slave regulator is slightly higher than the output from the back pressure regulator even when the slave regulator is fully closed.

The actual back pressure seen by the sample also includes the pressure head from the fluid in the sample volume change tube (VCB). If the height of the water column in the VCB is increased, by lowering the cell relative to the VCB, the difference between cell pressure and sample pressure is reduced.

The amount of pressure bias from the slave regulator increases as pressure increases (Figure 12). If it is desired to have zero effective stress on the sample throughout the saturation phase, the pressure head due to the VCB must equal or exceed the maximum slave regulator bias that will occur. To maintain the desired zero effective stress on the sample, when pressure head exceeds the bias, it is necessary to apply additional pressure to the cell via the slave regulator.

In order to maintain zero effective stress on the sample, the additional slave regulator pressure applied must be adjusted throughout the saturation process. This adjustment is necessary because the slave regulator bias increases with increased back pressure and because the water level in the VCB drops as saturation proceeds. It is always desirable to have a slightly greater cell pressure than pore pressure (small positive effective stress). If the pore pressure exceeds the cell pressure (negative effective stress) the membrane surrounding the sample may balloon and fill with water from the VCB.

Consolidation Phase

The purpose of this phase is to monitor the progress of consolidation and to plot the progress of consolidation in real time.

Measured Values

1. Elapsed time since the beginning of the test.
2. Change in sample volume. The change in sample volume is determined by measuring changes in the height of the water column in the VCB.
3. Effective stress.

Key Features

1. The time increment between data points automatically doubles with each additional point up to a maximum increment of 30 minutes. The time increment is user adjustable. To allow the operator start data recording then open the sample to drainage 6 points are taken (approximately one per second) prior to any doubling of the time increment occurring.
2. The volume change versus $\sqrt{time}$ is plotted while test is in progress so that consolidation can be monitored visually. Each test stage is plotted independently.

Shear Phase

The purpose of the shear phase is to monitor the progress of the shear phase and to plot the stress-strain curve and the stress path as shear progresses.

Measured Values

The following values are measured during the shear phase:

1. Elapsed time from the beginning of the test.
2. The axial load.
3. Deformation (change in sample length).
4. Effective stress.

Calculated Values

The following values are calculated during the shear phase:

1. Adjusted area.
2. Deviator stress.
3. Strain.
4. p' and q.

Key Features

1. The time between data points is user adjusted. The decision to record a data point is made when a specified change in deviator stress, strain, or elapsed time has occurred. (For example an elapsed time of three minutes.)

2. Plots the stress-strain curve and the stress path in real time.

Special Considerations

1. Area adjustment. Since these tests are conducted undrained, there is no volume change. However, since the sample length is changing there must be changes occurring to the cross-sectional area in significant enough magnitude for volume to remain constant. If this change in area is not considered in the calculation of the deviator stress, the deviator stress will be over stated.
2. Multiple stage plots. If multiple stage plots are not utilized it is not possible to make use of the information of previous stages in the decision about when to stop shearing the sample.
3. Load bolt seating. Since the measurement of strain is based on the movement of the bottom of the cell relative to the reaction bar on the load frame, seating of the load piston in the load bolt is very important. If the load piston is not in contact with the load bolt when shearing begins, the gap will be measured as change in the sample length and strain. Careful seating of the load piston in the load bolt will eliminate this problem.

The software also provides an additional check on load piston seating. The software will not begin recording data or consider that strain has occurred until the axial load measured by the load cell increases.

APPENDIX C: SOIL PROPERTIES AND TEST CONDITIONS FOR EXAMPLE TESTS

The soil used for the example tests was a residual soil of volcanic origin obtained near Mary's Peak in the Oregon Coast Range. Soil samples were obtained by excavating from the top two feet of a road cut. Organic material was removed from the soil whenever possible during collection of the samples. Soil in this area of the Coast Range are subjected to extremely wet and extremely dry conditions during the course of most years.

The soil was a highly plastic silt with the physical properties shown in Table 3.

Table 3. Physical Properties

Liquid Limit (LL)	55.5%
Plastic Limit (PL)	45.4%
Plasticity Index (PI)	10.0
% Passing #200 Sieve	65.5%
Unified Soil Classification	MH
Optimum Moisture Content (standard AASHTO)	33.0%
Maximum Dry Density (standard AASHTO)	86.5 pcf[1]

[1]Pounds per cubic foot.

The three example tests (below, near, and above optimum moisture content) illustrated in the text were tested under the following conditions.

Table 4. Sample Test Conditions.

TEST[1]	INITIAL WATER CONTENT (%)	INITIAL DRY DENSITY (pcf)[2]	c′ (psi)[3]	φ′ (°)
Below Optimum	30.8	79.3	1.35	31.6
Near Optimum	33.6	83.3	1.52	32.7
Above Optimum	36.5	84.0	1.11	36.8

[1]Compaction moisture conditions are referenced to optimum moisture content (OMC) as determined by AASHTO T99. Because the samples were compacted in a different mold, the OMC for the test conditions is not the same.

[2]Pounds per cubic foot.

[3]Pounds per square inch.