ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
FACULTY OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

INVESTIGATION INTO SHEAR STRENGTH CHARACTERISTICS OF EXPANSIVE SOIL OF ETHIOPIA

By: Ayenew Zewdie

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A Thesis Submitted to the
School of Graduate studies of Addis Ababa University
In partial fulfillment of the requirements for the degree of
Master of Science in Geo-Technical Engineering

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Notations and Symbols

A_o	Initial cross sectional area of sample	mm^2
A’	Corrected area of sample	mm^2
C	Cohesion	kPa
D_o	Initial diameter of sample	mm
ε	Unit strain	mm/mm
FS	Free swell	percent
L_o	Initial height of sample	mm
LL	Liquid limit	percent
P	Axial force	kN
PI	Plastic index	percent
PL	Plastic limit	percent
S	Shear strength	kPa
SL	Shrinkage limit	percent
V	Volume of soil after swelling	cm^3
V_o	Volume of dry soil	cm^3
w	Moisture content	percent
б	Normal stress	kPa
$б_1$	Major principal stress	kPa
$б_3$	Minor principal stress	kPa
Φ	Angle of internal friction	degree

Abstract

Considerable research work has been done on the shear strength properties of most soils. However, very little information is available on the shear strength characteristics of expansive clays. This is mainly due to the fact that many investigators have concentrated most of their efforts towards finding solutions to the swelling behavior associated with these clays. However, the strength behavior of expansive clays need not be overlooked. Factors such as swelling and shrinkage, for example, influence the soil strength significantly and their effects are difficult to account for in most laboratory shear tests. Furthermore a thorough knowledge of shear strength is required to estimate the bearing capacity of expansive clay soils and also to evaluate the stability of dams, road and railway embankments built in these soils areas.

Therefore, in this paper, Unconsolidated Undrained triaxial shear strength tests were performed on unsaturated soil and on some saturated soil samples collected from different locations of Addis Ababa, unsaturated soil sample of Gambela and unsaturated soil sample of five major roads (i.e. Addis-Modjo, Addis-Ambo, Addis-Debreberhane, Addis-Weliso and Addis-Ghoastion roads). The purpose of the study was to know the shear strength properties of expansive soil. Range of values of shear strength parameters (cohesion and angle of internal friction) were obtained based on unsaturated soil samples.

According to the outcome of the research, the shear strength of expansive soil ranges from 30-150Kpa in cohesion and 3-25 degree in angle of internal friction in UU test on unsaturated soil. For saturated soil sample in UU test the cohesion ranges from 55-94Kpa. There is a decrease in strength in the saturated samples, which shows that the

degree of saturation and the suction pressure can have major influence on the shear strength of expansive soil.

1. INTRODUCTION

1.1 General

Expansive soil is defined as plastic clay soil that exhibits high volume change when its environmental conditions are altered from dry to wet. The degree of expansiveness depends on whether the soil mass contain active clay minerals or not. The most common active clay mineral is montmorillonite. The problems of expansive soil in civil engineering structures were first identified in the late 1930's. Since then so many countries reported the problem.

Expansive soil is a major problem soil of Africa and other parts of the world. Large parts of Ethiopia are covered by black and gray expansive soil. The expansive clays of Ethiopia are residual, derived from the weathering of basic volcanic rocks, which cover part of Ethiopian. In the capital, Addis Ababa, it has been noticed that expansive soils covers large parts of the city where recent constructions are carried out. Most of the structural damage on expansive soil results from the differential rather than the total movement of the foundation soil as a result of swell. Damages can occur within a few months following construction, may develop slowly over a period of about 5 years, or may not appear for many years until some activity occur to disturb the soil moisture equilibrium.

Expansive soils are difficult to use in the construction of highways, airfields and lightweight structures, because such light structures can't exert the necessary counter load to overcome the swelling. Substantial damage has been occurring in Ethiopia on buildings and roads that are constructed on expansive soil with sever economic consequences, psychological effects and loss of proper functioning of structures. Thus a detailed investigation and research should be carried out on expansive soil to know in more detail the behavior of the soil and make remedial measures that are safe and economical. Economic consequences resulting from failures associated with expansive soils are substantial. Structural crack may occur if their foundations are not adequately designed to withstand the stresses and strains caused by alternate heaving and shrinkage of the foundation soil. Cracks do not only affect the structural safety and aesthetics of the building but also bring about additional financial burden to owners for repair if the structure is to be salvaged at all.

The degree of expansiveness of the soils varies from place to place depending upon type of parent material, climate and topography. Amount of clay minerals in the soil, thickness of expansive soil zone, and thickness of the active zone also affects the degree of shrinkage and swell.

Types of structures most often damaged from swelling soil include foundation and walls of residential and light (one-or two-story) buildings, highways, canal and reservoir linings, and retaining walls. Lightly loaded one-or two-story buildings, warehouses, residences, and pavements are especially vulnerable to damage because these structures are less able to suppress the differential heave of the swelling foundation soil than heavy, multi-story structures.

1.2 Back ground

Expansive soil is known to be widely spread in Ethiopia. Although the extent and range of distribution of this problematic soil has not been studied thoroughly: the southern, south-east and south-west part of the city of Addis Ababa areas, where most of the recent construction are being carried out and central part of Ethiopia following the major trunk roads like Addis-Ambo, Addis-Woliso, Addis-Debre Berhan, Addis-Gohatsion, Addis- Modjo are some of the areas covered by expansive soils. Areas like some part of Mekele, Gondor, Bahirdar, Debreberihan and Gambela are also known to be partly covered by expansive soils. These areas are the place where this research is being carried out.

1.3 **Objective**

The objective of this research is to determine the shear strength properties of expansive soil. The shear strength can be determined through laboratory and field test. In the laboratory tests there are three methods, which are direct shear, triaxial and unconfined compression tests. In this research unconsolidated undrained triaxial test has been performed on unsaturated and saturated samples to get the shear strength parameters. The main reason for performing the UU test was

- In Ethiopia lateritic and expansive soils dominate the clay soil formation. These clay soils are under partially saturated condition as the physical condition for full saturation such as melting of snow cover, exposure to large water body like ocean, sea, etc rarely exists.
- When expansive soil gets in contact with water, it starts to expand and suction (negative pore pressure) will be developed. By measuring or controlling the volume change in the specimen, we can check weather the samples saturate or not. But the problem is that during shearing some of the pressure of $б_3$ (the confining pressure) will go to balance the suction. And the value of $б_3$ that applied for shearing may not be known, unless special apparatus is used.
- It is difficult to saturate expansive soil samples with in a short period of time. To saturate one sample it will take more than a week and with more number of samples tested it would be difficult to run the test with in the specified time.

However, UU test was also performed on saturated soil specimen to see the effect of saturation on the shear strength parameters. Unconfined compression tests were also done as a supplementary test. The shear strength of soil is one of the most important geo-technical engineering properties. The bearing capacity of shallow or deep foundations, slope stability, retaining wall design and pavement design are all influenced by the shear strength of the soil.

The shear strength is measured in terms of the soil parameters, cohesion C and angle of internal friction (Φ). Thus the main objective of the research is to investigate the shear strength parameters(C and Φ) of expansive soils of Ethiopia by carrying out laboratory tests on different samples.

1.4 **Scope of Investigation**

This research addresses the above objectives and provides correlations with other properties of expansive soils. For this intended purpose, disturbed samples were collected from different locations of Addis Ababa and along the five major roads. Index property test were made on these samples and classification was made according to different classification systems [Legesse,2004]. For samples that showed expansiveness undisturbed samples were collected and shear strength tests, i.e., unconsolidated undrained triaxial and unconfined compression test were done on unsaturated and some saturated samples. Besides these, there were also other parallel researches, which were done by other researchers on expansive clay soil from different places, which this research is part of. The result of this study can serve as a basis for further study of expansive clay soil found in the country.

1.5 Organization of the Thesis

The thesis is organized into 6 chapters. Chapter 1 presents the general description and major engineering problems associated with expansive soils. The origin, formation, mineralogy and the different characteristics of expansive soils are presented in chapter 2. The shear strength test on expansive soil is discussed in chapter 3. Chapter 4 presents the test that was made on expansive soils and the results obtained from the tests. Discussion on the results obtained from the tests is given in chapter 5. Conclusion and recommendations are given in chapter6.

2. LITRATURE REVIEW

2.1 Origin of Expansive soil

The origin of expansive soil is related to a combination of conditions and processes that results in the formation of clay minerals having a particular chemical and mineralogical make up, which, when in contact with water expands. Variations in the conditions and processes may also form other clay minerals, most of which are non expansive. The conditions or processes that determine the clay mineralogy include composition of the parent material and degree of physical and chemical weathering to which the materials are subjected.

2.1.1 Parent Material

The constituents of the parent material during the early and intermediate stages of the weathering process determine the type of clay formed. The nature of the parent material is much more important during these stages than after intense weathering for long periods of time.

The parent materials that can be associated with expansive soils are classified into two groups (Grim,1962). The first group comprises the basic igneous rocks and the second group comprises the sedimentary rocks that contain montomorillinite as a constituents.

The basic igneous rocks are comparatively low in silica, generally about 45% to 52%. Rocks that are rich in metallic base such as the pyroxenes, amphiboles, biotite and olivine fall within this category. Such rocks include the gabbros, basalts and volcanic glass. Shale and clay stones are one of sedimentary rocks that contain montmorillonite as a constituent. Limestone and marls rich in magnesium can also weather to clay.

These constituents of the shales and clay stones contain varying amount of volcanic ash and glass, which are subsequently weathered to montmorillonite.

2.1.2 Drainage

Topography has a major influence on drainage characteristics that in turn is known to have major effect on soil mineralogy. Its control over soil properties is particularly strong in tropical environments reflecting the importance of lateral movements of water and soil material.

Gentle slopes, usually less than 3^0 results in slow movements or stagnation of water to maintain high concentration of Ca for the synthesis of motmorillonite.

2.1.3 Climate

Climate is the principal factor governing the rate and type of soil formation. The two important components of climate are the amount and distribution of precipitation, and temperature. The temperature variable is adequately represented by mean annual temperature. According to Vant Hoff's principle the velocity of chemical reaction increases by a factor of 2 or 3 for every 10 ^{0}C rise of temperature [Grim,1962]. The two main rainfall parameters most widely available are the mean annual rainfall and length of the dry season. The amount and distribution of precipitation affects the availability of moisture and the relative humidity of the soil atmosphere; it influences the concentration or chemical activity of solutions in the system. Warm climate with alternating dry and wet seasons is favorable for the formation of montmorillonite.

2.2 Distribution of expansive soils

Potentially expansive soils can be found almost anywhere in the world. In the underdeveloped nations, many of the expansive soil problems may not have been recognized because of less intensity of construction. It is to be expected that more expansive soil regions related problems would be reported each year as the amount of construction increases. Expansive soils are in abundance where desiccation phenomenon is common i.e., where the annual evaporation exceeds the precipitation. The problem of expansive soil is widespread through out the five continents.

2.3 Mineralogical Characteristics

Two units are involved in the atomic structure of most clay minerals. One unit consists of closely packed oxygen or hydroxyls in which aluminium, iron or magnesium atoms are embedded in octahedral coordination. They are equidistant from six oxygens or hydroxyls. When aluminium is present, only two-third of the possible positions are filled to balance the structure, which gives the gibbsite structure and has the formula $Al_2(OH)_6$. When magnesium is present, all the positions are filled to balance the structure which is the basic structure and has the formula $Mg_3(OH)_6$. [Grim, 1962].

The second unit is built of silica tetrahedrons. In each tetrahedron, silicon atom is equidistant from four oxygen's, or hydroxyls if needed to balance the structure, arranged in the form of tetrahedron with the silicon atom at the center. The silica tetrahedral groups are arranged to form a hexagonal network, which is repeated indefinitely to form a sheet of the composition $Si_4O_6(OH)_4$. The tetrahedrons are arranged so that the tips of all of them point in the same direction, and the bases of all tetrahedrons are in the same plane. [Grim, 1962]

The above mineralogical structures lead to the formation of different types of clay minerals, namely kaolinite, halloysite, montmorillonite, illite, etc. But the most common minerals found in clay soils are grouped as kaolinite, illite and montmorillonite. They are essential hydrous aluminum silicates. Since clays minerals are products of chemical weathering of rocks, both climate, which detremines weathering, and the parent rock, influences the type of minerals found.

Montmorillonite, the principal clay mineral of expansive soil is made up of identical units of alumina octahedral sheets between two silica tetrahedral sheets. These sheets are bound rather loosely and are very unstable in water. Water molecules easily insert themselves between the sheets and this is the cause for the highly expansive nature of clays containing this mineral group.

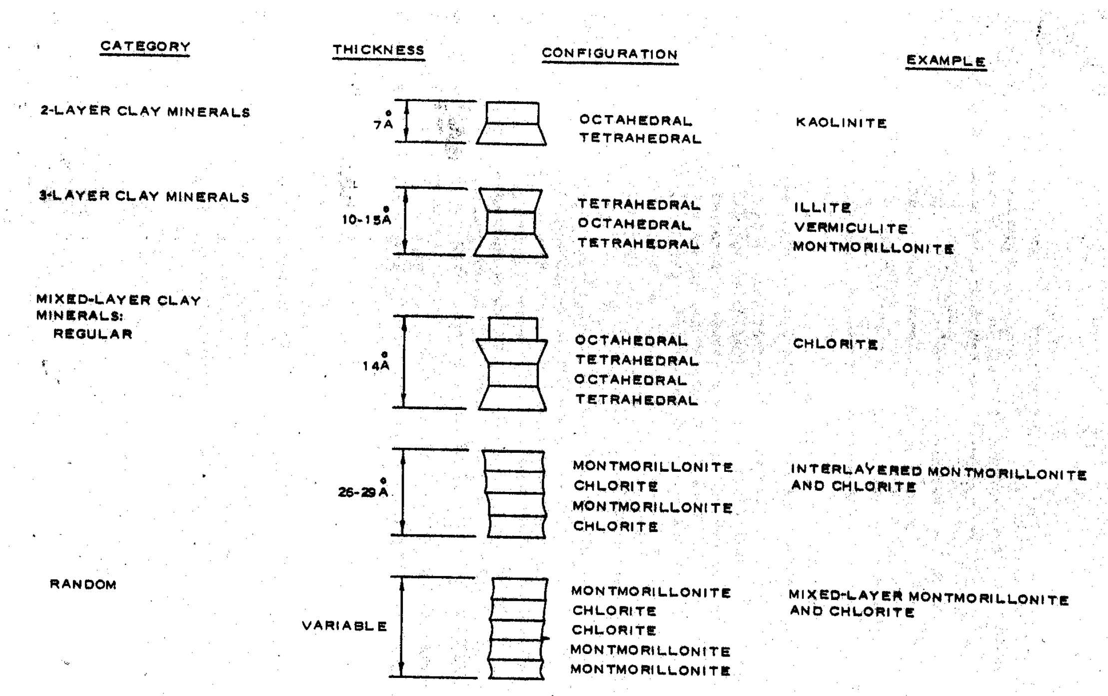

Fig.2.1 Typical structural configuration of clay minerals [Grim, 1962]

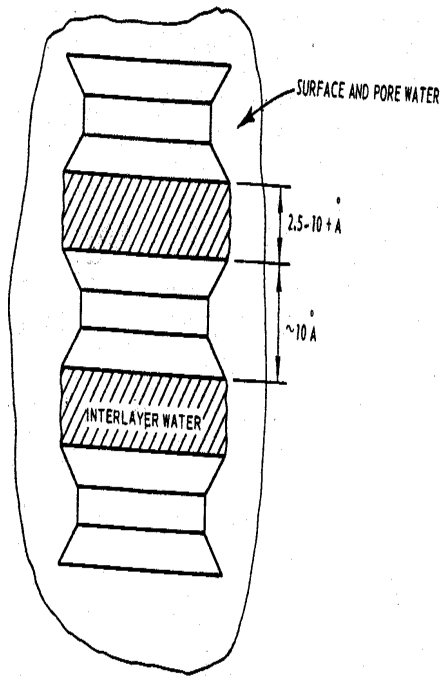

Fig. 2.2 Deflocculated clay mineral showing surface and interlayer water [Grim, 1962]

2.4 Identification and classification of expansive soil

Currently identification and classification of expansive soils are either on direct measurements of swell or on correlation of simpler test results with swell potential. These are referred to as direct and indirect techniques, respectively.

The key to all expansive soil classification systems is the method of measuring swell potential, since soils are rated by their measured swell potential. Swell potential may be measured directly in swell test or indirectly determined by correlation with other test results of swell test data. In almost every case swell potential is evaluated in the laboratory in a consolidation test device. This may yield swell potentials different from those for in-situ soils. Thus an accurate correlation between swell potential and other test results for a purpose of prediction of in-situ heave is difficult. These procedures, however, do provide good indicators of swell potential when the soil is subjected to the conditions used in the laboratory test.

2.4.1. Field identification

Some of the important field identification method that indicates the potential for expansiveness of a soil is the following: [Chen, 1966]

- A shiny surface is easily obtained when a partially dry piece of the soil is polished with a smooth object such as the top of a fingernail.
- The wet samples of the soil is sticky and it will be relatively difficult to clean the soil from the hands
- The appearance of cracking in nearby structures
- They usually have a color of black and/or gray
- Significant seasonal moisture variation in the region

- Open or closed fissures, (a joint or similar discontinuity)
- Slickenside, (highly polished or glossy fissure surface)
- Shattering or micro-shattering, (presence of fissures forming granular fragments of clayey soils)

2.4.2 Laboratory identification

Generally, there are three different method of identifying expansive soil in the laboratory.

i. Mineralogical identification

This method is used for identifying the mineralogy of clay particles such as characteristic crystal dimensions, characteristic reaction to heat treatment, size and shape of clay particles and charge deficiency and surface activity of clay particle. These properties are a fundamental factor controlling expansive soil behavior. [Chen, 1966]

The various techniques under these methods are

-X-ray diffraction

-Differential thermal analysis

-Dye absorption

-Electron microscope

-Base exchange capacity, etc

But these methods are not suitable for routine tests because of the following reason; they are time consuming, require expensive test equipment and, the results are interpreted by specially trained technicians.

ii. **Indirect methods**

These methods include simple soil property test that a practicing engineer resort to use for identifying expansive soil. Such tests are easy and can be performed in average soil mechanics laboratory, and yield an excellent indices of expansive properties. The commonly used test here is the Atterberg limit.

In this method, measurement of the plasticity and the shrinkage characteristics of the soil are conducted for identification of soils and provide a wide acceptable means of rating by using liquid limit, plastic limit, shrinkage limit, free swell tests, colloid content test, etc.

iii. Direct measurement

The most accurate and dependable method of determining the swelling potential and the swelling pressure of expansive clay is by direct measurement.

2.4.3 Classification of Expansive Soils

The parameters determined from expansive soil identification tests have been combined in a number of different classification schemes. But before using any soil classification system, engineers should understand the data base from which it was derived and establish its limitations; otherwise, poor reliability and lack of confidence in the system may result. The classification system used for expansive soils are based on indirect and direct prediction of swell potential, as well as combinations, to arrive at a rating.

Classification based on indirect predictions of swell potential

An indirect prediction of swell potential includes correlations based on index properties, swell, physical indicator and a combination of them.

The classification system developed based on single property alone such as: based on activity (Skempton, 1953), based on shrinkage limit and linear shrinkage (Altmeyer, 1956), based on index property (Kanntey and Brink, 1952), etc are difficult to use alone as a classification system because they may lead to wrong conclusion. Combining index property, swell and physical indicator can develop a better indirect classification system. Good examples for such classification methods are:

i. Bureau of reclamation method

This method is based on direct correlation of observed volume change with colloid content, plastic index and shrinkage limit. The classification is as follows[Chen, 1966]

Table 2.1 Classification of expansive soils based on Bureau of Reclamation method

Colloid content, %-1μm	PI, %	SL, %	probable expansion %	Degree of expansion
>15	<18	>15	<10	low
13-23	15-28	10-16	10-20	Medium
20-31	25-41	7-12	20-30	High
>28	>35	<11	>30	Very high

ii. Chen method

In this method, a correlation is made between swell data and particle size %< No. 200 sieve, liquid limit, and standard penetration resistance. The classification is as follows: [Chen, 1966]

Table 2.2 classification of expansive soils according to Chen

<No. 200 sieve %	LL	SPT Blows/ft	Probable Expansion,%	Degree of Expansion
<30	<30	>10	<1	Low
30-60	30-40	10-20	1-5	Medium
60-90	40-60	20-30	3-10	High
>95	>60	<30	>10	Very High

2.5 Mechanics of Swelling

2.5.1 General

Swelling in expansive soils will take place if there is change in the environment. Environmental change can consist of pressure release due to excavation, desiccation caused by temperature increase, and volume increase because of the introduction of moisture. By far the most important element for swelling is the effect of water on expansive soils. With the introduction of water volumetric expansion takes place. If pressure is applied to prevent expansion, the pressure required to maintain the initial volume is the swelling pressure. [Chen, 1966]

2.5.2 Moisture Transfer

The pattern of moisture migration depends on the geological formation, climatic condition, topographic features, soil types and ground water level. The most common method of moisture transfer is by gravity. The seepage of surface water, precipitation, and snow melting into the soil are common examples. The moisture migration can occur in all direction. Moisture migration can be caused by different reasons. Fractures and fissures, shrinkage cracks, capillary force, vapor transfer, thermal gradients, etc are some of the sources that cause moisture migration and swelling on expansive soils. [Chen, 1966]

2.5.3 Moisture Equilibrium

In natural ground, the moisture content of the partially saturated soil is in general equilibrium with the applied stress, the forces due to evaporation and transpiration at ground surface and the capillary forces. When building or pavement covers the area, the evaporation and transpiration forces are eliminated and a new set of equilibrium must be established. The new equilibrium requires the flow of moisture compatible with the new condition. The force causing the moisture change or flow is termed soil suction. [Chen, 1966]

2.5.4 Depth of Moisture Fluctuation

In covered area there is no gain or loss of moisture to the atmosphere. The moisture content of the soil decreases with depth as shown in curve 1 of Fig.2.3. In uncovered natural conditions evaporation and transpiration causes loss of moisture content in the soil near the ground surface. Hence the moisture content will increase with depth. However the influence of evaporation decreases with depth and at some depth, H_d, the moisture content equilibrium remains the same as the covered condition. The value of H_d depends on the climatic condition, type of soil, and the location of the water table. This depth represents the total thickness of the material, which has a potential to expand because of change of moisture content. The maximum depth of H_d is equal to the depth of the water table, and the minimum depth is equal to the depth of the seasonal moisture contents fluctuation (H_s). During wet months with heavier precipitation and higher humidity, the moisture content of near surface soil increases and the moisture profile represented by curve 2 alters its shape to curve 3.

The watering of lawns, planting of trees and shrubs, discharge of roof chains, formation of drainage channels and swales, and the possibility of utility line leakage will all increase the value of H_s.

When areas are covered by structures such as buildings, pavements, sidewalks or aprons evaporation is blocked or partially retarded. The moisture content beneath the covered area decreases due to gravitational migration, capillary action, and vapor and liquid thermal transfer and, in course of several years, the depth of seasonal moisture content fluctuation H_s can approach to the depth of desiccation H_d. [Chen, 1966]

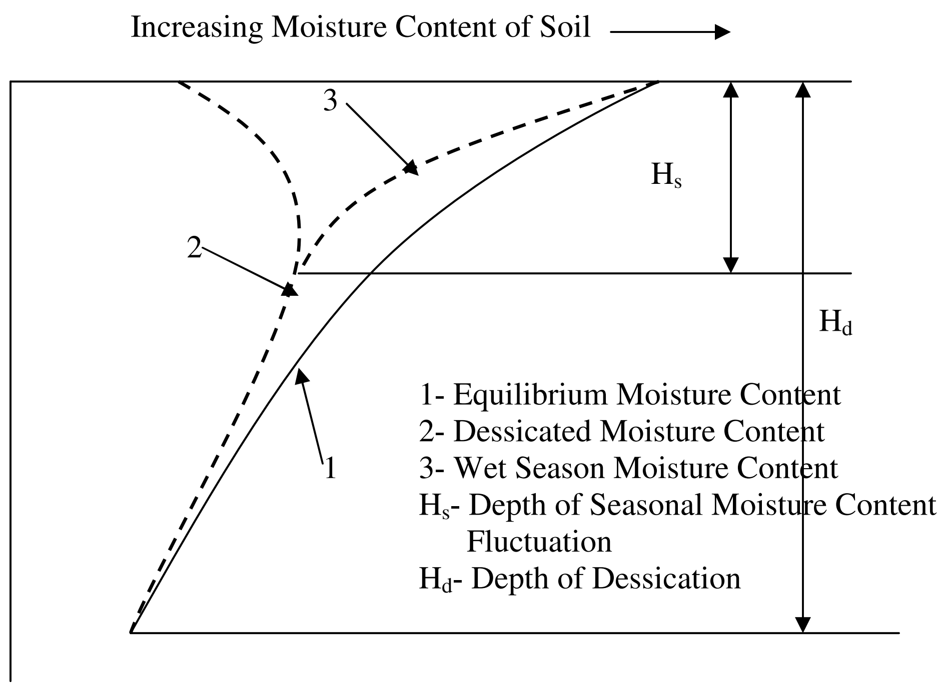

Fig. 2.3 Moisture content variation with depth below ground surface [Chen, 1966]

3.0 SHEAR STRENGTH TESTS

3.1 General

The shear strength of soil is one of the most important aspects of geo technical engineering. The bearing capacity of shallow and deep foundations, slope stability, retaining wall design and pavement design are all influenced by the shear strength of the soil. Structures and slopes must be stable and secure against total collapse when subjected to maximum anticipated applied loads. Thus limiting equilibrium method of analysis is conventionally used for their design, and these methods require determination of the ultimate or limiting shear resistance (shear strength) of the soil. [Holtz and Kovacs, 1981]

The shear strength can be determined in several different ways. In situ methods such as the vane shear test or penetrometers avoid some of the problems of disturbance associated with the extraction of soil samples from the ground. However, these methods only determine the shear strength indirectly through correlations with laboratory results or back calculated from actual failures. Laboratory tests, on the other hand, yield the shear strength parameters more directly. In addition, valuable information about the stress-strain behavior and development of pore pressures during shear can be obtained.

The shear strength of a soil is measured in terms of a limiting resistance to deformation offered by a soil mass or test sample when subjected to loading or unloading. The limiting shearing resistance corresponding to the condition generally referred to as 'failure', can be defined in several different ways. It is the resistance developed from a combination of particle rolling, sliding, and crushing and reduced by any excess pore pressure that develops during particle movement. The shear strength of a test sample is measured in the laboratory by subjecting it to certain defined conditions and carrying out a particular kind of test. Failure can occur in the soil as a whole, or within limited narrow zones referred to as failure planes.

There are different criteria of 'failure', from which the shear strength of a soil is determined. They are [Holtz and Kovacs, 1981]

i. Maximum deviator stress

The maximum or peak deviator stress is the stress that is associated with 'failure' in the testing of soil samples. It is the condition of maximum principal stress difference. If the vertical and horizontal total principal stresses are denoted by σ_1 and σ_3 respectively, the peak deviator stress is written $(\sigma_1 - \sigma_3)_f$, and the corresponding strain is denoted by ε_f.

ii. Maximum principal stress ratio

If the principal effective stresses σ_1', σ_3' are calculated for each set of readings taken during an undrained test, values of the principal stress ratio σ_1'/σ_3' can be calculated and plotted against strain. The maximum value of the ratio occurs at about the same strain as the peak deviator stress in many undrained test on normally consolidated clays.

iii. Limiting strain

This criterion is not often used in multistage drained triaxial tests. However, for soils in which a very large deformation is needed to mobilize the maximum shear resistance, a limiting strain condition might be more appropriate.

Three types of laboratory tests are commonly used to determine shear characteristics of soils. These tests are the direct shear test, the triaxial compression test and the unconfined compression test. The material characteristics that can be determined from these tests are the strength parameters (angle of internal friction, and cohesion). In some triaxial tests properties related to volume change such as modulus of elasticity and Possion's ratio can be obtained. These parameters are used for analysis and design in conventional civil engineering problems relating to slope stability, bearing capacity and any other situations where shear strength controls.

It should be noted, however, that laboratory strength test are meaningful only if the laboratory conditions of loading, drainage etc adequately represent the actual field conditions and also the soil sample being tested is representative of the insitu soil. Out of the three types of tests mentioned above, the triaxial compression test is more versatile and simulates the in situ conditions better. Therefore, it is used for this study.

3.2 Triaxial Compression Test

In triaxial compression tests used in this study a cylindrical soil sample of standard size (dia=38mm and height=76mm) is encased by a thin rubber membrane and placed inside a chamber. For conducting the test, the chamber is filled with water and the sample is subjected to a confining pressure by application of pressure to the water in the chamber. Axial (or deviator) stress is applied through a vertical loading ram. The main advantages of this type of test, it include the possibility to control drainage conditions can be controlled and the availability of apparatus and procedures for measuring pore pressure during the test. [Parry, 1995]

In triaxial tests, the distribution of stress throughout the sample is not uniform because of restraint on the sample due to the end caps. The end caps that transmit the vertical load cause a horizontal shear and hence create restraint on the end of the sample. For this reason it is important to ensure that the length of the sample is not less than about twice its diameter. The length to diameter ratio is controlled by two factors; with length greater than two-and-a half times the diameter there is a danger of side buckling; with length less than one-and- a half times the diameter the whole of the sample is restrained by the friction of the end plates on the end of the sample. Therefore, a length-diameter ratio (one-and-a half to two-and-a half) is satisfactory. [Bishop and Henkel, 1962]

The shear strength of a soil is heavily dependent on the type of test, and pore water conditions, which may be generalized as follows:

i. Unconsoildated-undrained test (UU or U) -the sample may (or may not) be confined by a consolidation pressure and is simply tested to failure in compression or shear with out drainage. This is commonly termed an "undrained" test for strength s_u. This test gives Φ=zero for saturated soils and a range from 0 to Φ' for others depending on water content. The unconfined compression test is a U test with a failure compressive strength q_u.

ii. Consolidated-undrained tests (CU) -here the sample is consolidated by some pressure prior to failure in compression or shear.

iii. Consolidated-drained tests (CD) -here the sample is consolidated just like the consolidated undrained tests but during testing to failure the test is done so slowly that excess pore pressures from the shear strains are not sufficiently large to significantly affect the effective stresses.

In general, if excess pore pressure does not develop or are insignificant one obtains the effective stress parameters Φ' and C'. If we can measure the excess pore pressure we can compute both total stress and effective stress parameters.

Due to the difficulty to consolidate expansive soils and because of the fact that the consolidation swelling behavior is so complex and deserves special consideration and special laboratory arrangement only UU test is performed in this study.

UU test is performed with the drain valve closed for all phases of the test and before the sample has a chance to consolidate (if S<100%). The test is commenced immediately after the cell pressure is stabilized.

3.3 Shear Strength Characteristics

3.3.1 General

The shear strength is measured in terms of two soil parameters, cohesion or inter particle attraction, and angle of internal friction, Φ, the resistance to inter particle slip. Grain crushing, resistance to rolling, and other factors are implicitly included in these two parameters. This behavior is well represented by the Mohr-Coulomb failure criterion given as,

$$S = c + б \tan Φ \qquad [2.1]$$

Where: S= shear strength

б= normal stress on shear plane

c= Cohesion

The shear parameters are often taken as constant but they depend on drainage condition, previous stress history, and current state (particle packing or density or water content). Therefore, soils seldom exhibit unique strength parameters and obtaining accurate values is not a trivial task. Also while equation above (2.1) has a linear form, in real soils, for reasons just cited it is often nonlinear.

The shear envelope defined by the above equation obtained from the locus of tangent points to a series of Mohr's circles constitutes the limiting state of stress, which can be imposed on the soil. In general, since two parameters are involved, two or more tests must be performed for a graphical (most common) or simultaneous equation solutions. (Bowels, 1988)

In this study only UU test was carried out on unsaturated soil samples and five saturated soil samples because of the following reasons

- It is assumed that UU test could give a reasonable value for quick shear condition, which is one of the possible drainage conditions during shear failure in practice.
- To ensure that no pore pressure is induced in the specimen during shear for the materials with very low permeability, the rate of loading must be very slow. The time required to fail the specimen ranges from a day to several weeks. Such a long time leads to practical problems in the laboratory such as leakage of valves, seals, and the membrane that surrounds the samples for which the laboratory used in this study is not very adequate.
- There is a practical difficulty to consolidate expansive soil before the shearing phase in other tests and the consolidation pressure needed may exceed the swelling pressure and , thus the swelling potential may be affected.

3.3.2 Unconsolidated- Undrained (UU) test with out Saturation

In this test, the specimen is placed in the triaxial cell with the drainage valves closed from the beginning. Thus, even when a confining pressure is applied, no consolidation can occur if the sample is 100% saturated. Then the specimen is sheared undrained. The sample is loaded to failure in about 10 to 20min.; usually pore water pressures are not measured in this test. This test is a total stress test and it yields the strength in terms of total stresses. It is sometimes called Q-test (quick) since the sample is loaded to failure much more quickly.

Even if samples has to be tested based on saturation, in this research most samples are sheared with out saturation, except five samples where both saturated and unsaturated samples are tested for shear strength property. The Mohr- failure envelopes for UU test are straight for 100% saturated clays. All test specimens for fully saturated clays are presumably at the same water content (and void ratio), and consequently they will have the same shear strength since there is no consolidation allowed. Therefore, all Mohr circles at failure will have the same diameter and the Mohr failure envelope will be a horizontal straight line. The UU test, as previously mentioned, gives the shear strength in terms of total stresses, and the slope Φ_t of the UU Mohr failure envelope is equal to zero. The intercept of this envelope on the τ-axis defines the total stress strength parameters C, or $\tau_f = C$ where τ_f is undrained shear strength.

For partially saturated soils in case where the soils have high moisture content near saturation, a series of UU test will define an initially curved envelope until the clay becomes essentially 100% saturated due simply to the cell pressures alone. Even though the drainage valves are closed, the confining pressure will compress the air in the voids and decrease the void ratio. As the cell pressure is increased, more and more compression occurs and eventually, when sufficient pressure is applied, essentially 100% saturation is achieved. Then, as with the case for initially 100% saturated clays, the Mohr failure envelope becomes horizontal. [Holtz, 1981],however soils may also be tested for partially saturated condition in area where the physical condition for saturation does not exist. [Fredlund and Rahardjo, 1993]

3.3.3 Unconfined Compression Test

This test is a special case of UU test with the confining or cell pressure equal to zero. The stress conditions in the unconfined compression test specimen are similar to those of UU test, except that σ_3 is equal to zero. Practically speaking, for the unconfined compression test to yield the same strength as the UU test however, several assumptions must be satisfied. These are as follows:

1. The specimen must not contain any fissures, silt seams, or other defects.
2. The soil must be very fine grained.
3. The specimen must be sheared rapidly to failure.

Unconfined compression test is also called unconsolidated undrained test since the test was run by placing the specimen in the triaxial apparatus with $б_3$=0. [Holtz, 1981]

3.3.4 Previous Shear Strength Studies on Expansive Soil

Unfortunately very little information is available on the shear strength characteristics of expansive soils. But on paper by Saile E.L. and Bucher F. [1984] under the title "Shear Strength of Tropical Clays", they presented their findings based on Consolidated Undrained triaxial test on undisturbed samples. The conclusions they made after performing several tests was the extra highly swelling clay exhibits high strength and the natural water content have a significant influence on the shear strength of expansive clay. For detailed discussion of the test, the specific paper can be consulted. [Saile and Bucher, 1984].

4.0 LABORATORY TESTS

4.1 Laboratory Tests on Disturbed Samples

To determine the shear strength properties of a soil samples, laboratory test should be done on undisturbed samples. But it was time taking to run a large number of tests on undisturbed soil. So in order to select the locations for undisturbed samples, disturbed samples were collected first from different locations of Ethiopia. Thus

- 78 disturbed samples were collected from 18 test pits which are located at different locations of Addis Ababa.
- Both disturbed and undisturbed samples were collected from Gambela region.
- 25 disturbed samples were collected from 5 test pits on the major trunk roads.

The location for this test pits are shown in fig. 4.1. Then to classify as per classification system explained in chapter 2, index property tests like liquid limit, plastic limit, free swell, etc were made on disturbed sample. Based on the result obtained from the index tests, the location of undisturbed sample was selected for the locations where the soil shows more expansiveness and uniqueness. The summery of the index tests done is shown in Table 4.1., Table 4.2 and Table 4.3. Index tests were made following the ASTM procedure [ASTM, 1966].

Fig 4.1 Location of disturbed soil sample from Addis Ababa

Table 4.1 Summery of laboratory test result for disturbed samples from A.A. [Legesse, 2004]

Soil property	Value in %
Liquid Limit (LL)	96-121
Plastic Limit (PL)	24-29
Plastic Index (PI)	70-87
% retained in the 2µm sieve	30-85
% finer than 75 µm	49-86
Free swell (FS)	75-140

As per the classification system stated in chapter 2, the test results show that the expansive soil taken from the pits are highly expansive. The profile of most of the test pits show the Gray colour soil is overlain by dark (black) expansive soil. The average depth for the boundary between the gray and black expansive soils is 1.45m to 2m. [Legesse, 2004]

The number of samples collected from Addis Ababa was more due to the fact that expansive soils abundantly exists in the city and major construction works are in progress. Samples are also collected from other places such as Gambela and five major roads, which start from Addis Ababa and lead to the different regions as shown on Fig.4.2.

Gambela is one the region where expansive soil exists. Six test pits were dug from which both disturbed and undisturbed samples were taken. The test results on disturbed samples are shown in Table 4.2.

Fig. 4.2 Locations from which expansive soil samples are collected on major trunk roads

Table 4.2 Summery of laboratory test result on disturbed samples from Gambela [Legesse, 2004]

Soil property	Value in %
Liquid Limit (LL)	68-84
Plastic Limit (PL)	20-28
Plastic Index (PI)	40-56
% retained in the 2μm sieve	28-60
% finer than 75 μm	58-80
Free swell (FS)	33-75

Based on the classification system stated in Chapter 2 and the test result, the expansive soils of Gambela region have moderate to high degree of expansion. All major roads that begin from Addis Ababa cross through expansive soil deposits, samples were also taken from the five major trunk roads. These are Addis-Modjo road, Addis- Ambo road, Addis-Debreberhane road, Addis-Weliso and Addis-Gohasion road. A total of 23 disturbed samples were taken from these roads. Index property tests were performed on these samples in order to classify the soils and to select the representative locations for further undisturbed sampling. The test results on disturbed samples collected from the major trunk roads are summarized in Table 4.3.

Table 4.3 Summery of laboratory test result on disturbed samples from major trunk road [Legesse, 2004]

Soil property	Value in %
Liquid Limit (LL)	69-97
Plastic Limit (PL)	25-33
Plastic Index (PI)	41-66
% retained in the 2µm sieve	33-85
% finer than 75 µm	64-88
Free swell (FS)	50-120

Based on the different classification system presented in chapter 2, the test result in Table 4.3 show that the expansive soil samples taken from the different trunk roads fall in very high swelling potential group. And on the basis of this result one test pit was selected for undisturbed sampling along each roads for which extensive coverage of expansive soil exists.

4.2 Laboratory tests on undisturbed samples

4.2.1 Tests on undisturbed samples from Addis Ababa

The test results in Table 4.1 show that all the disturbed samples fall under very high expansive soil category. This allows the use of fewer samples for tests like Triaxial testing which requires more resource without lossing representation. Thus 17 samples were chosen from 9 test pits for undisturbed sampling. The distribution of the test pits were chosen in such a way that an even geographical distribution is attained. The locations of the test pits for undisturbed samples are shown in Fig. 4.3. The applied all-round pressure ($\sigma3$) was varied in the range of 200-500KN/m^2. The axial stress was then increased at a constant rate of strain until failure. The peak strength was recorded. Table 4.4 gives a summery of the unconsolidated-undrained triaxial test results. Fig.4.4-Fig.4.6 are plots of devaitor stresses versus axial strains and Fig. 4.7-Fig. 4.9 are Mohr circle diagrams at failure for some samples. Since the samples don't attain full saturation, the strength obtained is not only due to cohesion, but also due to the normal pressure applied. The unconsolidated undrained strength parameters of the soil under investigation were finally obtained by drawing tangent line (Mohr envelope failure line) to total stress Mohr circles.

Fig 4.3 Location of Undisturbed soil samples in the city of Addis Ababa

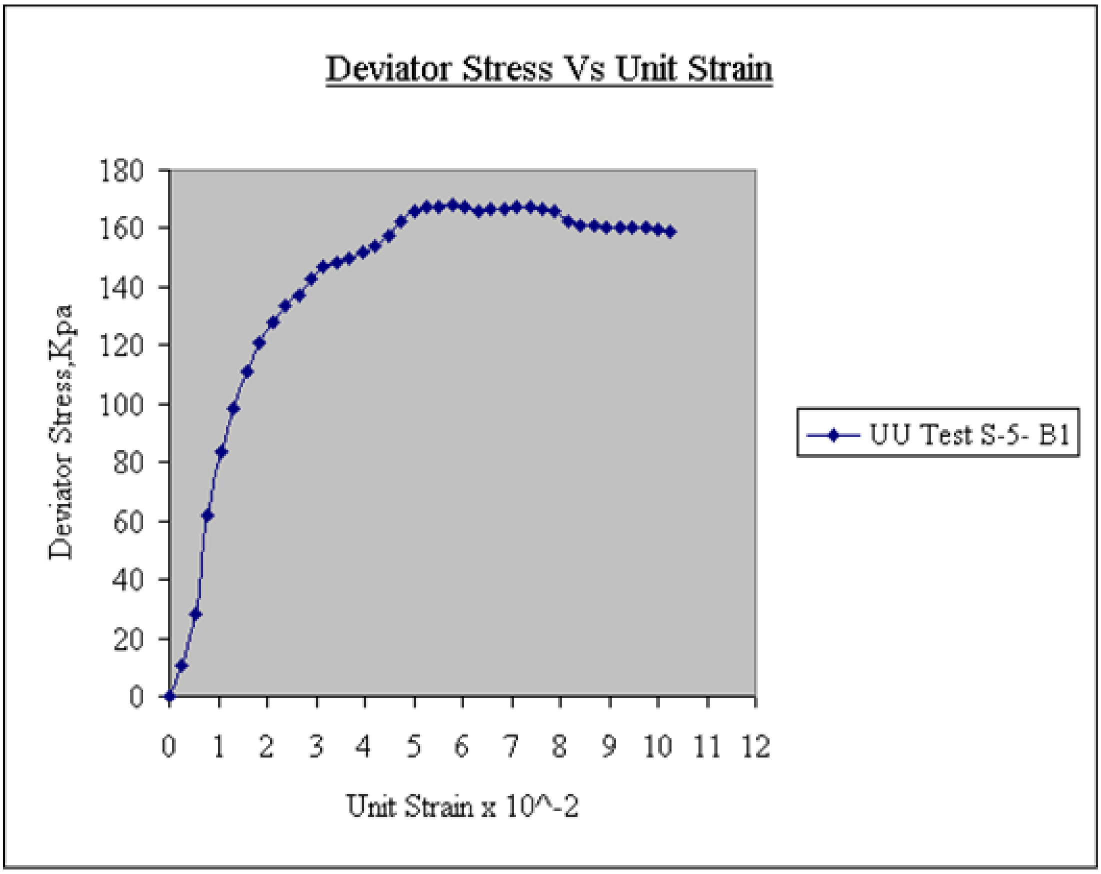

Fig 4.4 Deviator stress versus unit strain for S5(Mekanisa area) in UU test for Black soil

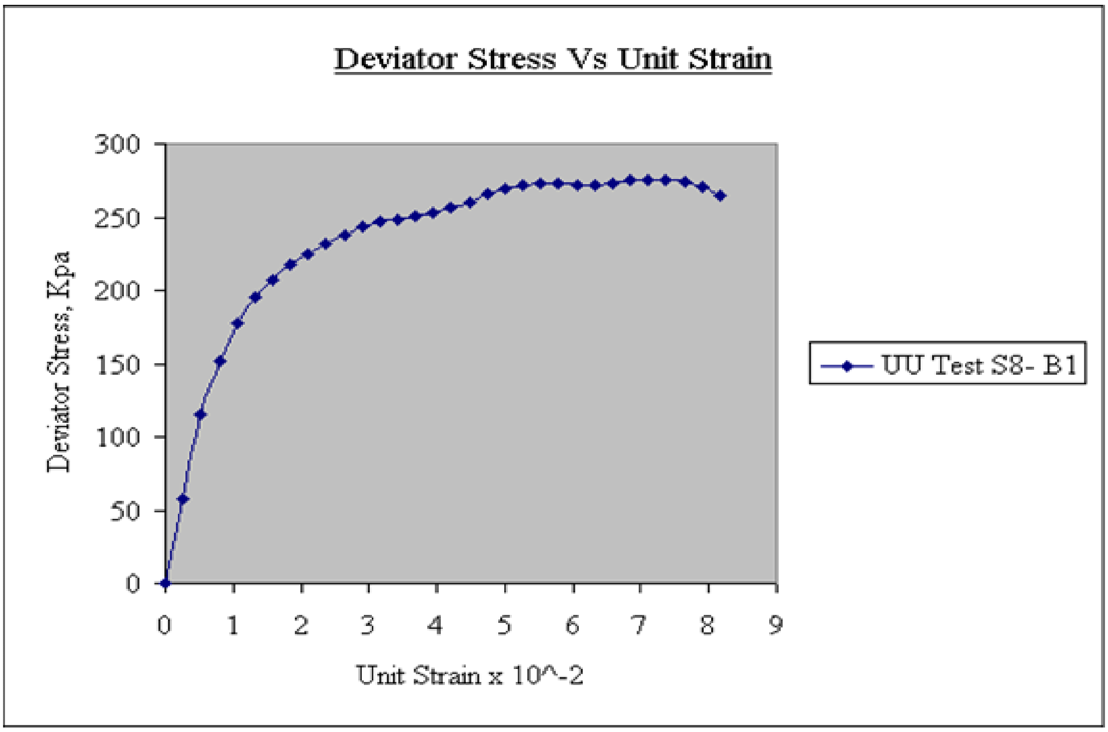

Fig 4.5 Deviator stress versus unit strain for S8(Old airport area) in UU test for Black soil

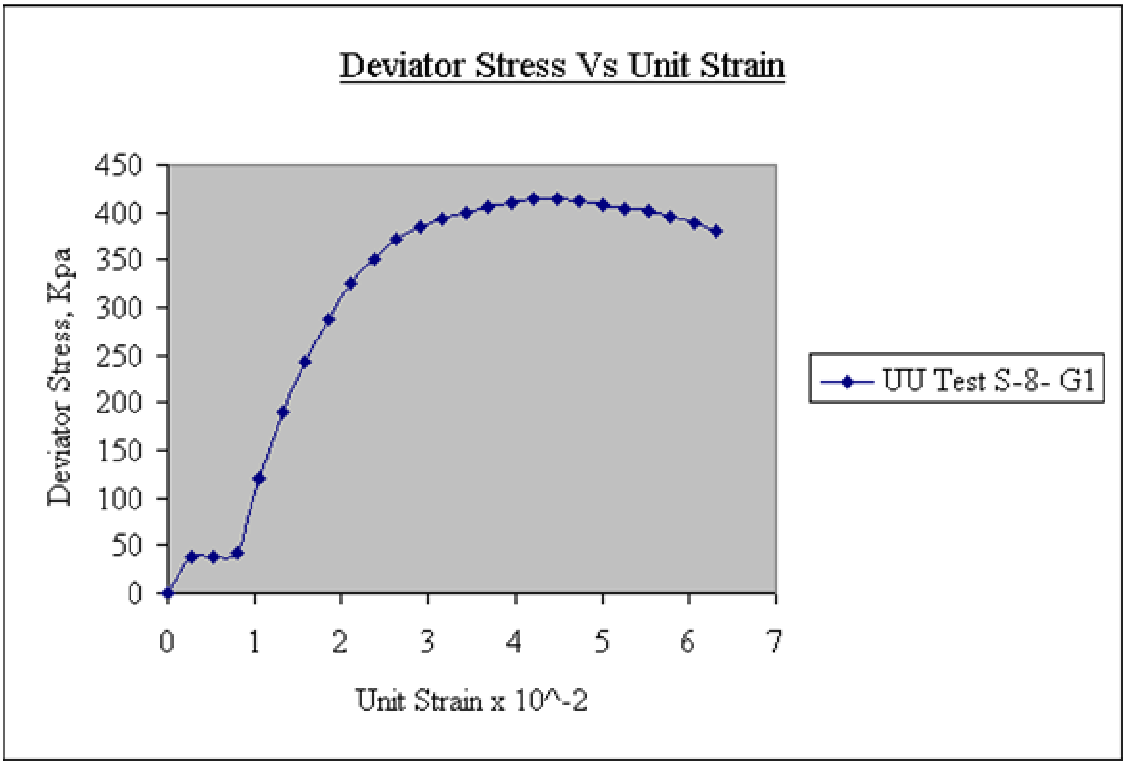

Fig 4.6 Deviator stress versus unit strain for S8(Old airport area) in UU test for Gray soil

Besides UU test, the unconfined compression tests were also carried out and the test results are presented in Table 4.6. Some of the stress-strain curves for unconfined compression tests are presented in Fig. 4.10, 4.11 and 4.12. Mohr circle at failure are also presented in Fig. 4.13, 4.14 and 4.15 for the above samples. The unconsolidated-undrained strength of the soil was obtained by drawing a horizontal line tangent to the circles as shown in the figures.

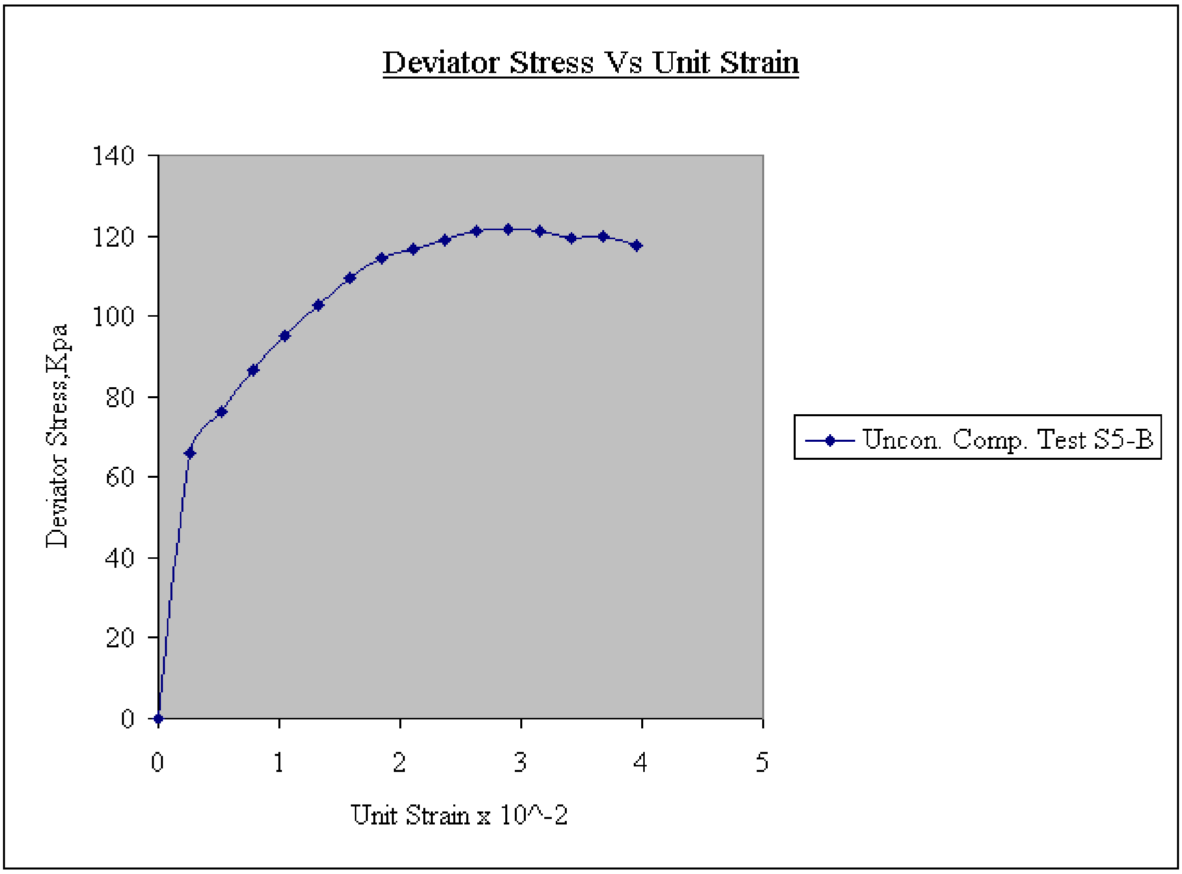

Fig 4.10 Deviator stress versus unit strain for S5 (Mekanisa area) in Unconfined Compression Test for Black soil

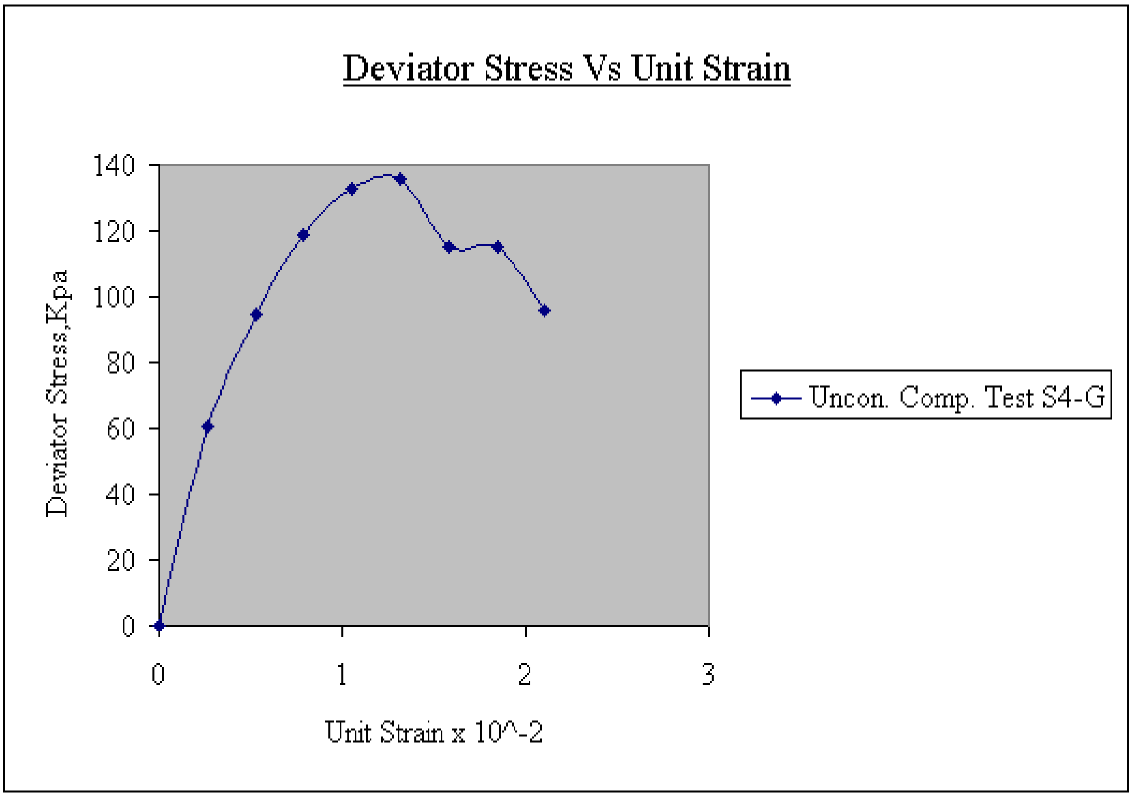

Fig 4.11 Deviator stress versus unit strain for S4(Ayat area) in Unconfined Compression Test for Gray soil

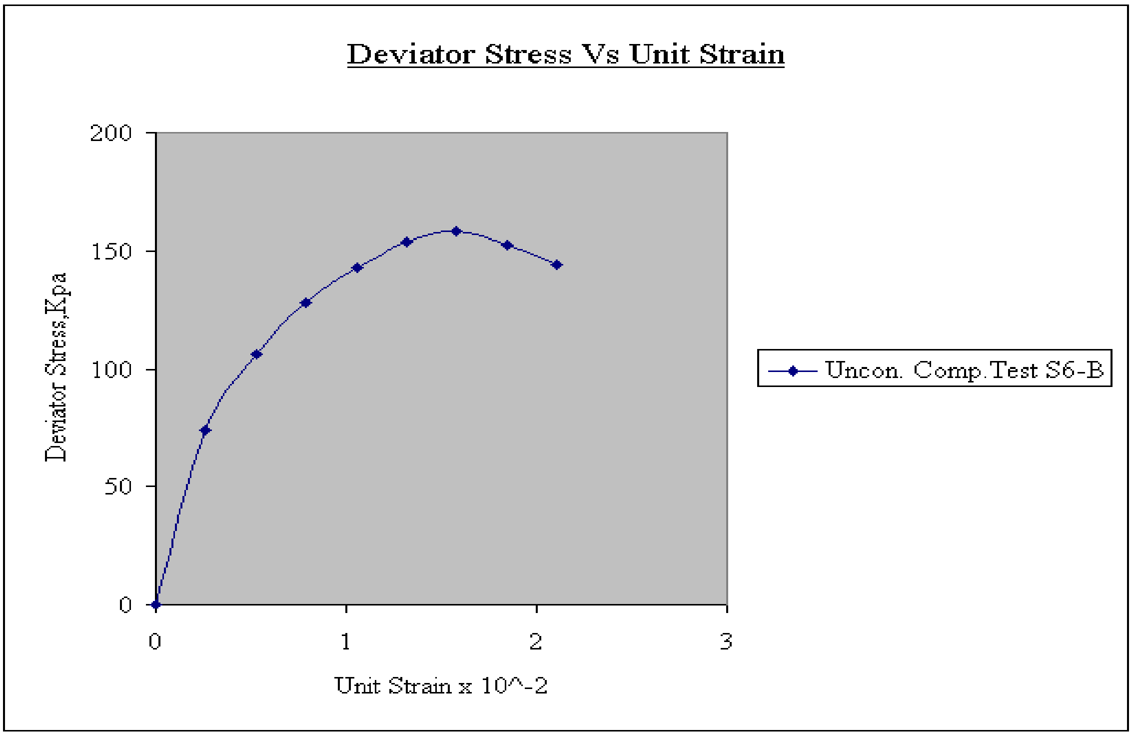

Fig 4.12 Deviator stress versus unit strain for S6 (Kality area) in Unconfined Compression Test for Black soil

Table 4.4: Summery of unconsolidated-undrained Triaxial Test on unsaturated samples and index property test for A.A. soils

Site Name	colure	Moisture Content %	Liqid limit(LL)	Plastic Limit(PL)	Plastic index(PI)	dry density	Specific Gravity	Cohesion	Ang. of Int. friction
S1	Black	38.4	101	24	77	1.25	2.77	29.85	25
S2	Black	37.5	110	34	76	1.24	2.78	31.98	9
	Grey	42	121	37	84	1.17	2.82	54.94	10
S3	Black	46	98	26	72	1.2	2.78	18.26	5
	Grey	41.9	101	28	73	1.22	2.83	65.00	0
S4	Black	41	96	26	70	1.22	2.79	96.58	15
	Grey	44.3	111	24	87	1.17	2.82	88.59	4
S5	Black	33.2	96	26	70	1.26	2.76	71.51	3
S6	Black	42	105	29	76	1.2	2.85	10.47	15
	Grey	49.6	110	26	84	1.1	2.81	43.37	5
S7	Black	50.8	96	25	71	1.09	2.79	75.00	9
	Grey	42.1	101	29	72	1.23	2.83	37.27	17
S8	Black	38	106	25	81	1.22	2.78	123.95	3
	Grey	41.5	109	26	83	1.2	2.81	148.46	9
S9	Black	46	100	24	76	1.14	2.77	70.16	7
	Grey	45.5	106	28	78	1.15	2.8	57.01	7

The result obtained were reasonable, but some of the results show big values of angle of internal friction, which are not expected in clay soil under undrained conditions, where the main shear strength parameters is cohesion, the UU test was also performed on saturated samples. Such a higher value of friction angle is due to the presence of granular and fragmented material.

The Unsaturated-Undrained test was done for 5 saturated soil samples for which the results are shown in Table 4.5. Some practical difficulties were encountered during saturation. The first was how saturation was to be done? Saturation is achieved when the air in the voids is expelled and water fills in the pores. This is observed by checking the volume change in the samples. When the volume change becomes constant, full saturation is assumed. But saturating the sample by applying the confining pressure at once, it was found that only the top part was properly saturated.

But, when the samples were left to saturate for more days with periodic increased application of the confining pressure, full saturation was achieved and in this case the pore pressure measured becomes equal to the confining pressure. In this way all five samples were saturated and the test was done.

Table 4.5: Results of unconsolidated-undrained Triaxial Test on saturated samples

Site Name	Unsaturated soil		Saturated soil	
	Φ	C	Φ	C
S4 Black	15	96.58	0	68.5
S5 Black	3	71.51	0	55.6
S7 Black	9	75	0	63.4
S8 Black	3	123.95	0	93.45
S1 Black	25	29.85	5.85	14.63

Unconfined compression test was performed with out applying confining pressure to the soil specimen throughout the test. The soil specimen is sheared by applying an axial load until failure is reached. The deviator stress, $(\sigma_1 - \sigma_3)_f$, is equal to the major principal, σ_1, since the confining pressure, σ_3, is equal to zero. Unconfined compression test on unsaturated soil will grately underestimate the available shear strength, but it may used to see the effect of confining pressure on shear strength property. [Fredlund and Rahardjo, 1993]. Unconfined compression test were made on certain undisturbed soil sample as shown on the next page.

Table 4.6: Summery of Unconfined Compression Test result for A.A. soil

Site Name	coloure	Moisture Content	Liqid limit(LL)	Plastic Limit(PL)	Plastic index(PI)	Cohesion=Undrained Strength
S1	Grey	39.6	105	25	80	147.5
S3	Black	46	98	26	72	22.5
	Grey	41.9	101	28	73	32.5
S4	Black	41	96	26	70	125
	Grey	44.3	111	24	87	67.5
S5	Black	33.2	96	26	70	62.5
S6	Black	42	105	29	76	80
	Grey	49.6	110	26	84	55
S7	Black	50.8	96	25	71	42.5
	Grey	42.1	101	29	72	130
S8	Grey	41.5	109	26	83	145
S9	Black	46	100	24	76	62.5
	Grey	45.5	106	28	78	90

The unconfined compression test of black soil sample of S1 shows very high result. This is because the samples contains pieces of gravel which magnify the result. So the result did not reflect exactly the undrained strength of the soil. It is not used in the analysis and shown only for completeness.

Unconfined compression test helps to see the effect of confining pressure on the shear strength of soils. But the main objective of this thesis is to see into the shear strength parameters of expansive soils of Ethiopia, and to see some correlations of shear strength with other index properties.

4.2.2 Tests on Undisturbed Soils Samples from Gambela

Another area investigated in this study was Gambela Region. Index test were done on disturbed sample from Gambela region [Legesse, 2004]. Triaxial (unconsolidated undrained) test were also done on 6 samples collected from the three test pits at different locations. The test results on these samples are shown in Table 4.7 below.

Table 4.7: Summery of unconsolidated-undrained Triaxial Test of sample from Gambela

Site Name	coloure	Moisture	Liqid limit(LL)	Plastic Limit(PL)	Plastic index(PI)	Cohesion	Ang. of Int. friction
G-AB1	Black	26.3	71	28	43	53.3	17
	Gray	26.08	75	32	43	165.6	15
G-GU1	Black	19.95	46	21	25	115.2	20
	Gray	26.53	70	28	42	159	13
G-GU2	Black	24.05	68	20	48	155.1	18
G-TA1	Black	22.86	69	29	40	148.6	15
	Gray	23.52	72	30	42	139.4	12

4.2.3 Tests on Undisturbed soil Samples from Major Trunk Roads

Samples were also brought from five major roads that starts from Addis Ababa were also tested. Here, the samples were collected from the roads during rainy period. Thus the samples have higher moisture content. Therefore additional disturbed samples with less water content were tested. To see the density-moisture relation, proctor compaction test was initially done. After the proctor test was done the sample were tested for shear strength in UU test based on the optimum water content and max. dry density. The result is shown in Table 4.8.

Table 4.8: Summery of unconsolidated-undrained Triaxial Test on samples from major trunk roads

Site Name	colure	Moisture	Liqid limit(LL)	Plastic Limit(PL)	Plastic index(PI)	dry density	Cohesion	Ang. of Int. friction
S1-Addis-Modjo Road	Black	40.02	94	34	60	1.185	75.59	16
S9-Addis-Ambo Road	Black	44.69	95	32	63	1.177	115.2	6
S18-Addis-Weliso Road	Black	42.51	96	30	66	1.215	93.65	10
S20-Addis-Ghasion Road	Black	45.57	73	35	38	1.20	106.17	2
S1*-Addis-Modjo Road	Black	40.3	94	34	60	1.188	148.82	1
S9*-Addis-Ambo Road	Black	43.4	95	32	63	1.181	112.03	3
S10*-Addis-Debreberhane Road	Black	38.5	88	32	56	1.235	145.05	1
S18*-Addis-Weliso Road	Black	39.3	96	30	66	1.252	150.41	1
S20*-Addis-Ghasion Road	Black	38.7	73	35	38	1.254	81.04	3

Note: * shows the samples are prepared from disturbed samples by compaction and let to stay for few days until they regained their original structure

5.0 DISCUSSION OF THE TEST RESULTS

The laboratory test results for the shear strength tests are tabulated and some of them are shown graphically in the previous chapter. The mineralogical composition test done on the same soil shows that the dominant expansive soil forming minerals are kaolinite, montomorillonite and illite. [Legesse, 2003]

The result of index property tests on sample collected from different part of the country are shown in Tables 4.1, 4.2, and 4.3 in chapter 4. The index property tests made on disturbed samples show that all expansive soils investigated fall under high to very high degree of expansive group.

The free swell test results shown in the same Tables vary from 75-140%. The grain size analysis made indicates that the soils tested have high percentage of clay fraction and consequently high values of Atterberg Limits as shown in Tables 4.1, 4.2, and 4.3. The Atterberg Limits range from 68% to 121% for Liquid Limit, from 21% to 33% for plastic limit. Specific gravity varied from 2.76 to 2.83 as shown in Table 4.4.

The shear strength parameters from undisturbed samples collected from different locations of Addis Ababa are shown in Table 4.4 with the index property test results. From the table we see that the angle of internal friction varied from 0 to 25^o and cohesion ranges from $10KN/m^2$ to $148.46KN/m^2$ for the total normal stress on unsaturated soil samples. This shows that the shear strength parameters of the soil under investigation vary with a wide range. In general soils do not have the same strength characteristics. However, these variations are also contributed mainly by the variation in moisture content. The strength of unconsolidated undrained specimen is entirely due to cohesion (which is independent of normal pressure) if the sample is 100% saturated. As it is seen in Table 4.4 sample S1black has Φ value of 25^o which is not expected in clay soil, but for the same sample the value of Φ decreased to 5.85^o upon saturation. When the sample is saturated it should give zero angle of internal friction. However this sample has Φ value different from zero. This is mainly due to the fact that the sample is not pure clay. The sample has 25% sand content and some pieces of gravel. These are the main reason for high value of Φ.

During the saturation process, it was too difficult to allow water to displace the air out of the samples and saturate samples in short time. It takes more than 10 days for one sample to saturate 100%. And, the shear strength parameters of saturated samples are lower than the parameters for unsaturated samples. Table 4.6 shows the result of unconfined compression test done on the same soil. As it is seen in the table, the soil that is grouped in the same degree of expansiveness shows a wide variation in shear strength. The unconfined strength varied from 20 to 147.5KN/m^2. These wider ranges conform to the UU test results. But in general it is observed that at higher moisture content the soil has a lower strength.

Table 4.7 shows the result of unconsolidated undrained (UU) test results for samples brought from Gambela region. The magnitude of angle of internal friction and cohesion range from 11-22^o and 53-165.6KN/m^2 respectively. These values are much greater than the strength parameters obtained from UU test of samples collected from Addis Ababa. As shown in the table, the moisture content of samples from Gambela region is lower than samples from Addis Ababa. This is the main reason for their higher strength parameters

Table 4.8 shows the result obtained from UU test for samples from main major trunks roads for undisturbed and compacted samples. Since the undisturbed soil samples were taken during rainy season, they have much water contents and their strength parameter is lower than the remolded samples. The magnitude of angle of internal friction and cohesion ranges 2-16^o and 35-115.2KN/m^2 respectively. But for the remolded samples at their optimum moisture content and max. dry density, the angle of internal friction and cohesion ranges 1-3^o and 80-148.82KN/m^2.

6.0 CONCLUSIONS AND RECOMMENDATIOS

The present research work has mainly to establish and have the ranges of values of the shear strength parameters of expansive soil in Ethiopia based on unconsolidated undrained triaxial test on unsaturated and saturated soil samples. The conclusion drawn from this study is that

1. The result obtained can't generalize the shear strength of expansive soil, but the result can show the shear strength of the soil at the specific conditions (unsaturated and saturated state). The shear strength parameters of expansive soil range 30-150Kpa in cohesion and 3-25^o in angle of internal friction in UU test on unsaturated soil.
2. The result obtained from five saturated soil sample has shown the shear strength of expansive soil range 14-94Kpa in cohesion and 0-5^o in angle of internal friction. This shows saturation reduces the shear strength parameters.
3. The shear strength of expansive soil ranges 22.5-147.5Kpa in unconfined compression test, but these values of strength should not be used, as unconfined test don't simulate the field conditions exactly.
4. Moisture has a significant effect on the shear strength property of expansive soil. The shear strength of expansive soil reduces when the moisture content increased. When the shear strength of expansive soil is mentioned it should always be referenced with the moisture content.

This study did not consider the stress history of the soil samples, which have a great influence on the shearing strength of soils. It is recommend to extend the research by considering the stress history of the samples through series of consolidation tests and to do UU and other types of triaxial tests on saturated soil samples. The shearing parameters in this study were obtained from UU test. The other types of triaxial test have also their own practical significance; hence it is suggested to include the other types of tests in the future study.

Fig. 4.2 Locations from which expansive soil samples are collected on major trunk roads

_________ shows the locations from which samples were collected from major trunk road and Gambela region

Fig 4.2 Locations from which expansive soil samples are collected on major trunk roads

7. References

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8. APPENDIX

Appendix A. Shear strength parameter computation

Shear strength test (UU Test)

Parameters and Formula used

1. Initial dimension of the specimen

   Initial height of the specimen, L_o=76mm

   Initial diameter of the specimen, D_o=38mm

   Initial cross-sectional area of the specimen, A_o=1134.1mm^2

   Initial volume of the specimen, V_o=86192.74mm^3

2. Calculation are made of axial strain and stress so that a stress-strain curve can be drawn. The unit strain ε is computed from mechanics as

   $$\varepsilon = \Delta L/L_o \quad \text{mm/mm}$$

   where ΔL= total sample deformation, mm

   L_o= original sample length, mm

3. In order to obtain the major principal stress $б_1$, it is necessary to know the confining chamber pressure since this pressure acts both on top of the sample and on sides. With the application of external load via the loading piston, the major principal stress increases as

   $$б_1 = б_1 + P/A'$$

   Where P= the instantaneous machine load on the specimen at some time after the test is started

A'= the corrected area of the specimen

A'= $A_o/(1-\varepsilon)$

The value of P/A'= б is often called the deviator stress. It is evident that load and deformation readings are obtained for this test and that a plot of devaitor stress versus unit strain are plotted. The maximum deviator stress is taken for plotting the Mohr's circle.