UNSATURATED SHEAR STRENGTH CHARACTERSTICS AND STRESS - STRAIN BEHAVIOUR OF SILT SOILS OF HAWASSA

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List of symbols

u_a	Pore air Pressure
u_w	Pore water Pressure
MS	Matric suction
CU	Consolidated undrained
CD	Consolidated drained
CW	Constant water test
UC	Unconfined compression
c'	Effective cohesion
ϕ'	Effective internal angle of friction
ϕ^b	Angle indicating the rate of increase in shear strength relative to matric suction
τ	Shear stress
σ	Normal stress
σ'	Effective normal stress
γ_d	Dry density
LL	Liquid limit
PI	Plasticity index
NP	None plastic
S	Degree of saturation
BS	British Standard Methods of test
ASTM	American Society for Testing and Materials
AASHTO	American Association of State Highway and Transportation Officials
USCS	Unified Soil Classification System
TMI	Thornthwaite Moisture Index
HAE	High-air-entry
ΔVc	Total volume change
SWCC	Soil Water Characteristics Curves

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Abstract

Most of the shear strength tests and studies done in this country particularly in Addis Ababa are using the conventional triaxial machine, which is unable to measure the matric suction of the soil. However, a few research works have recently been done on the shear strength characteristics for unsaturated soils in Addis Ababa. These previous studies on the unsaturated soil are only on red clay and expansive soils of certain areas of Addis Ababa and a single study on Arbaminch expansive soils. This thesis provides the results of triaxial tests which were carried out to study the unsaturated shear strength characteristics of Hawassa silt soil with suction measurement. The soil tested was reconstituted using dynamic compaction. Consolidated undrained (CU) triaxial tests were performed on both saturated and unsaturated samples.

In this work, disturbed silt soil samples were collected from test pits dug to a depth of 2.5m and the shear strength tests were conducted on compacted samples. The saturated shear tests were carried out for consolidation pressures of 100 kPa,200 kPa and 300 kPa.The unsaturated shear tests were carried out for same net confining pressure of 200 kPa but with varying matric suctions of 30 kPa,60 kPa,100 kPa,120 kPa and 150 kPa. All the specimens tested were compacted at the same initial moisture content and dry density corresponding to field conditions.

The results from the consolidated undrained triaxial tests disclosed that as the matric suction increases from 0 to 150 kPa, the maximum deviator stress increases from 202.48 kPa to more than double of 443.14 kPa under the net confining pressure of 200 kPa. This signifies that shear strength of the silt soil sample considerably increases due to matric suction and the rate of increase has been found to be significantly higher compared to what was observed in clays under unsaturated condition. Moreover, the shape of the stress-strain curve of the unsaturated cases resemble that of the saturated samples and for almost the whole range of the applied matric suction, the increase in matric suction did not affect the shape of the stress–strain curve.

1. INTRODUCTION

1.1. General

A large portion of the world population is found in the arid and semi arid regions of the world. In these regions the ground water table is deep because the evaporation from the ground surface exceeds the annual precipitation. Structures are built on or in the earth's surface and hence the safety of any geotechnical works is closely related to the shear strength of the soil. Shear strength forms an important engineering property in the design of numerous geotechnical and geo-environmental structures involving compacted or natural soils such as earth dams, retaining walls, pavements, liners etc. Hence, there are structures that are anticipated to remain in an unsaturated state during their entire life period. As a result, the unsaturated soil condition is relevant in the design of these geotechnical and geo-environmental structures.

Civil Engineers may be concerned with naturally occurring soils in the design of slopes and foundations or may excavate and rework soils during construction of an embankment. In any case, whether the soil is undisturbed or reworked, the civil engineer will be concerned with the engineering behavior of an uncemented or only slightly cemented, aggregate of mineral grains with water contained in the pore spaces [1].

There are numerous materials encountered in engineering practice whose behavior is not consistent with the principles and concepts of classical, saturated soil mechanics. Commonly, it is the presence of more than two phases that results in a material that is difficult to deal with in engineering practice. Soils that are unsaturated form the largest category of materials which do not adhere in behavior to classical, saturated soil mechanics [2].

Several disciplines such as soil science, hydrogeology, petroleum, agricultural, ceramics especially geotechnical and geo-environmental engineering have contributed towards our present understanding of the mechanics of unsaturated soils. Significant advancements were made particularly during the last two decades with respect to the development of the theoretical frameworks, experimental methods and numerical techniques related to geotechnical and geo-environmental engineering applications. During this period, four International Conferences on Unsaturated Soils were also held: Paris (France) in 1995, Beijing (China) in 1998, Recife (Brazil) in 2002, and Phoenix (USA) in 2006. As a result, we have a better understanding of the engineering behavior of unsaturated soils today. Approximately 20% of the publications of recent years in geotechnical and geo-environmental journals are either directly or indirectly related to the research area of unsaturated soils [3].

Shear strength is one of the fundamental properties of unsaturated soils. Many geotechnical problems such as bearing capacity, lateral earth pressures, and slope stability are related to the shear strength of a soil. The shear strength of a soil can be related to the stress state in the soil. Numerous shear strength tests and other related studies on unsaturated soils have been conducted during the past 30 years [2].

In Ethiopia, a few research works have been done related to unsaturated soil which were only conducted on red clay and expansive soils of certain areas in Addis Ababa [4, 5] , and a single study on Arbaminch expansive soils [23].

Hawassa town, the present study area, is the capital town of the Southern Regional Government. It is situated at 275 km south of Addis Ababa and falls in the central part of the Ethiopian Rift Valley. The Hawassa area has a sub-humid climate and receives a mean annual precipitation of about 963 mm mainly concentrated in long rainy season [6]. It is one of the rapidly growing towns in the country. During recent years there is a sudden increasing demand to construct new light to heavy buildings in the town. Due to geology and time the soils of the area are relatively young. These are derived from the late tertiary to quaternary volcanic rocks [7].

1.2. Back ground of the problem

There are vast areas, particularly in the tropical and subtropical regions, where the soils are generally unsaturated. In addition, even in many temperate regions the soil above the water table remains unsaturated.

Apart from natural and geological processes, man-made activities such as excavation, remolding and recompacting may also lead to the desaturation of saturated soils and hence the formation of unsaturated soils.

Many of the more traditional geotechnical engineering problems fall wholly or partly into the category of unsaturated soil mechanics problems. Compaction, for example, a classical application involving unsaturated soil, has been routine practice for improving the mechanical and hydraulic properties of soil since far before the formation of civil engineering as a formal discipline in the mid-nineteenth century. Compacted soil comprising the many earthworks constructed all over the world is most appropriately considered from an unsaturated soils framework [19].

These natural and man-made materials are difficult to be considered and understood, particularly where volume changes are concerned, within the framework of classical saturated soil mechanics. Hence, there is a need to assess the shear strength of unsaturated soils for safe and cost effective design of structures founded on unsaturated soils.

What is more, as mentioned above, in our country only very few researches have been done on this area and hence, there is a need to explore the case on other soil types like silt.

Most soils exist in an unsaturated state with continuous air in their voids. The pore air pressure (u_a) , is usually equivalent to the atmospheric pressure, but the pore water pressure (u_w) will be sub-atmospheric, i.e. negative due to capillary effects in the small pores of soils. This negative pore water pressure or suction produces an additional component of effective stress.

The shear strength of a soil is related to stress state in the soil and unlike saturated soils the stress state variables used for unsaturated soils are the net normal stress $(\sigma - u_a)$, and the matric suction $(u_a - u_w)$ [2]. But the conventional triaxial machine does not allow the measurement of suction $(u_a - u_w)$ which is one of the stress state variables in unsaturated soils. Therefore, to conduct shear strength tests on unsaturated soils, the equipment which allows the independent measurement of the pore pressures has been employed.

1.3. Objectives of the Study

The objective of this research work is to investigate the shear strength characteristics and stress – strain behavior of silt soils for unsaturated case using the modified triaxial testing machine which allows the independent measurement of pore-air and pore water pressures and to compare it with the results obtained for saturated soil samples.

1.4. Scope of the Study

The origin of the soils in the study area can be divided into two broad groups, transported and residual. The area is largely covered by thick (>30 m) soils [8]. As per the examination and investigation done by Lamore, Y.(2006) [8], the soils of the Sandy silt soils occur in almost all parts of the town around lake Hawassa. These soils exist extensively to the northern and north-eastern part of the town and are usually interstratified with silty sand soils. These soils underlay the clay soils to the eastern part and show varied colors including dark yellowish brown and light gray in dry state and olive color when wet [8]. In addition, test results obtained from Construction Design Share Company for Construction of buildings in the town indicates the presence of these sandy silt soils in shallow foundation level (Appendix B).

Four test pits were dug and disturbed soil samples were taken for the testing of the shear strength and index properties. Hence, the laboratory tests conducted on the four pits have been supported by visual identification and secondary data obtained from various sources to come up with realistic representative data for the shear test. The saturated as well as unsaturated tests have been conducted using the modified trixial machine. From the laboratory index test result, the samples from the test pits can be grouped in to two. Three of the samples are categorized as sandy silt with no plasticity while one showed very low plasticity. The shear strength tests have been done for the group of samples of the same index properties of no plasticity. Samples of same dry density and moisture content have been prepared for the shear tests.

While sampling, insitu dry density and moisture content tests were done on site so as to use later the density and moisture content for compacting the sample in the laboratory to be used for the shear strength testing.

This paper studies compacted silt soils in order to obtain experimental data which may help to understand their unsaturated shear strength behavior and it provides a better understanding of the mechanical properties of silt soils and their effect on the design of civil engineering structures. The results from this research can also be used as a resource for further investigation related to the study of unsaturated silt soil properties for different localities in the country.

1.5. Outline of the Thesis

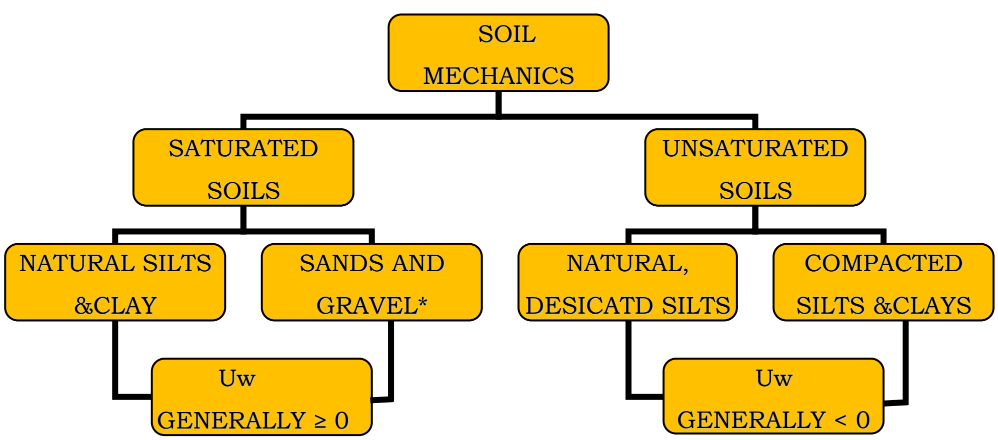
The research report is divided in to five Chapters, each Chapter covers specific topic in the research work. In Chapter one background of the problem, objective and the scope of the work are presented. Chapter two deals with a brief literature review of unsaturated soil mechanics and method of determining shear strength of unsaturated soils. Chapter three deals with materials, methods and laboratory tests and results. Chapter four covers discussion of test results and the last chapter presents conclusion and recommendation.

2. LITERATURE REVIEW

2.1 Introduction to unsaturated soil mechanics

2.1.1 General

The development of soil mechanics for unsaturated soils began about two to three decades after the commencement of study of soil mechanics for saturated soils. The basic principles related to the understanding of unsaturated soil mechanics were formulated mainly in the 1970s [4]. The development of classical soil mechanics has led to an emphasis on particular types of soils. The common soil types are saturated sands, silts and clays, and dry sands. The general field of soil mechanics can be subdivided into that portion dealing with saturated soils and that portion dealing with unsaturated soils (Fig. 2.1). The differentiation between saturated and unsaturated soils becomes necessary due to basic differences in their nature and engineering behavior. An unsaturated soil has more than two phases, and the pore-water pressure is negative relative to the pore-air pressure. Any soil near the ground surface, present in a relatively dry environment, will be subjected to negative pore-water pressures and possible desaturation. Unsaturated soils have recently gained widespread attention in many studies and construction works all over the world, since many soils near the ground surface are considered unsaturated and also those compacted soils comprising the many earthworks constructed all over the world are most appropriately considered from an unsaturated soils framework [2,19].



* may be saturated or dry

Figure 2.1 Categories of soil mechanics [1]

Classical soil mechanics and geotechnical engineering have been often taught with an implicit assumption that soil is either dry or saturated [2]. Soil behavior, it is argued, is governed solely by Terzaghi's effective stress principle [10]. In fact, dry and saturated states are just two extreme and limiting conditions of a soil. In other words, dry and saturated conditions are just two special cases of an unsaturated (i.e. not-saturated) soil which has a degree of saturation that lies between 0 and 100 percent.

In many engineering problems, however, a soil is often neither saturated nor dry. Relatively, limited research has been conducted on unsaturated soils [11]. Clearly, there is an urgent need to improve the understanding of the behavior and mechanics of an unsaturated soil. As stated above, for convenience, the general field of classical soil mechanics is often subdivided into that portion dealing with saturated soils and that portion dealing with unsaturated soils. Although this artificial division between saturated and unsaturated soils can be shown to be unnecessary, it may still be helpful to make use of the knowledge gained from saturated soils as a reference and then to extend it to the broader unsaturated world as shown in Figure 2.2, which provides a visual aid for the generalized world of soil mechanics [12]. For simplicity, this world of soil mechanics is divided by the water table. Below the water table, soil behavior is governed by effective stress ($\sigma - u_w$), whereas the unsaturated soil above the water table is governed by two independent stress variables, net normal stress(σ-u_a) and matric suction (u_a - u_w) [13,14].

At low matric suctions, where the suction is lower than the air-entry value of the soil, the soil is at or near saturation condition and behaves as though it is saturated. Consequently an increase in matric suction produces the same increase in shear strength as does an increase in net normal stress. However, at matric suctions higher than the air-entry value of the soil, the soil starts to desaturate. The increase in shear strength with respect to matric suction becomes less than the increase with respect to the net normal stress. In other words, the ϕ^b angle is generally equal to $\phi^{'}$ at low matric suction and decreases to a lower value at high matric suction [15].

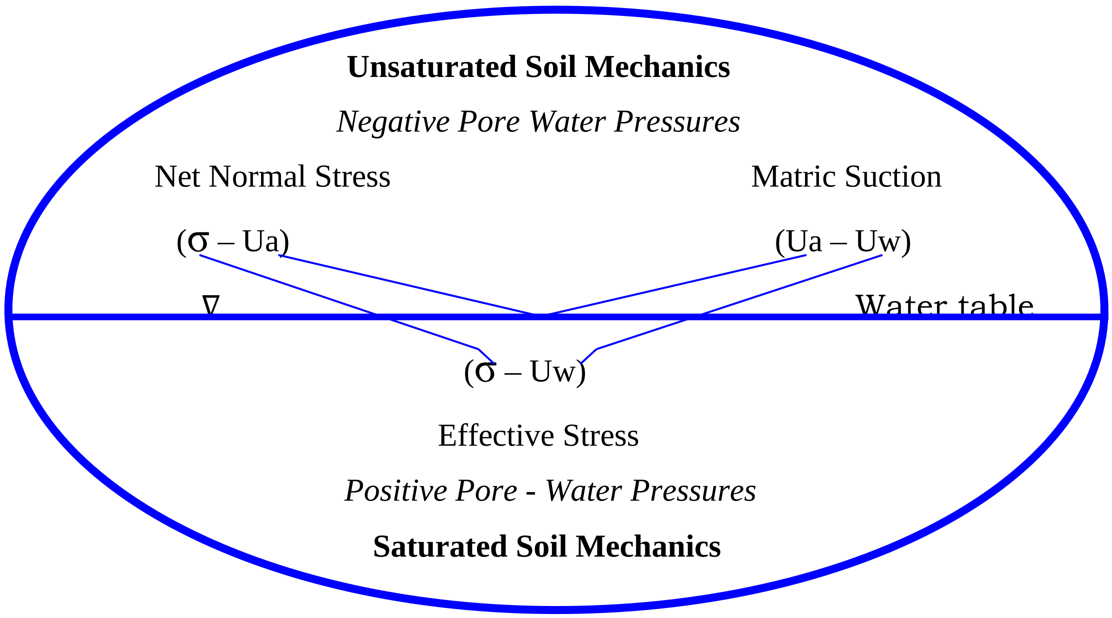


Figure 2.2 A visualization aid for the generalized world of soil mechanics [11].

2.1.2 Role of Climate

Climate plays an important role in whether a soil is saturated or unsaturated. Water is removed from the soil either by evaporation from the ground surface or by evapotranspiration from a vegetative cover. These processes produce an upward flux of water out of the soil. On the other hand, rainfall and other forms of precipitation provide a downward flux into the soil. The difference between these two flux conditions on a local scale largely dictates the pore-water pressure conditions in the soil. A net upward flux produces a gradual drying, cracking, and desiccation of the soil mass, whereas a net downward flux eventually saturates a soil mass. The depth of the water table is influenced, amongst other things, by the net surface flux. A hydrostatic line relative to the groundwater table represents an equilibrium condition where there is no flux at ground surface. During dry periods, the pore-water pressures become more negative than those represented by the hydrostatic line. The opposite condition occurs during wet periods [11].

Significant areas of the earth's surface are classified as arid zones. The annual evaporation from the ground surface in these regions exceeds the annual precipitation. Figure 2.3 shows the climatic classification of the extremely arid, arid, and semi-arid areas of the world. Meigs [11] used the TMI to map these zones. He excluded the cold deserts. Regions with a TMI less than -40 indicate areas about 33% of the earth's surface is considered arid and semi-arid [17]. Arid and semi-arid areas usually have a deep groundwater table. Soils located above the water table are desaturated due to the excessive evaporation and evapotranspiration and have negative pore-water pressures.

Climate has a further possible effect on the properties of tropical residual soils – that of unsaturation. Even in sub humid tropical areas, water tables are often deeper than 5 to 10m and the effects of unsaturation, desiccation and seasonal or longer term re-wetting have to be taken into account in geotechnical design [18].

Many soils exhibit extreme swelling or expansion when wetted. Other soils are known for their significant loss of shear strength upon wetting. Changes in the negative pore-water pressures associated with heavy rainfalls are the cause of numerous slope failures. Reductions in the bearing capacity and resilient modulus of soils are also associated with increases in the pore-water pressures. These phenomena indicate the important role that negative pore-water pressures play in controlling the mechanical behavior of unsaturated soils [2].

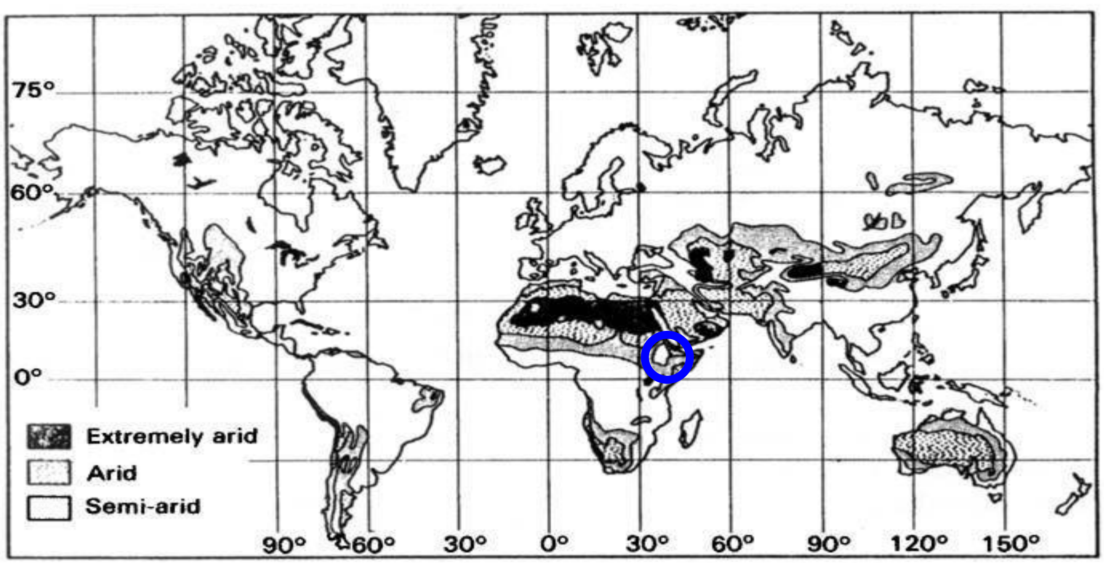


Figure 2.3 Extremely arid, arid, and semi-arid areas of the world (16, 17)

2.1.3 Typical Profiles of Unsaturated Soils

The microclimatic conditions in an area are the main factors causing a soil deposit to be unsaturated. Therefore, unsaturated soils or soils with negative pore water pressures can occur in essentially any geological deposit, such as residual soil, a lacustrine deposit and soils in arid and semi arid areas with deep ground water table [2, 15]. The unsaturated soil zone plays a critical role in biological, physical, and chemical weathering processes that have occurred throughout the history of earth. The history of soil formation is the history of the unsaturated zone [19]. As a result of physical and chemical weathering processes largely controlled by environmental factors at the ground surface, parent rock weathers to a residual soil profile of distinct horizons and chemical composition [19]. An unsaturated soil could be a residual soil, a lacustrine deposit, a bed rock formation, and so on. However, there are certain geological categories of soils with negative pore-water pressures that have received considerable attention in the research literature. Tropical residual soils have some unique characteristics related to their composition and the environment under which they develop [2].The in situ water content of residual soils is generally greater than its optimum water content for compaction. Their density, plasticity index, and compressibility are likely to be less than corresponding values for temperate zone soils with comparable liquid limits. Their strength and permeability are likely to be greater than those of temperate zone soils with comparable liquid limits [20].

Residual soils have been of particular concern in recent years. Once again, the primary factor contributing to their unusual behavior is their negative pore-water pressures. Attempts have been made to use saturated soil mechanics design procedures on these soils with limited success [2]. Most classical concepts related to soil properties and soil behavior have been developed for temperate zone soils and there has been difficulty in accurately modeling procedures and conditions to which residual soils will be subjected. Engineers appear to be slowly recognizing that residual soils are generally soils with negative in situ pore-water pressures and that much of the unusual behavior exhibited during laboratory testing is related to a matric suction change in the soil [21]. There is the need for reliable engineering design associated with residual soils. When the degree of saturation of a soil is greater than about 85%, saturated soil mechanics principles can be applied. However, when the degree of saturation is less than 85%, it becomes necessary to apply unsaturated soil mechanics principles [15].

2.1.4 Unsaturated Soil in practice

Unsaturated soils were either ignored in civil engineering design and construction analyses or were approached inappropriately from the traditional framework of saturated soil mechanics**.** The first fifty years of soil mechanics history have been primarily concerned with soils saturated with water and most soil mechanics principles developed in that period apply to saturated soils only. This shortcoming is actually one of the main driving forces for the emerging subject of unsaturated soil mechanics. Another important driving force is due to the distinct volume, strength and flow characteristics of certain soils when they become unsaturated with water [22]. Rapid advancement in our understanding of unsaturated soil behavior over the last 30 to 40 years,however,has led to today's civil engineer to realize that there is now an opportunity to approach problems involving unsaturated soils on a much more basis. The expanding knowledge base on the fundamental principles of unsaturated soil mechanics is increasingly being incorporated into a diverse array of practical engineering problems [19].As mentioned earlier, in Ethiopia limited academic researches on unsaturated soil began since 2009 and there has not been any practice with respect to unsaturated soil mechanics till now.

2.1.5 Phases of Unsaturated Soils

Unsaturated soil is a multiphase system comprised of three phases of matter: gas, liquid, and solid. The gas phase is generally bounded by the pore space not occupied by liquid. The matter within this pore space may be any gas, vapor, or combination thereof. The liquid phase is generally bounded by the pore space not occupied by gas. The matter within this pore space may be any liquid or miscible or immiscible combination of two or more liquids (water, oil, non–aqueous phase liquids, etc.). The solid phase consists of the soil grains or particles and may range from relatively fine-grained materials such as silts and clays, to organic material, to relatively coarse-grained materials such as sand or gravel [19].

An unsaturated soil is commonly referred to as a three-phase system [2]. These phases are: air, water and solid.

However, recent research results have realized the important role of the air–water interface (i.e. the contractile skin) which should be warranted as an additional phase when considering certain physical mechanisms. This is because when the air phase is continuous, the contractile skin interacts with the soil particles and provides an influence of the mechanical behavior of soil. The mass and volume of each phase can be schematically represented by a phase diagram as shown in Figure 2.4.

The thickness of the contractile skin is in the order of only a few molecular layers. Therefore the physical subdivision of the contractile skin is considered as part of the water phase without any significant error. A simplified three phase diagram is used when referring to the summation of masses and volumes of all soil particles.

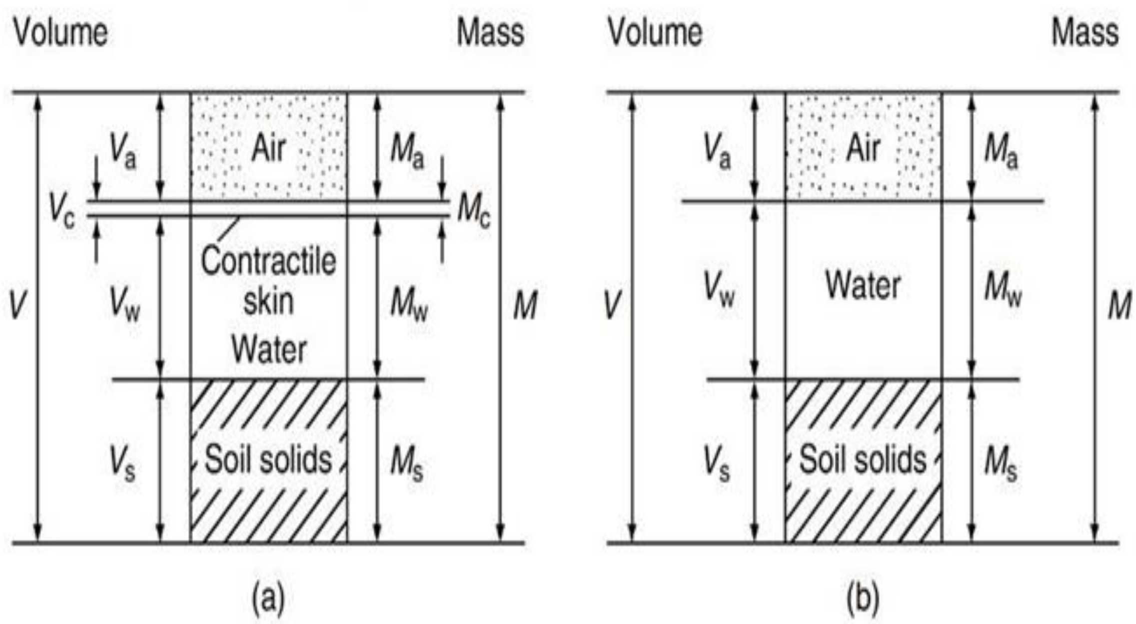


Figure 2.4 Rigorous and simplified phase diagrams for an unsaturated soil. (a) Rigorous four phase unsaturated soil system ;(b) simplified three phase diagram.

2.2 Stress State Variables in Soils

2.2.1 General

The effective stress concept has been well accepted and studied for saturated soils. Numerous attempts have been made to develop a similar concept of effective stress for unsaturated soils. However, unsaturated soils are more complex, and it has been more difficult to arrive at a consensus regarding the description of the stress state. The use of a single-valued effective stress for unsaturated soils has encountered many difficulties, and has led numerous researchers to the realization that two independent stress state variables should be used for unsaturated soils. The mechanical behavior of a soil (i.e., the volume change, seepage and shear strength behavior) can be described in terms of the state of stress in the soil. The state of stress in a soil consists of certain combinations of stress variables that can be referred to as stress state variables [2].

2.2.2 Stress State Variables for an Unsaturated Soil

An understanding of the meaning of effective stress proves to be valuable when considering the stress state description for unsaturated soils.

During the past two decades there has been an increasing use of two independent stress variables to describe the behavior of unsaturated soils [24, 25]. Most consideration has been given to describing the volume change behavior (and suitable constitutive relations) for unsaturated soils. Several investigations have been made into the shear strength characterization of unsaturated soils [26, 27]; however, none has proven completely successful. From a stress field analysis any two of three possible stress variables can be used to define the stress state in an unsaturated soil [13]. Possible combinations are: (1) $(\sigma - u_a)$ and $(u_a - u_w)$, (2) $(\sigma - u_w)$ and $(u_a - u_w)$ and (3) $(\sigma - u_a)$ and $(\sigma - u_w)$ where (σ = total normal stress, u_a = pore air pressure, and u_w = pore-water pressure. The shear strength, using two possible combinations of stress state variables, is presented and the relationship between the two cases is shown. The first stress state variables used are $(\sigma - u_w)$ and $(u_a - u_w)$. The advantage of this combination of variables is that it provides a readily visualized transition from the unsaturated to the saturated case. The disadvantage arises in that, when the pore-water pressure is changed, two stress state variables are being affected. The relative significance of each variable must be born in mind when considering the shear strength. The second combination of stress state variables used is $(\sigma - u_a)$ and $(u_a - u_w)$. The advantage of this combination is that only one stress variable is affected when the pore-water pressure is changed. Regardless of the combination of stress variables used to define the shear strength, the value of shear strength obtained for a particular soil with certain values of σ, u_a, and u_w must be the same.

It is worth considering the merits of selecting $(\sigma - u_a)$ and $(u_a - u_w)$ as stress state variables, rather than $(\sigma - u_w)$ and $(u_a - u_w)$. The former combination has the advantage that the pore air pressure u_a is zero in many practical situations, so that net stress and matric suction simplify to total stress and negative pore water pressure respectively. In addition, the pore water pressure, which is commonly negative, is often very difficult to measure. This leads to uncertainty in the value of only one stress state variable if $(\sigma - u_a)$ and $(u_a - u_w)$ are selected, but uncertainty in the values of both stress state variables if $(\sigma - u_w)$ and $(u_a - u_w)$ are chosen [11]. Thus the $(\sigma - u_a)$ and $(u_a - u_w)$ are chosen to be the most satisfactory combination from a practical analysis standpoint [28]. The importance of soil suction in describing the mechanical behavior of an unsaturated soil was recognized by many researchers around the world. This fact led to the inclusion of the negative pore-water pressure in a single-valued effective stress equation for unsaturated soils. However, later on three independent normal stresses state variables are extracted from the equilibrium equations for the soil structure. These are $(\sigma - u_a)$, $(u_a - u_w)$ and (u_a) that governs the equilibrium of the soil structure and the contractile skin. However, unlike saturated soils, the mechanical behavior of unsaturated soils depends on two independent stress-state variables. These variables are the stress tensor $(\sigma - u_a)$, which is referred to as net normal stress, and the difference between the pore-air pressure (u_a), and pore-water pressure (u_w), which is referred to as matric suction $(u_a - u_w)$ .These combinations appear to be the most appropriate for use in engineering practice [2]. The stress variables, (u_a) can be eliminated because of insignificant contribution. Therefore, the $(\sigma - u_a)$ and $(u_a - u_w)$ are referred to us the stress state variables for an unsaturated soil.

The complete form of the stress state for an unsaturated soil in terms of two independent stress tensors can be represented in a matrix form as shown below:

$$\begin{bmatrix} \sigma_x - u_a & \tau_{xy} & \tau_{xz} \\ \tau_{yx} & \sigma_y - u_a & \tau_{yz} \\ \tau_{zx} & \tau_{zy} & \sigma_z - u_a \end{bmatrix}$$

And

$$\begin{bmatrix} u_a - u_w & 0 & 0 \\ 0 & u_a - u_w & 0 \\ 0 & 0 & u_a - u_w \end{bmatrix}$$

The soil matric suction ($u_a - u_w$) is a quantity that can be used to characterize the effect of moisture on volume. It is a measure of the energy or stress that holds the soil water in the pores or a measure of the pulling stress exerted on the pore water by the soil mass.

2.2.3 Stress State Variables for Saturated Soil

Saturated soil can be viewed as a special case of an unsaturated soil as the degree of saturation *S*, approaches 100%.Then, the four phases in an unsaturated soil reduce to two phases for a saturated soil (i.e., soil particles and water). The stress tensor for a saturated soil indicates that the difference between the total stress and the pore-water pressure forms a stress state variable. This stress state variable, ($\sigma - u_w$), is commonly referred to as effective stress [10]. Effective stress law is essential stress state variable which is required to describe the mechanical behavior of a saturated soil.

$$\sigma' = \sigma - \upsilon_w \tag{2.1}$$

Where: σ’ = effective normal stress

σ = total normal stress

u_w = pore water pressure

The volume change process and the shear strength characteristics of a saturated soil are both controlled by the effective stress.

The complete form of the stress state for saturated soil in terms of stress tensors can be represented in a matrix form as shown below:

$$\begin{bmatrix} \sigma_x - u_w & \tau_{yx} & \tau_{zx} \\ \tau_{xy} & \sigma_y - u_w & \tau_{zy} \\ \tau_{xz} & \tau_{yz} & \sigma_z - u_w \end{bmatrix}$$

2.3 Shear strength theory

2.3.1 General

Unsaturated shear strength is of relevance in many geotechnical applications involving either compacted or natural soils (road and railway embankments, flood defenses, earth dams, retaining structures, landslides) [29].

The shear strength of a soil can be related to the stress state in the soil. The stress state variable used for saturated soils is the effective normal stress ($\sigma - u_w$). For unsaturated soils, the effective normal stress ($\sigma - u_a$) and the matric suction ($u_a - u_w$) are used as stress state variables.

The shear strength of saturated soil is described using the Mohr-coulomb failure criterion and the effective stress concept [10].

$$\tau_{ff} = c' + \left(\sigma_f - \upsilon_w\right)_f \tan\phi' \tag{2.2}$$

Where τ_{ff} = shear stress on the failure plane at failure

c' = effective cohesion, which is the shear strength intercept when the effective normal stress is equal to zero.

$\left(\sigma_f - u_w\right)_f$ = effective normal stress on the failure plane at failure

σ_{ff} = total normal stress on the failure plane at failure

u_w = pore-water pressure at failure

ϕ' = effective angle of internal friction

The above equation defines a line as shown in Fig.2.5. The line is commonly referred to as a failure envelope. The envelope represents possible combinations of shear stress and effective normal stress on the failure plane at failure.

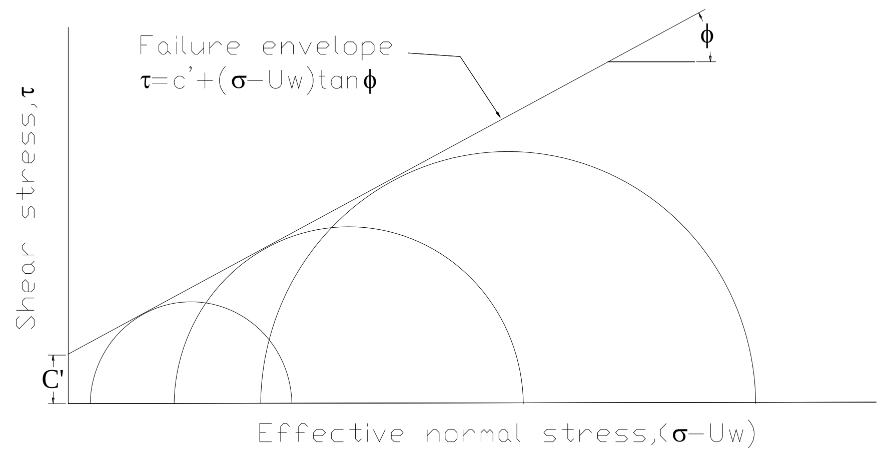


Figure 2.5 Mohr-Coulomb failure envelope for a saturated soil

The failure envelope for a saturated soil is obtained by plotting a series of Mohr circles corresponding to failure conditions on two dimensional plot, as shown in Fig 2.5. The line tangent to the Mohr circles is known as the failure envelope as described by equation 2.5

2.3.2 Failure Envelope for Unsaturated Soils

In case of unsaturated soil, the Mohr circles corresponding to failure conditions are plotted in three dimensional manner as illustrated in Fig.2.6. The three dimensional plot has the shear stress, τ as the ordinate and the two stress state variables, $(\sigma - u_a)$ and $(u_a - u_w)$ as abscissas. The frontal plane represents a saturated soil when the matric suction becomes zero.

The Mohr circles for unsaturated soils are plotted with respect to the net normal stress axis, $(\sigma - u_a)$ in the same manner as the Mohr circles plotted for saturated soils with respect to the effective stress axis, $(\sigma - u_w)$.

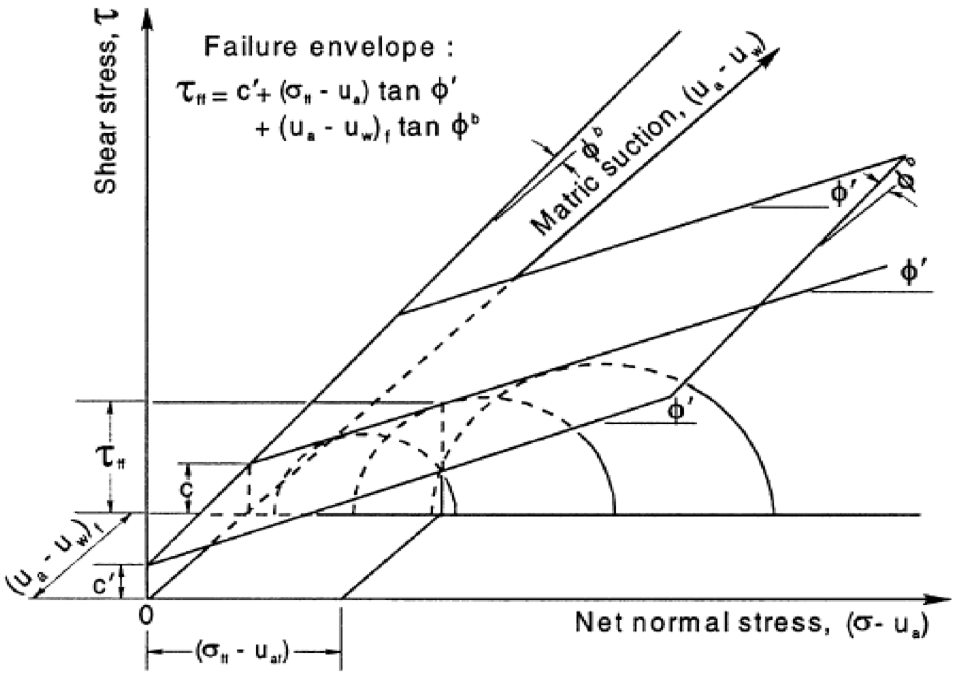


Figure 2.6 Extended Mohr –coulomb failure envelope for unsaturated soils

However, the location of the Mohr circles plotted in the third dimension is a function of the matric suction,(Fig.2.6). The surface tangent to the Mohr circles at failure is referred to as the extended Mohr-coulomb failure envelope for unsaturated soils. The extended Mohr-coulomb failure envelope defines the shear strength for unsaturated soils [2]. The shear strength equation for unsaturated soils can be expressed as:

$$\tau_{ff} = c' + (\upsilon_a - \upsilon_w)_f \tan\phi^b + (\sigma_f - \upsilon_a)_f \tan\phi' \qquad (2.3)$$

Where τ_{ff} =shear stress on the failure plane at failure

c' = intercept of the “extended “Mohr –Coulomb failure envelope on the shear stress axis where the net normal stress and the matric suction at failure are equal to zero

$(\sigma_f - u_a)_f$ = net normal stress state on the failure plane at failure

ϕ' = angle of internal friction associated with the net normal stress state variable, $(\sigma_f - u_a)_f$

$(u_a - u_w)_f$ = matric suction on the failure plane at failure

ϕ^b = angle indicating the rate of increase in shear strength relative to the matric suction

2.3.3 Tests on shear strength of unsaturated soils

There are two generic types of testing methods commonly used for shear strength testing of soils in the laboratory, namely the direct shear test and the triaxial test [18] Among the triaxial tests, the unconfined compression test is a special case of triaxial compression test which is only used to determine undrained shear strength of cohesive soils.

2.3.4 Triaxial tests on unsaturated soils

The triaxial testing equipment has a considerable versatility and permits a large variety of test procedures to determine triaxial strength, stiffness and characteristic stress ratios of a soil specimen. In addition the test can be used to measure consolidation and permeability characteristics [18]. The test is usually performed on a cylindrical soil specimen enclosed in a rubber soil membrane placed in the triaxial cell. The cell is filled with water and pressurized in order to apply a constant all round pressure or confining pressure. The soil specimen can then be subjected to an axial stress through the loading ram in contact with the top of the specimen. There are various triaxial test procedures which are used for testing unsaturated soils based on the drainage conditions adhered to during the first and second stages of the triaxial test.

The application of the confining pressure is considered as the first stage in a triaxial test. The soil specimen can either be allowed to drain (i.e., consolidate) during the application of the confining pressure or drainage can be prevented. The term consolidation is used to describe the process whereby excess pore pressures due to the applied stress are allowed to dissipate, resulting in volume change.

The consolidation process occurs subsequent to the application of the confining pressure if the pore fluids are allowed to drain. On the other hand, the consolidation process will not occur if the pore fluids are maintained in an undrained condition. The consolidated and unconsolidated conditions are used as the first criterion in categorizing triaxial tests.

The application of the axial stress is considered as the second stage or the shearing stage in the triaxial test. In a conventional triaxial test, the soil specimen is sheared by applying a compressive stress. The total confining pressure generally remains constant during shear. The axial stress is continuously increased until a failure condition is reached. The axial stress generally acts as the total major principal stress, σ_1, in the axial direction, while the isotropic confining pressure acts as the total minor principal stress, σ_2 , in the lateral direction. The total intermediate principal stress, σ_2, is equal to the total minor principal stress, σ_3 (i.e.,$\sigma_2= \sigma_3$) [2].

Various triaxial test procedures are used for unsaturated soils based upon the drainage conditions adhered to during the first and second stages of the triaxial test. The triaxial test methods are usually given a two-word designation or abbreviated to a two-letter symbol. The designations are: 1) consolidated drained or CD test, 2) constant water content or CW test, 3) consolidated undrained or CU test with pore pressure measurements, 4) undrained test, and 5) unconfined compression or UC test. In the case of CD and CU tests, the first letter refers to the drainage condition prior to shear, while the second letter refers to the drainage condition during shear. The constant water content test is a special case where only the pore-air is kept in a drained mode, while the pore-water phase is kept undrained during shear (i.e., constant water content). The pore-air and pore water are not allowed to drain throughout the test for the undrained triaxial test. The unconfined compression test is a special loading condition of the undrained triaxial test.

Saturated soils of clays, very fine sand, silts and silty sands are likely to fail in the field under condition similar to those under which consolidated undrained (CU) tests are made due to their poor drainage qualities [33]. As the silt soils in the study area are anticipated to exhibit low permeability characteristics, the consolidated undrained (CU) test procedure was employed in this study.

Hence, a series of CU tests has been carried out and the test procedure is presented below in detail. Interested reader is referred to Fredlund and Rahardjo (1993) [2] for direct shear and other triaxial testing procedures.

2.3.5 Consolidated undrained test

The consolidated undrained or CU test uses a condition where the soil specimen is consolidated first and then sheared, with pore-air and pore-water under undrained conditions. The consolidation process brings the soil specimen to the desired stress state [i.e., $(\sigma_3 - u_a)$ and $(u_a - u_w)$]. The axis-translation technique is then used to establish matric suctions greater than 101.3 kPa as explained in section 3.5.2.After the consolidation stage the soil specimen is sheared by increasing the deviator stress $(\sigma_1 - \sigma_3)$, until failure is reached. The drainage valves for both the pore-air and the pore-water pressures remain closed (under undrained conditions) during shear. Excess pore air and pore water pressures are developed during undrained loading. The pore-air and pore-water pressures should be measured during the shearing process. The net confining pressure, $(\sigma_3 - u_a)$ and the matric suction, $(u_a - u_w)$, are altered throughout the test due to the changing pore- air and pore-water pressures. At failure, the magnitude of the net major and minor principal stresses and the matric suction are a function of the pore – pressures.

A typical stress path for a consolidated undrained test is illustrated in Fig 2.7 The stress state at the end of consolidation is represented by point *A* where the net confining pressure is $(\sigma_3 - u_a)$ and the matric suction is $(u_a - u_w)$. Shear causes stress state to move from point *A* to point *B*, along stress path $\overline{AB}$. The stress state at failure is represented by stress point *B*, corresponding to a different net confining pressure and matric suction from those associated with stress point *A*. As indicated in the drawing, the pore-air pressure is assumed to increase continuously during shear. This causes the net confining pressure to decrease [i.e. $(\sigma_3 - u_a)_f < (\sigma_3 - u_a)_{initial}$]. The matric suction is also assumed to decrease continuously [i.e $(u_a - u_w)_f < (u_a - u_w)_{initial}$]. The failure envelope is tangent to Mohr circle at failure (e.g at stress point c) and inclined at an angle of ϕ' with respect to the $(\sigma - u_a)$ axis. The failure envelope intersects the shear strength verses $(u_a - u_w)$ plane at a cohesion intercept, *c*. The intersection line joining the cohesion intercepts produced by tests at different matric suctions gives the angle, ϕ^b [2].

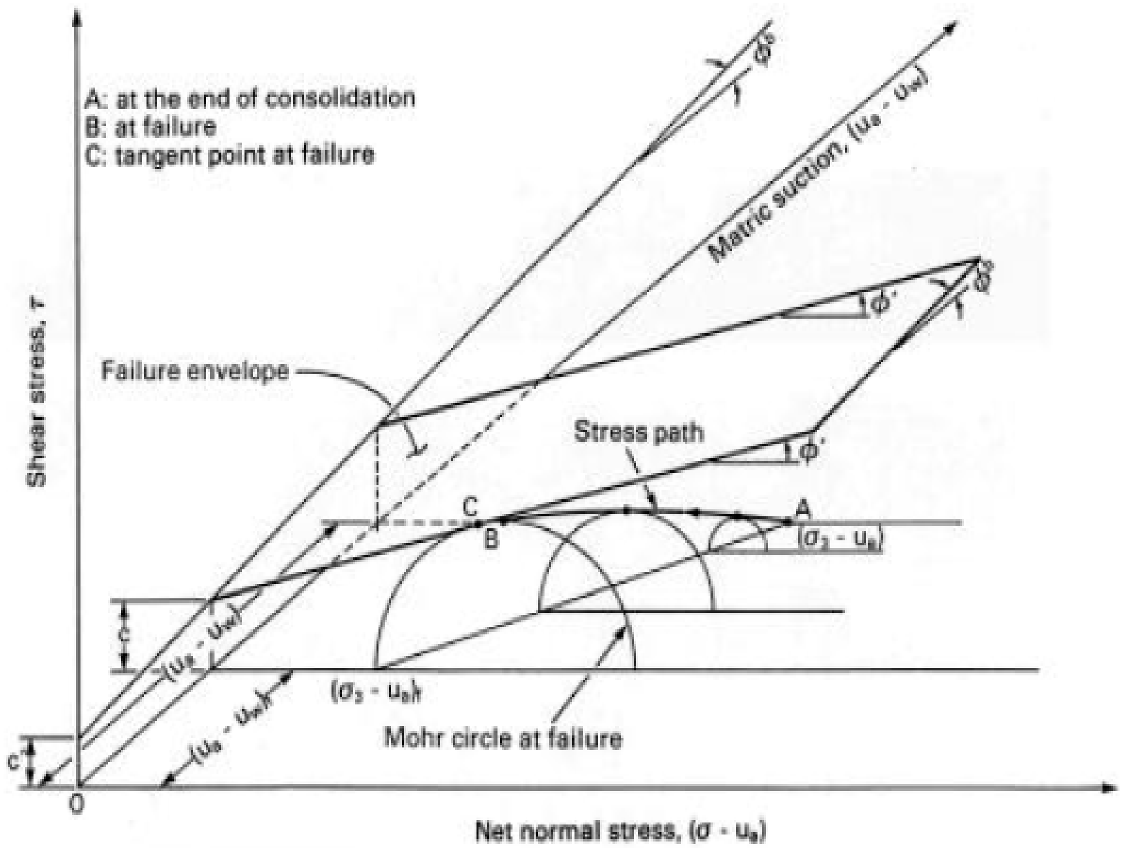


Figure 2.7 Typical stress path followed during consolidated undrained test

2.3.6 Previously Conducted Unsaturated Shear Strength Tests in Ethiopia

Three previously conducted Masters theses works have attempted to study the unsaturated shear strength behavior of clayey soils found in Addis Ababa and Arbanminch towns. One study was on red clay soil of Addis Ababa [4] and the other two on expansive soils found in Addis Ababa [5] and Arabminch [23]. It was reported that the deviatoric stress of Addis Ababa red clay increases form 315.20 kPa to the 429.80 kPa for a matric suction increase off 70 kPa under a confining pressure of 250 kPa [4]; while for Addis CMC area expansive soil the deviatoric stress increases from 54.00 kPa to 93.40 kPa for a matric suction increase of 75 kPa with a confining pressure of 150 kPa and for Arbaminch expansive soil it increases from 107.6 kPa to 174.35 kPa for a matric suction of 200 kPa and a confining pressure of 200 kPa. The findings of the researchers are presented in tables below.

Table 2.1 Failure deviator stress for saturated and unsaturated case of Addis Ababa red clay soil [4].

Test pit	Sample state before shear commencement (kPa)	Net confining pressure(kPa)	Deviator Failure stress(kPa)
Kolfe 1	Saturated soil sample	250	315.2
	Matric suction 30	250	340.5
	Matric suction50	250	392.2
	Matric suction 70	250	429.8
Kolfe 2	Saturated soil sample	150	188.0
	Matric suction 30	150	247.8
	Matric suction50	150	296.8
	Matric suction 70	150	339.9

Table 2.2 Failure deviator stress for saturated and unsaturated case of Addis Ababa expansive soil [5].

Test pit	Sample state before shear commencement (kPa)	Net confining pressure (kPa)	Deviator Failure stress(kPa)
CMC area	Saturated soil sample	150	54.0
	Matric suction 25		62.8
	Matric suction 50		76.6
	Matric suction 75		93.4
Bole area	Saturated soil sample	250	90.7
	Matric suction 25		106.2
	Matric suction 50		115.4
	Matric suction 75		130.1

Table 2.2 Failure deviator stress for saturated and unsaturated case of Arbaminch expansive soil [23].

Test pit	Sample state before shear commencement (kPa)	Net confining pressure(kPa)	Deviator Failure stress(kPa)
AMU-HSC site	Saturated soil sample	200	107.6
	Matric suction 100 kPa		138.2
	Matric suction 150 kPa		157.2
	Matric suction 200 kPa		174.4

3. LABORATORY TEST RESULTS

3.1 General

Soils are usually heterogeneous and complex materials. The complexity is contributed by their existence in almost innumerable varieties by its combination of solid, liquid and gases, where in many instances the solid particles vary in size from big boulders to colloidal size. To understand the behavior of the soils, methods of evaluations have been centered on their physical property as the physical characteristics are of importance to the engineers. The physical properties of soils, which serve mainly for identification and classification, are commonly known as index properties.

Four test pits were dug to a depth of 2.5m as most of the foundation constructions are founded around this depth as shown in Figure 3.1. From the test pits four disturbed soil samples were taken for the testing of the shear strength and index properties.

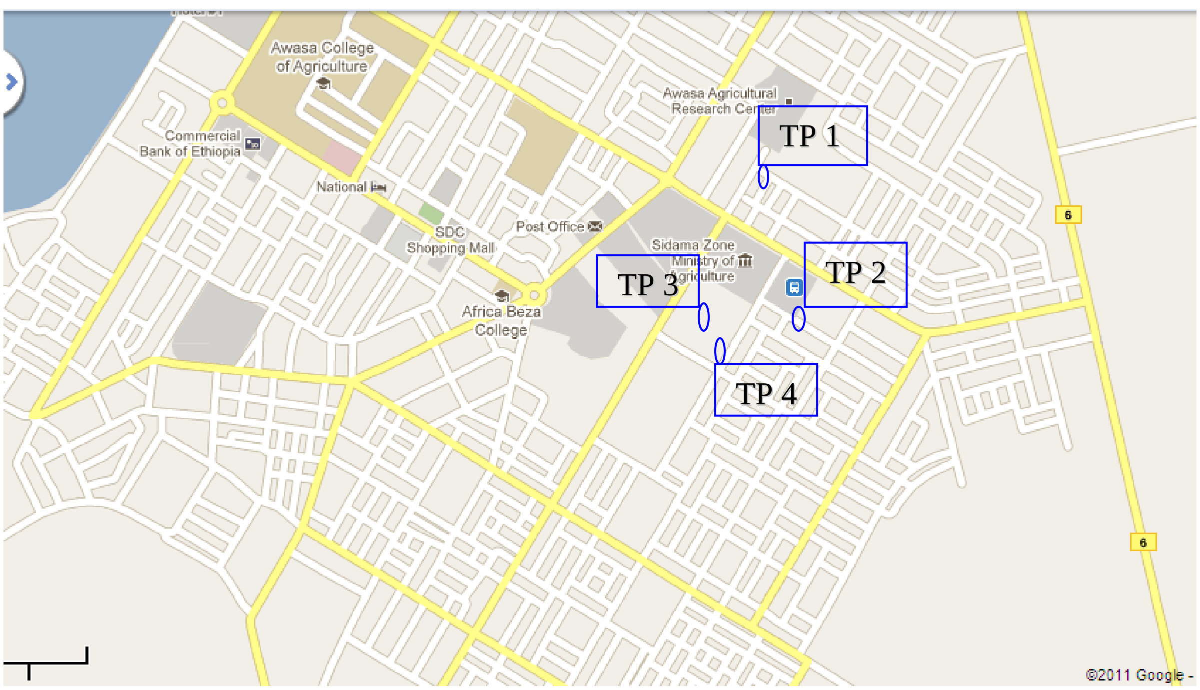


Figure 3.1 Location of the study area

3.2 Index properties

3.2.1 Grain size analysis

The sizes of soil particles and their distribution throughout the soil mass are important factors which influence soil properties and performance.

Coarse grain fraction of each soil sample was analyzed by utilizing sieve analysis where as for fine fraction; hydrometer analysis was conducted as per ASTM Designation D422-63 [30], and the results are presented on Appendix A-1 to A-4.

Based on the gradation test results, the soils considered in this study were found to be 55.5 to 60.7% silt, 37.10 to 41.2% sand and 2.1 to 3.2% clay by proportion.

3.2.2. Atterberg Limit

3.2.2.1 Liquid limit (LL)

The liquid limit test has been carried out as per the procedures outlined on BS-1377 -2:1990 [31] and the obtained results are shown in Table 3.1 below.

3.2.2.2 Plastic limit (PL)

For the determination of the PL value, the test is conducted as per the procedure outlined on BS-1377 -2:1990 [31] and the obtained results are shown below. All soils considered in this study were found to be Non-plastic except one sample from test pit-2 with very low plasticity (PI = 6.02).

Details of the test results for the Atterberg limit tests are shown in Appendix A-5 to A-8.

3.2.3 Free swell

The test is performed by placing 10 ml dry soil specimen, passing through 425micron sieve, into 100ml graduated cylinder and filled with water. The swelled volume was noted after the soil left to settle for about 24hrs. he results are indicated in Table 3.1.

3.2.4 Specific gravity

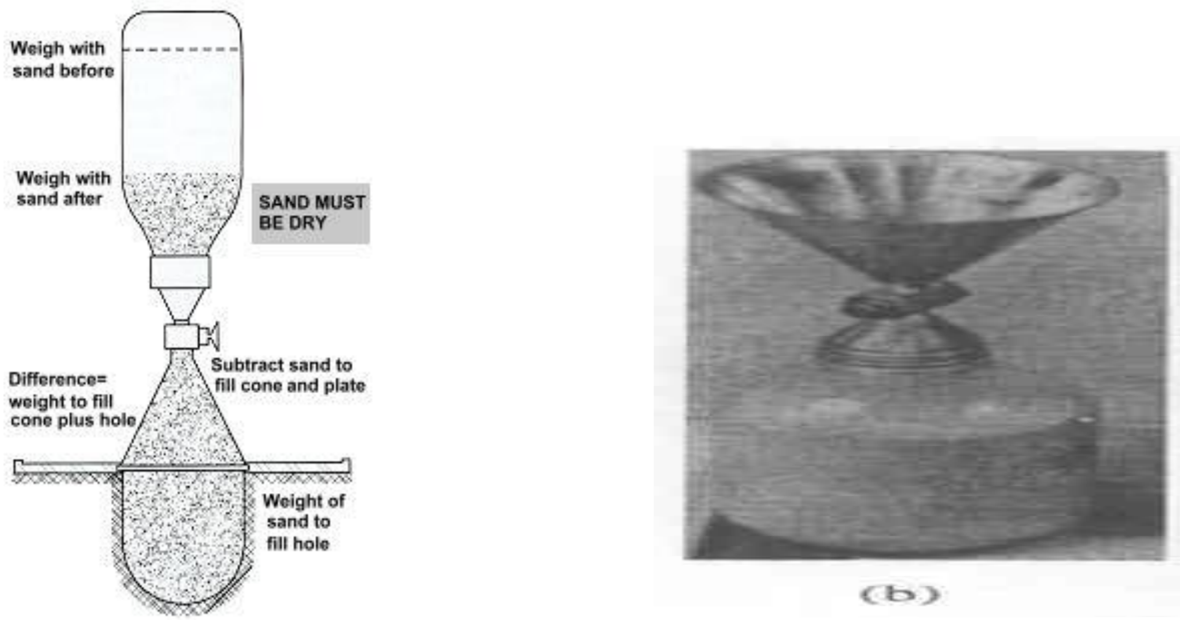
The specific gravities of the soils under investigation were determined using the ASTM procedure, designation D 854-83 [30] and the results are tabulated in Table 3.1.

Table 3.1 Atterberg Limits, Free swell and Specific Gravity results of the study area

Location	Depth (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Free Swell (%)	Specific Gravity
Pit -1	2.5	0	0	N P	0	2.49
Pit -2	2.5	38	33	6	0	2.51
Pit -3	2.5	0	0	N P	0	2.48
Pit -4	2.5	0	0	N P	0	2.48

3.3 In situ Dry Density and Moisture content test

In order to get the in situ dry density and moisture content of the soil sampled from the pits in the study area, the Sand replacement method as shown below is used (ASTM Designation D-1556) [30]. The sand for the sand cone method consists of a sand pouring jar shown in Figure. 3.1. The jar contains uniformly graded clean and dry sand, and a small hole (6" x 6" deep) is dug in the excavated pit to be tested. Sand is run into the hole from the jar by opening the valve above the cone until the hole and the cone below the valve is completely filled. The valve is closed. The jar is calibrated to give the weight of the sand that just fills the hole, that is, the difference in weight of the jar before and after filling the hole after allowing for the weight of sand contained in the cone is the weight of sand poured into the hole.



(a) (b)

Figure 3.2 Sand-cone apparatus: (a) Schematic diagram, and (b) Photograph

The specific volume of the hole is determined by filling it with the calibrated dry sand. The weight of soil removed from the hole is determined and its water content is also determined. Then the dry weight of the soil removed obtained and is divided by the volume of sand needed to fill the hole. This gives us the density of the in situ soil in gram per cubic centimeter.

These tests have been done at bed of the four pits dug, and the results are indicated in Table 3.2.
Table 3.2. Insitu dry density and moisture content at the time of sampling.

Pit No.	Depth of Sampling (m)	Natural moisture content at the time of sampling (%)	In situ dry density, γ_d (gm/cc)
1	2.5	19.89	1.33
2	2.5	21.60	1.37
3	2.5	21.80	1.31
4	2.5	18.70	1.34

3.4. Soil Classification

Soil classification is an important aspect to know the characteristic of the soil under interest. There are different methods of classification based on the identification tests performed on the soil. Unified Soil Classification System (USCS) and the American Association of State Highway Transport Officials (AASHTO) method are among the widely used schemes of soil classification. The USCS is used for general geotechnical purpose, while AASHTO is specifically used in road sector. Hence, the USCS is used to classify the soils considered in this study.

3.4.1. USCS Classification Method

The Unified Soil Classification System is based on the recognition of the type and predominance of the constituents considering grain-size, gradation, plasticity and compressibility. It divides soil in to three major divisions: coarse-grained soils, fine gained soils and highly organic (peaty) soils. In the field, identification is accomplished by visual examination for the coarse-grained soils and a few simple hand tests for the fine-grained soils. In the laboratory, the grain-size curve and the Atterberg limits can be used [33].

The USCS uses symbols for particular size groups. These symbols and their representations are: G–gravel, S–Sand, M–Silt, C–Clay. These are combined with other symbols expressing gradation characteristics–W for well-graded and P for poorly graded and plasticity characteristics – H for high and L for low, and a symbol O for the presence of Organic material. The flowchart shown in Fig. 3.2A provides systematic means of classifying an inorganic fine-grained soil according to the USCS [23, 35].

Experimental results of soils tested from different parts of the world were plotted on a graph of plasticity index (ordinate) versus liquid limit (abscissa). It was found that clays, silts, and organic soils lie in distinct regions of the graph called the plasticity chart, Figure 3.2 B.

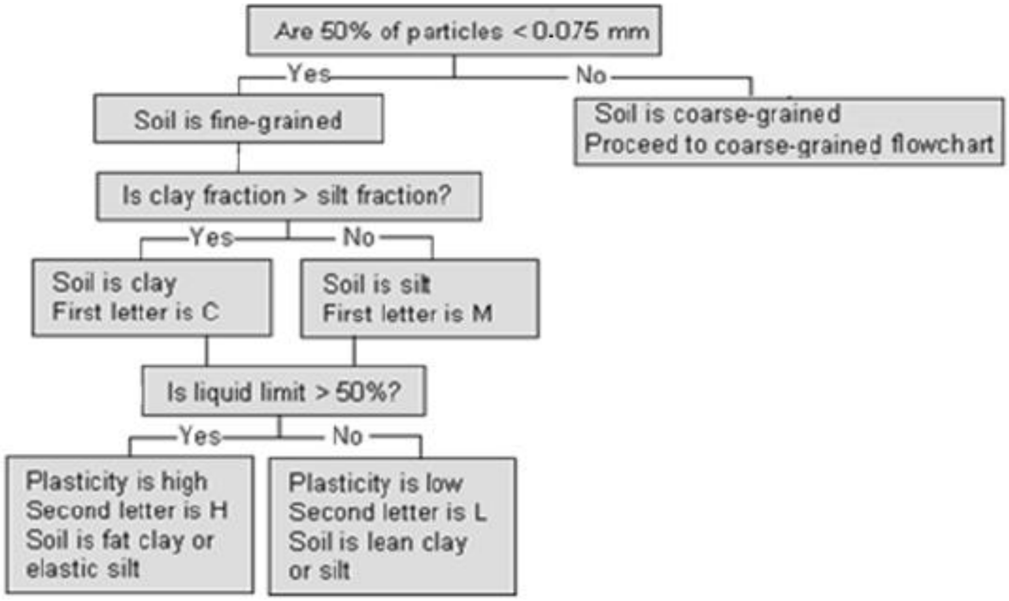


Figure 3.3A Unified soil classification flowcharts for inorganic fine-grained soils [23, 35]

The A-line separates clays from silts and the U-line indicates the upper limit of the relationship between PI and LL. Accordingly, the soil under study is plotted on the plasticity chart.

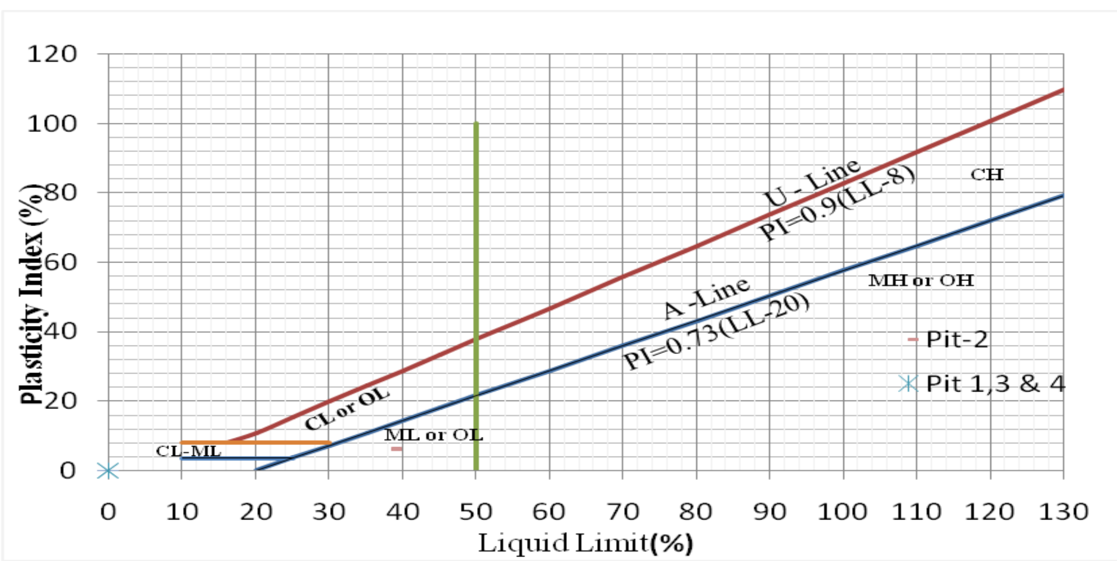


Figure 3.3B Plasticity chart of the area according to USCS

According to this classification scheme, the soil under study is more likely to be silt soil of no or very low plasticity (ML) with group name sandy silt, as the silt fraction is higher than the clay fraction; having more that 50% of particles with a grain size less than 0.075mm.

However, as the property of the soil under study is more dependent on particle size distribution, there is a need to appraise other classification system based on grain size.

3.4.2 Textural Classification (Modified Triangular Diagram)

The triangular classification system suggested by U.S Bureau of Public Roads is commonly known as the textural classification system. Later, the Mississipi River Commission (USA) proposed a modified triangular diagram (Fig 3.2C) so as to eliminate terms which are not used in soil engineering. The term texture is used to express the percentage of the three constituents of soils, namely, sand, silt and clay [36].

According to MIT textural classification system, the percentage of sand (size 0.06 to 2.0 mm), silt (size 0.002 to 0.06 mm) and clay (size less than 0.002mm) are plotted along the three sides of an equilateral triangle. If the soil contains a certain percentage of soil particles larger than 2.0 mm, a correction is applied in which the sum of the percentages of sand, silt and clay is increased to 100 %.

Such a classification is more suitable for describing coarse-grained soils rather than clay soils, whose properties are more dependent on particle size distribution. Hence, according to this classification system, the soil under study lies in sandy silt part of the triangular chart (Fig 3.3C).

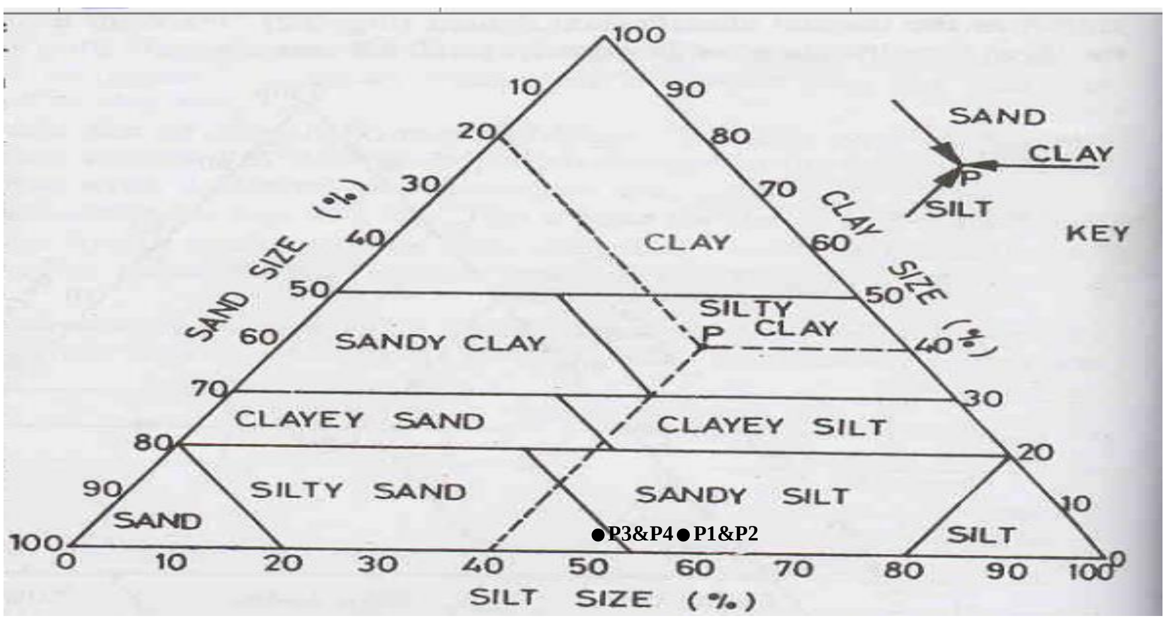


Figure 3.3C Modified Triangular Diagram[36].

3.5. Shear strength tests

3.5.1. The modified triaxial Machine

The shear testing device most commonly used both for design and research is the trixial apparatus [1]. The conventional trixial apparatus has been used for testing saturated soils which has been described in detail by Bishop and Henkel [34] in their standard text on triaxial testing of soils. The conventional triaxial equipment requires modifications when testing the shear strength of unsaturated soils. These modifications must accommodate independent measurement (or control) of the pore-air pressure and the pore water pressure. The axis-translation technique forms the basis for these modifications.

The modified triaxial equipment, 50kN modified double wall triaxial machine (Fig. 3.3) was employed for conducting the tests both for saturated and unsaturated cases. This triaxial machine has got specially designed high air entry disk which allows the passage of water only when it is fully saturated. The axis-translation technique translates the highly negative pore-water pressure to a pressure that can be measured without cavitations in the measuring system. In addition, a high air entry disk with an air entry value greater than the matric suction being measured must be used in order to prevent the passage of pore-air into the measuring system.

The general term axis-translation refers to the practice of elevating pore air pressure in unsaturated soil while maintaining the pore water pressure at a measurable reference value, typically atmospheric. As such, the matric suction variable (u_a - u_w) may be controlled over a range far greater than the cavitation limit for water under negative pressure. The origin of reference, or ‘‘axis,’’ for the matric suction variable is ‘‘translated’’ from the condition of atmospheric air pressure and negative water pressure to the condition of atmospheric water pressure and positive air pressure.

Matric suction may be accurately controlled in this manner because positive air pressure may be easily controlled and measured. Axis translation is accomplished by separating the air and water phases of the soil through the minute pores of a high-air-entry (HAE) material. When saturated, these materials have the unique capability of restricting the advection of air while allowing free advection of water. If a specimen of soil is placed in good contact with a saturated HAE material, positive air pressure may be applied to the pore air on one side, while allowing the pore water to drain freely through the material under atmospheric pressure maintained on the other side. Separation of the air and water pressure is maintained as long as the applied pressure does not exceed the air-entry pressure of the HAE [19].

We can measure the pore air pressure and pore water pressures independently with the help of pore-air and pore-water measuring transducers. The equipment has got volume measuring units for the measurement of the amount of water that comes in and out of the specimen, loading ram which allows application of deviator stress for shear loading, and air-water cylinder where the air pressure comes from the compressor converted to water pressure. Each of the measuring units and transducers are connected to data logger, which is connected to the computer where the software for shear strength test has been installed, having 16 channels calibrated for the specific purpose which they are required to measure.

Figure 3.4 Modified triaxial machine

3.5.2 Testing procedures

Various triaxial test procedures are used for unsaturated soils based upon the drainage conditions adhered to during the consolidation and shearing stages of the triaxial test. Hence the drainage conditions dictate the test method to be used in the laboratory which attempts to simulate the loading conditions that are likely to occur in the field.

As mentioned earlier, the consolidated undrained (CU) test procedure was employed in this study.

3.5.2.1 Sample preparation

Soil specimens which are “identical” in their initial conditions are required for the determination of the shear strength parameters in the laboratory. If the strength parameters of an undisturbed soil are to be measured, the tests should be performed on specimens with the same geological and stress history. On the other hand, if strength parameters for a compacted soil are being measured, the specimens should be compacted at the same initial water content and with the same compactive effort [2].

Disturbed soil samples were collected from the field from a test pit dug to a depth of 2.5m. Using the dry density and moisture content obtained from insitu test, compacted samples were prepared in a mold with known volume to prepare the specimen of dimension 70mm x 140mm for the shear test. To take out the required dimension of soil sample from the mold, the sample extractor having dimension of 70mmX140mm has been pushed in to the mold which the soil is compacted to the required density. After the sampling tube (70mm x 140mm) is pushed into the mold, the soil with the sampler tube has been taken out and finally the soil sample with the required dimension is extruded from the sampler tube with another sample extractor. (Appendix B).

3.5.2.2 Mounting of sample in to the triaxial machine

Before mounting the sample on the equipment the following procedures have been followed:

1. To remove the entrapped air in the system, flushing was done by opening the volume change units and allowing de-aired water in to the system.
2. The high air-entry disk must be fully saturated before mounting the sample in the tixial cell.

3. After properly trimming the specimen it was mounted on the pre-saturated high entry disk.
4. The specimen is then enclosed with rubber membrane and O-rings are placed over the membrane on the bottom pedestal.
5. Then the coarse porous disk and the loading cap are placed on the top of the specimen.
6. After putting O-rings on top of the sample above the rubber membrane, the cell chamber was placed on its proper place and filled with de-aired water.

The sample during preparation (compaction), extracting and mounting is as indicated on Appendix B.

The figure below shows the assemblage of a triaxial cell used for testing the shear strength of unsaturated soil.

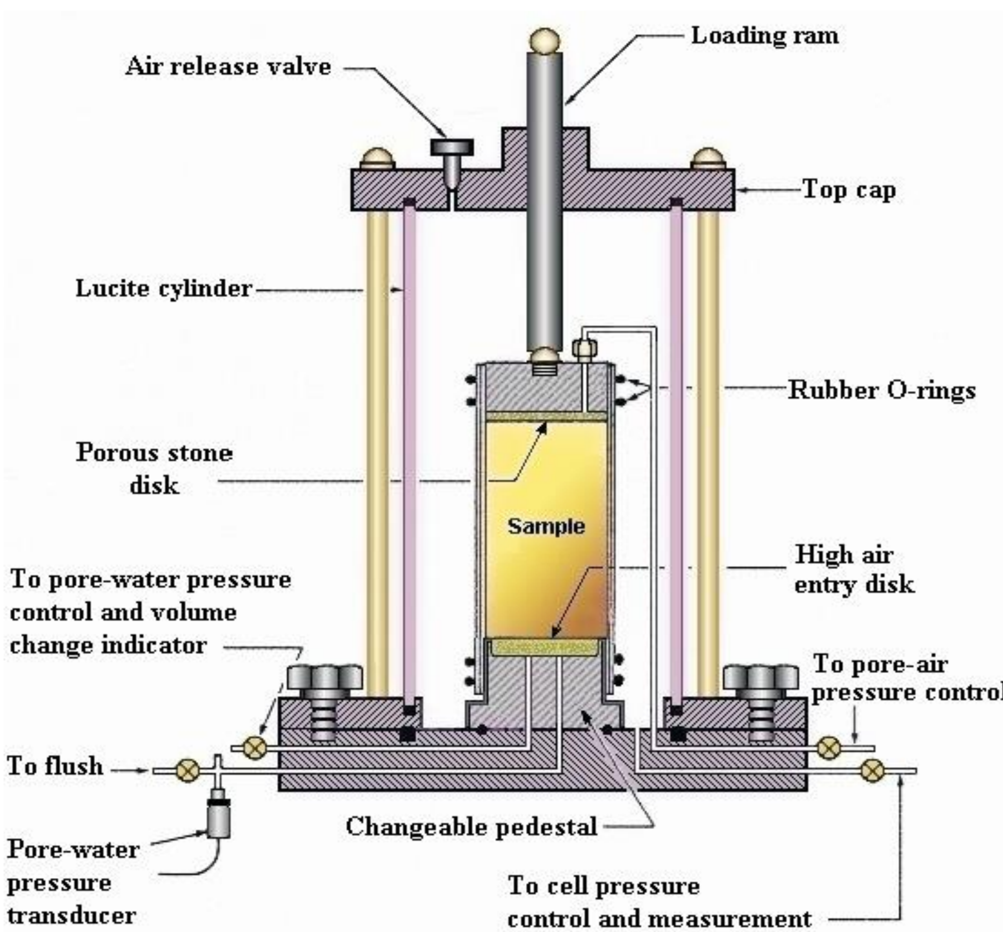


Figure 3.5 Section of Modified triaxial equipment for testing of unsaturated soils.

3.5.2.3 Testing

To carry out the test the following test procedures have been followed:

i. **Saturation Stage**

The objective of the saturation stage is to ensure that all the voids are filled with water without producing undesirable prestressing of the specimen or allowing the soil to swell.

Initially, unsaturated specimens, either undisturbed or compacted, must be saturated prior to testing [2]. Saturation is commonly achieved by incrementally increasing the pore-water pressure. At the same time, the confining pressure is increased incrementally in order to maintain a constant effective stress in the specimen. As a result, the pore air pressure increases and the pore-air volume decreases by compression and dissolution into the pore-water. The simultaneous pore-water and confining pressure increases are referred to as a “back pressuring the soil specimen.”

The saturation of the soil specimen in series of tests was accomplished by using the backpressure technique [31].

Back Pressure Technique

The technique of applying a constant back pressure to the pore water is commonly used in soil testing. The purpose of back pressure is to saturate the sample by dissolving any air or gas present in the pore water. Provided that the results are expressed in terms of effective stress, the magnitude of the back pressure will have no influence on the test [2].

The predetermined chamber pressure and backpressure were applied at several stages keeping the difference between the two values to 10kPa until saturation is achieved. The cell pressure should always be larger than the back pressure, so that the effective stress is positive. Saturation of the sample is ensured when the pore water pressure increment divided by the back pressure increment parameter *B* which is given by a relation:

$$B = \frac{\delta u}{\delta \sigma}$$

(Where δu-change in pore water and $\delta\sigma$- change in stress) is equal to or greater than 0.95 [31].

The pressures used for saturation are shown in the table below.

Table 3.3 Pressures used for saturation

Saturation Method	Back Pressure Increments	Cell Increments	50,100,150,200,250,300 kPa
		Back Increments	40,90,140,190,240 kPa

ii. Consolidation Stage

The consolidation stage follows immediately after the saturation stage. Consolidation of the specimen for these tests is isotropic. The objective of the consolidation stage is to bring the specimen to the state of effective stress and matric suction required for carrying out the compression test.

Consolidation Procedure :

After completion of the saturation stage, the back pressure valve remains closed and the final pore pressure and volume-change readings are recorded. For unsaturated soils due to the application of the pore air in to the sample, the procedure is somehow different from the procedure to be followed for saturated soils. The consolidation procedure shall be as follows:

1. The cell pressure (σ_3) would be provided through the cell pressure line by adjusting the value which will give the required effective consolidation pressure and the back pressure is adjusted if necessary.
2. The pore air pressure would be adjusted to the value which gives the required level of matric suction by opening the pore air line..
3. When a steady value of pore water pressure is reached, the values were recorded. The readings of the volume-change indicator were recorded at a convenient moment (zero time).
4. The consolidation stage would be started by opening the back pressure valve and pore-air line and record the readings of the volume-change indicator at suitable intervals of time until there is no further significant volume change.

5. When consolidation is complete, the reading of the volume-change indicator is recorded and the total volume change (ΔVc) during the consolidation stage calculated. Then the compression stage followed.

iii. **Shearing Stage**

After the consolidation stage the samples were axially loaded in compression keeping the cell pressure constant. All the tests were performed under a strain controlled condition.

Shearing Procedure

1. After the consolidation stage, the drainage valves would be closed so that the test will be undrained.
2. The loading piston would be lowered to just touch the top of loading cap.
3. A constant shearing strain rate of 0.073mm/min was applied obtained from the calculation based on the procedure illustrated in BS-1377 -2:1990 [31].
4. The sample is then sheared by pressing the switches of motor, logger and computer with closing the drainage valve.
5. The changes in displacement, pore- water pressure, pore- air pressure, deviator stress are measured automatically.
6. Then the test has been stopped when one of the following criteria has been clearly identified [31].

   a) Maximum deviator stress;

   b) Maximum effective principal stress ratio;

   c) Constant shear stress and constant pore pressure

The different failure criteria produce similar shear strength parameters [2].

3.6 Results obtained from consolidated undrained test

3.6.1 Classification of the testing program

The laboratory testing program was divided into two main groups where all tests are conducted on the same modified triaxial machine. The first group deals with samples that have 100 % degree of saturation (i.e., zero matric suction). The main goal of testing this group was to evaluate the effective shear strength parameters, c' and ϕ', and to have the ability to compare the behavior of saturated samples with that of unsaturated samples. The second group deals with unsaturated samples. Five samples at five different matric suctions, (30, 60,100,120 and 150 kPa), were tested under the effect of the same prescribed effective confining pressure of 200 kPa.

The values of the initial matric suctions and the confining pressures are shown in the table below.

Table 3.4 Test parameters used for saturated and unsaturated case

Sample No.	**Confining Stress, σ_3 (kPa)**	**Matric Suction (kPa)**
1	100	0
2	200	0
3	300	0
4	200	30
5	200	60
6	200	100
7	200	120
8	200	150

3.6.2 Test Results

From the result of the conducted tests, the maximum deviator stress at failure and the stress – strain diagrams are presented below

Table 3.5 Maximum deviator stress obtained at failure

Sample No.	Soil Sample	Confining Stress, σ_3 (kPa)	Deviator Failure Stress(kPa)
1	Saturated Soil sample	100	111.60
2	Saturated Soil sample	200	202.48
3	Saturated Soil sample	300	294.15
4	Matric Suction, 30kpa	200	245.59
5	Matric Suction, 60kpa	200	290.00
6	Matric Suction, 100kpa	200	349.60
7	Matric Suction, 120kpa	200	399.36
8	Matric Suction, 150kpa	200	443.14

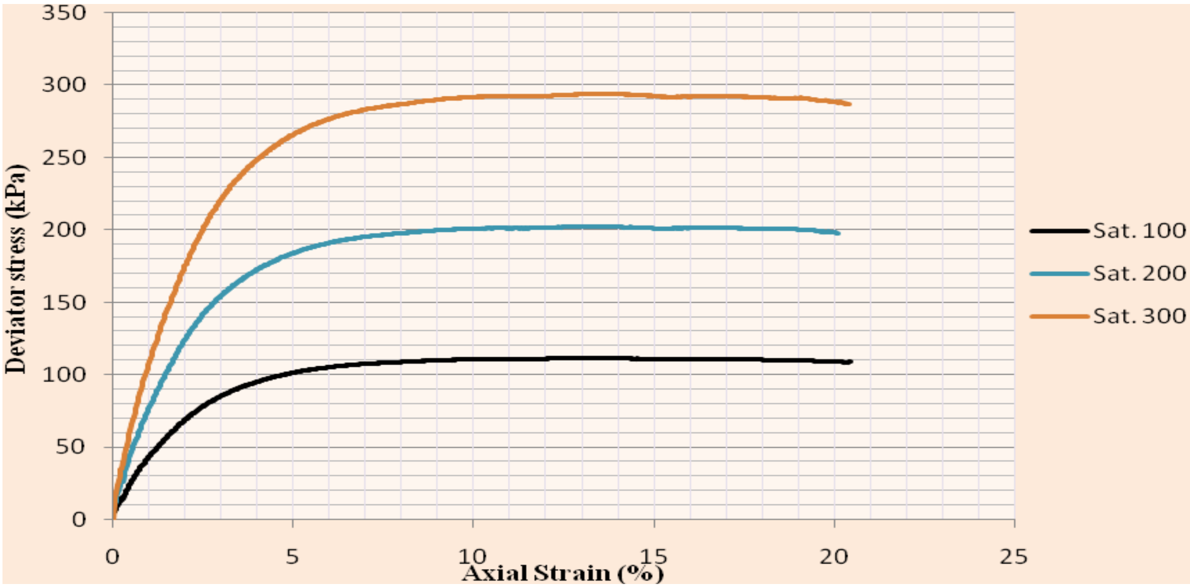


Figure 3.6 Deviator stress Vs Axial Strain for consolidation stress of 100,200 and 300 kPa with zero matric suction.

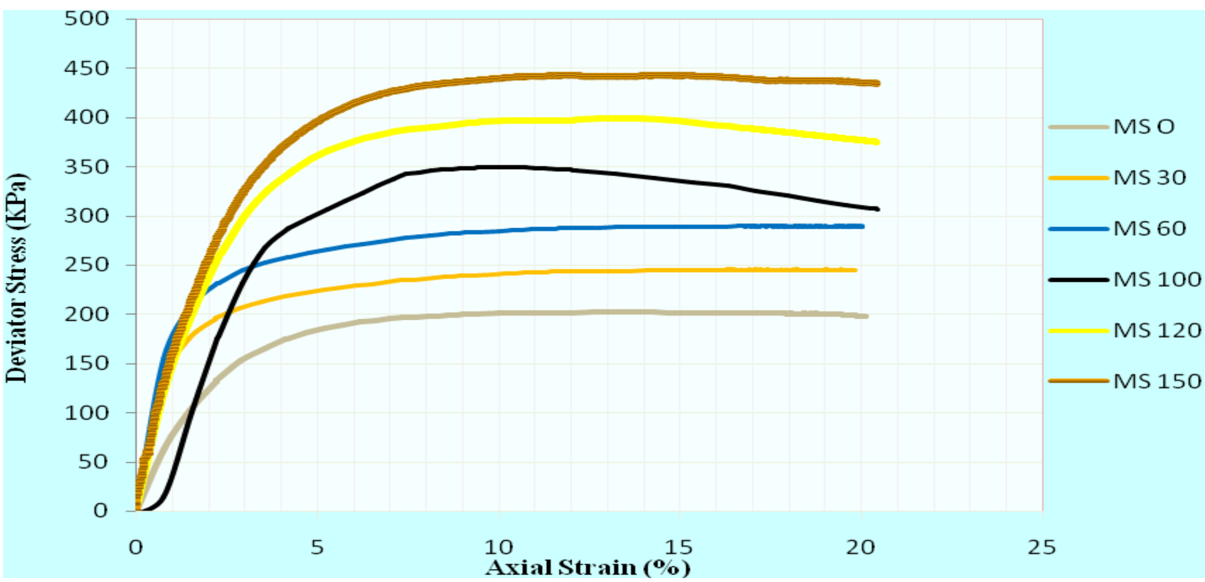


Figure 3.7 Deviator stress Vs Axial Strain for consolidation stress of 200 kPa with various matric suctions.

3.6.3 Analysis of the ϕ^b parameter

Mohr circle for the net normal stress has been drawn for maximum deviator stress obtained from triaxial test carried out on saturated soil sample. Then, a best tangent line for the three circles has been drawn to obtain the c′ and ϕ', values of the soil (Figure 3.7). The data for ϕ^b is interpreted by plotting the maximum deviator stress conditions (for the various matric suctions) on a conventional Mohr–Coulomb type of plot and then projecting the failure stress circles on the shear stress versus net normal stress plane. Lines, parallel to the best tangent line, are drawn tangent to the failure stress circles to give a cohesion intercept for each suction level (Figure 3.8a). The cohesion values are then plotted versus the matric suction to give the angle ϕ^b (Figure 3.8b).

The parameters, which are used to draw Figures 3.7 and 3.8, are tabulated in Table A-9 and Table A-10 in appendix A and the results obtained from the Figures are summarized in Table 3.6.below.

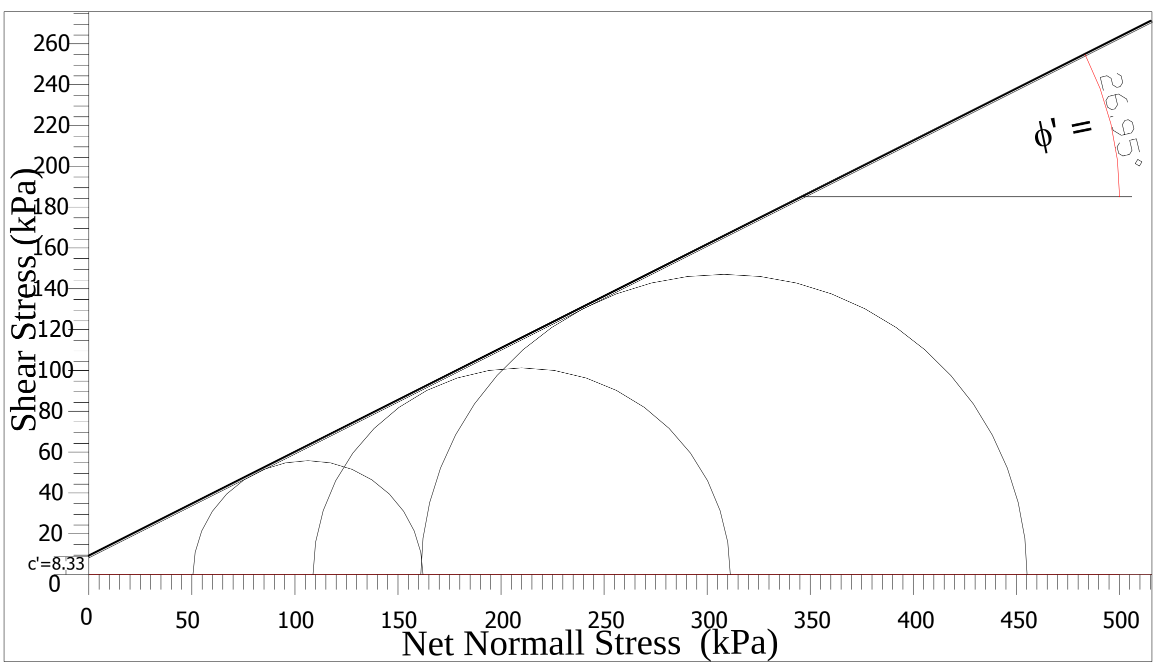
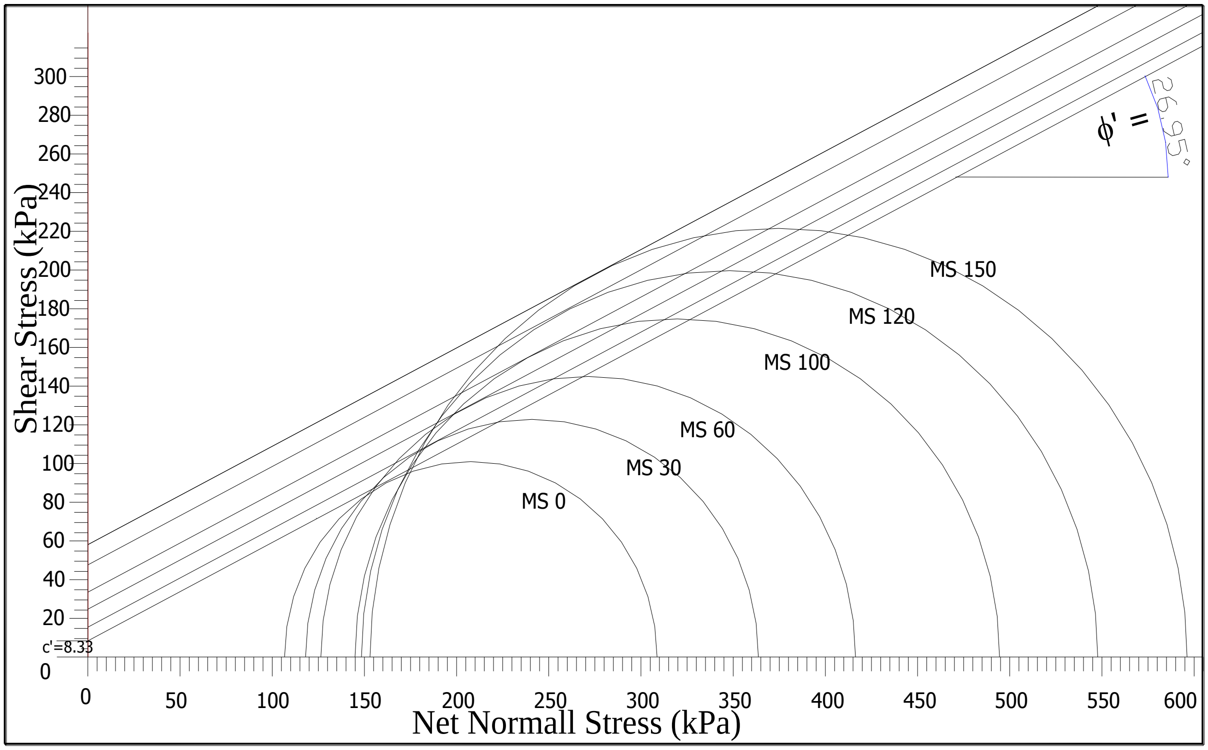
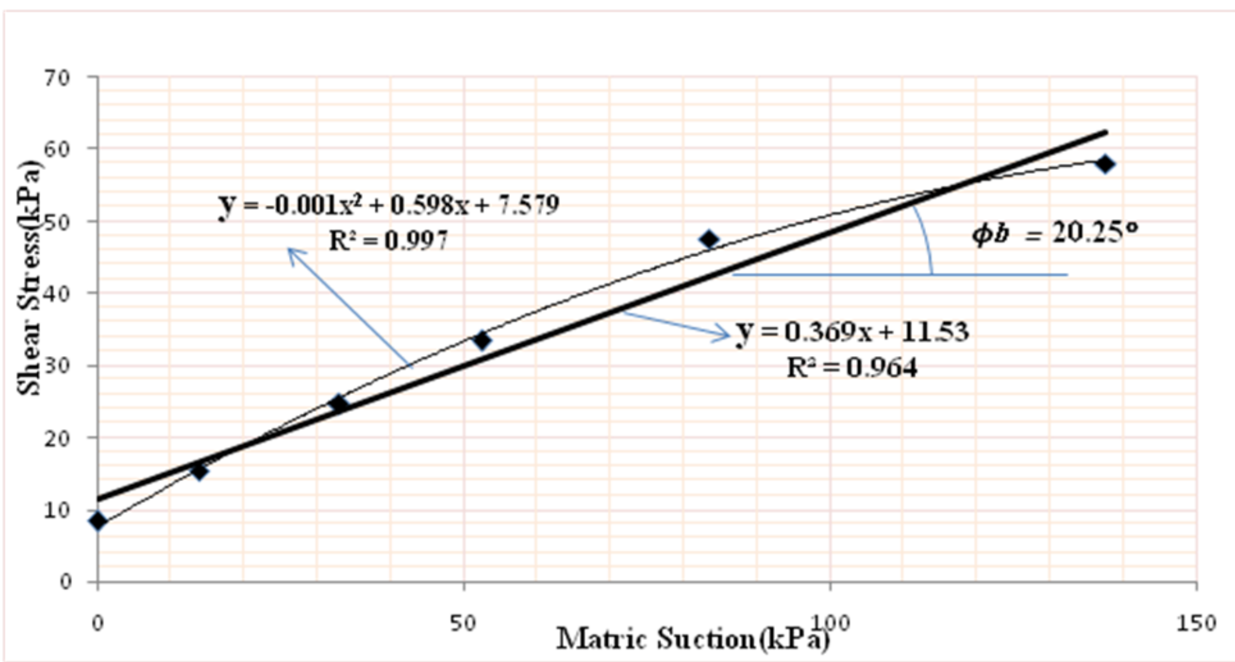


Figure 3.8 Mohr circle for saturated soil under the effect of ($\sigma_{3\,(1)}$ = 100, $\sigma_{3\,(2)}$ =200 and $\sigma_{3\,(3)}$ =300 kPa) effective consolidation pressure.



(a)



(b)

Figure 3.9: Two-dimensional presentation of failure envelope (a) failure envelope projected on the shear stress versus net normal stress plane, (b) intersection between the failure envelope and the shear stress versus matric suction plane (represented by line and curve).

Table 3.6 Summary of shear strength parameters.

Effective Consolidation Pressure(Kpa)	*c*' **(kPa)**	*c* **(kPa)**	**Matric Suction,** MS**(kPa)**	ϕ' (deg)	ϕ^b (deg)
200	8.33	15.39	13.90	26.95	26.93
200		24.67	33.09		25.81
200		33.50	52.40		24.57
200		47.61	83.56		24.31
200		58.71	137.45		11.64

4. DISCUSSION OF THE TEST RESULTS

As one can see from Figure 3.5 and Table 3.5, the strength of saturated samples increases as the net confining pressure increases. On the other hand, at the same net confining pressure, the strength of an unsaturated sample is greater than that of a saturated one. For example, in this study the deviator stress for unsaturated sample (with matric suction of 30 kPa) at net confining pressure of 200 kPa is roughly 1.21 times that of saturated sample at same net confining pressure.

Moreover; as shown in Figure 3.6 and Table 3.5, as the matric suction increases from 30 kPa to 150 kPa, the deviator stress increases from 245.59 to 443.14 kPa with the same net confining pressure, in other words an increase in matric suction leads to an increase of shear strength. However, based on this rate of increase it may be inferred that matric suction would have pronounced effect on silt soils than those of clays when comparing it with the findings of Getaneh (4) , Habtom (5) and Bantayehu (23),

The interpretation of this phenomenon, as affirmed also in the above theses, is that as air enters the pores, a contractile skin begins to form around the points of contacts between particles. The capillary action arising from suction at the contractile skin increases the normal forces at the inter-particle contacts. These additional normal forces may enhance the friction and the apparent cohesion at the inter-particle contacts. As a result, the unsaturated soil exhibits a higher strength than the saturated one.

The shear strength characteristics of an expansive silty clay from china were studied by Zhan and Ng with suction controlled direct shear test on both compacted and natural specimens. These studies have shown that the ϕ^b angle is generally equal to ϕ' at low matric suction and decreases to a lower value at high matric suction which shows there is a relationship between the ϕ^b angle and matric suction as presented in Fredland and Rahardjo [2].

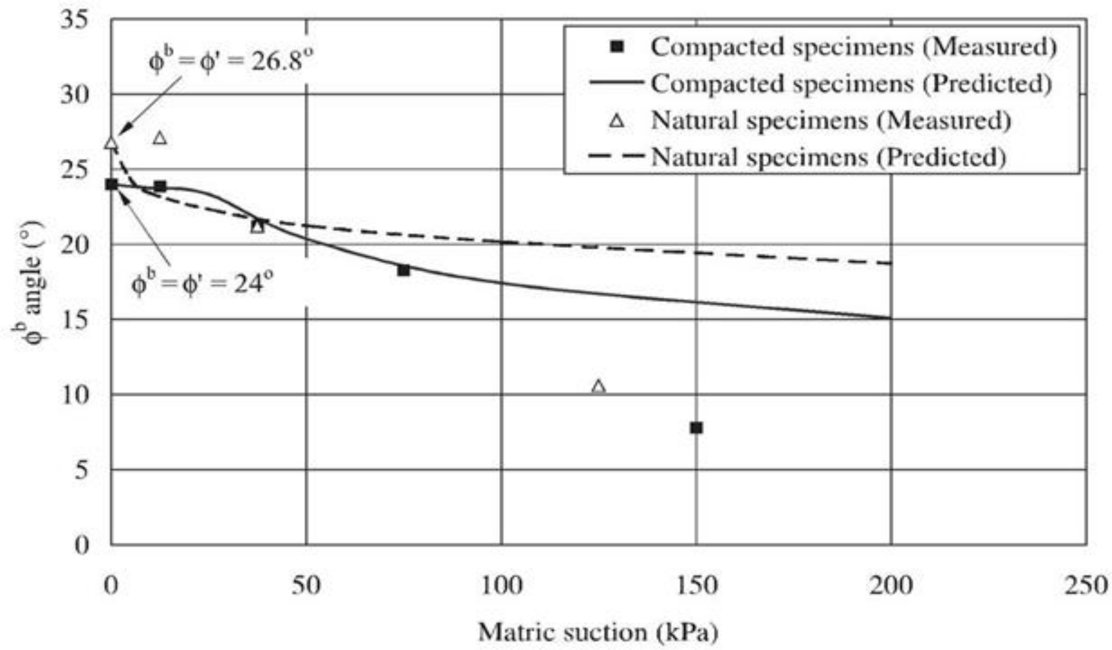


Figure 4.1 Variations of ϕ^b angles with matric suction for natural and compacted specimens

From the results of this study, it can be observed that, ϕ^b angle seems to be equal to ϕ' at low matric suctions and decreases to a lower value at high matric suction as shown in Figure 4.2.

Figure 4.2 shows the variations of ϕ^b angles with matric suction of Hawassa silt soil together with China ,Arbaminch,CMC, Bole and Kolfe areas soils.

Also, one can see from figure 3.8b, as the matric suction increases from 0 to 150 kPa , the contribution of suction to shear strength of the compacted silt sample increases from *C'* = 8.33 kPa to *C'* = 58.05 kPa, non linearly. .

The failure envelope for unsaturated soil is drawn on three dimensions as shown in Figure 4.3. This figure indicates the contribution of the matric suction on the shear strength of the soil when the value increases from 30kPa to 150 kPa.

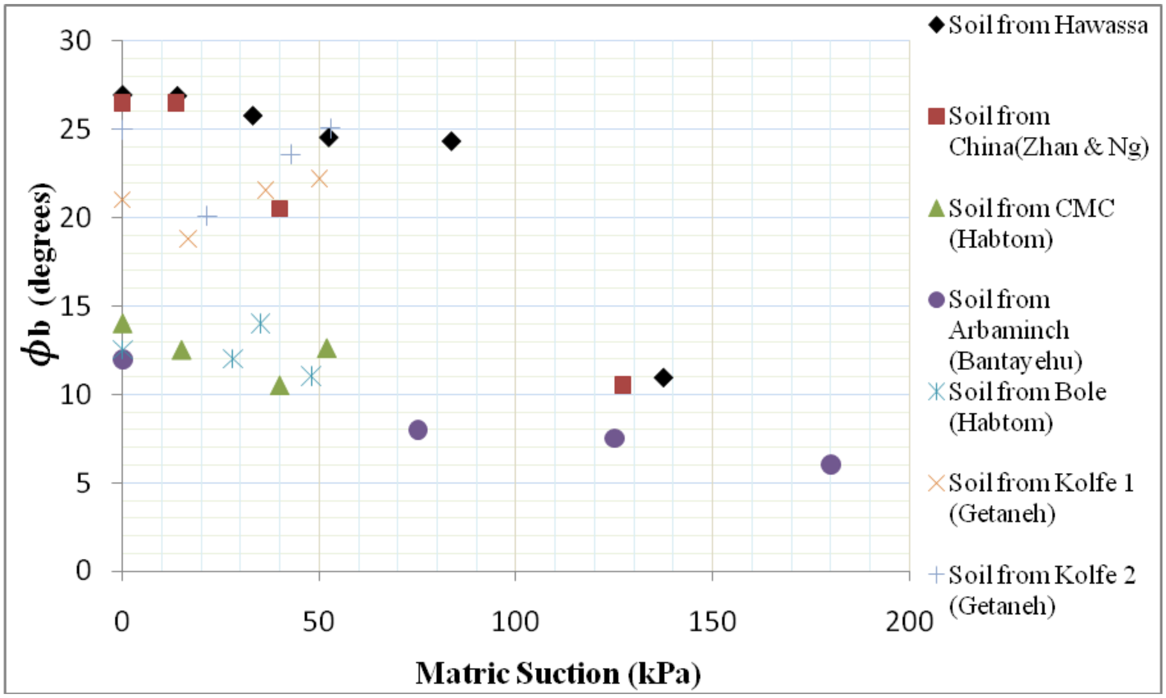


Figure 4.2 Variations of ϕ^b angles with matric suction of Hawassa silt soil together with China ,Arbaminch,CMC, Bole and Kolfe area soils.

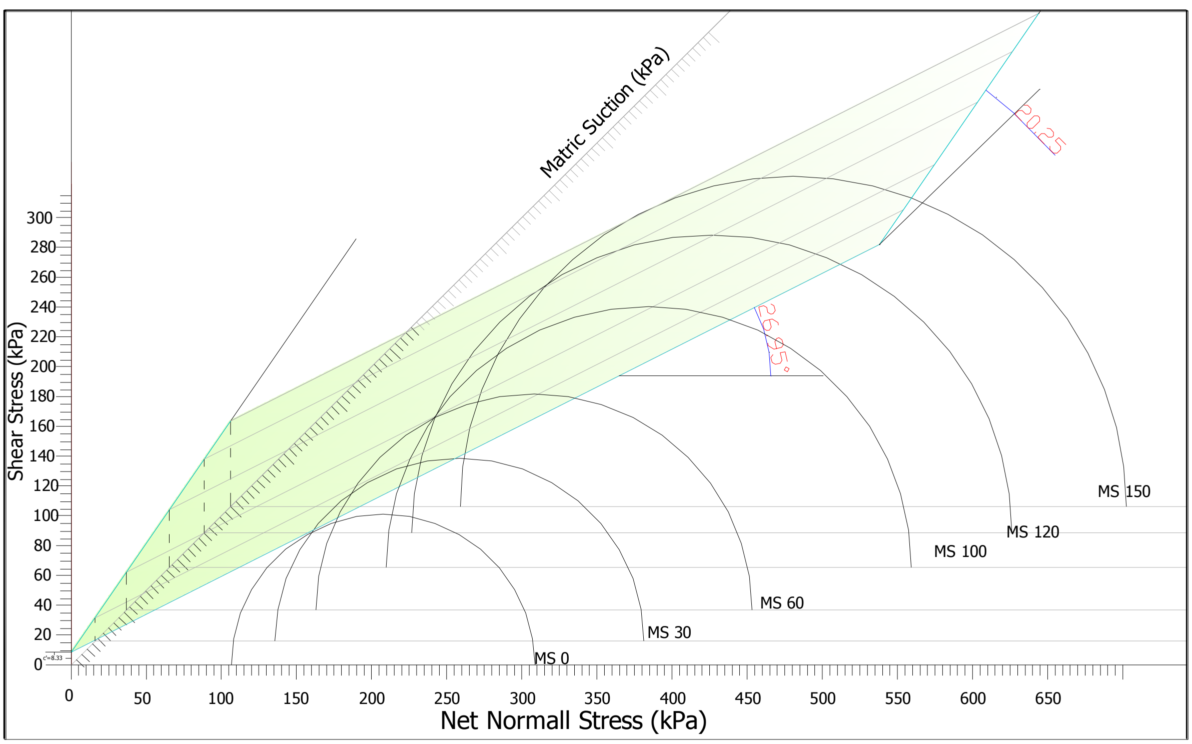


Figure 4.3 Failure envelope for unsaturated case

5. CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The results obtained from consolidated undrained triaxial tests performed on compacted silt soils of Hawassa in its unsaturated form indicated that as the matric suction increases from 0 to 150 kPa, the deviator stress increases from 202.48 kPa to 443.14 kPa under a net confining pressure of 200 kPa, which shows the shear strength of the silt samples significantly increase as a result of increasing matric suction. In comparison to previously reported results on unsaturated clays, the effect of matric suction on the deviatoric stress of silt soils is much more considerable as presented in 2.3.6.

The result also showed that, an increase in matric suction did not affect the general shape of the stress–strain relationship as shown in Figure 3.6. Thus, for the whole range of the applied matric suction, the shapes of the stress–strain curves for unsaturated samples resemble those for saturated samples.

It can also be concluded that, the contribution of suction to shear strength of the soil is a nonlinear function of matric suction under the applied matric suction. In other words, the contribution of suction to shear strength increases nonlinearly for the applied matric suction, however, the rate of increase with matric suction gradually decreases as shown by the experimental points in Figure 3.8b.

5.2 Recommendation

Further detail investigation in the shear strength of unsaturated soil can be done by taking more number of samples and conducting the tests over a wide range of matric suction to see effects like nonlinearity of shear strength versus matric sution.

Experimental techniques to determine the unsaturated soil properties are however costly and time consuming. Hence, with the already available data and other additional studies on unsaturated trixial tests on typically silt soils from different localities in the country, will enable other researcher interested in this area to set models that can be set up for the prediction of SWCC and unsaturated shear strength properties for the soil type.

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Appendices

Appendix-A

Laboratory test results

Table A-1 Results obtained from Gradation test

Project:	M.Sc. Thesis Work
Location:	Awassa Town
Test Pit No.:	1
Depth:	2.50 meter
Test Method:	ASTM D 421
Specific Gravity:	2.49
Test Temperature Degree:	22
Initial Dry Mass	1500g

Sieve Analysis

Sieve Size (mm)	Weight retained g	Percentage retained (%)	Percentage passing %
37.5	0	0	100
19	0	0.00	100.00
9.5	0	0.00	100.00
4.75	15	1.00	100.00
2.36	23	1.53	98.47
2.00	25	1.67	95.00
1.18	32	2.13	92.87
0.6	44	2.93	89.93
0.425	51	3.40	86.53
0.3	61	4.07	85.87
0.15	93	6.20	79.67
0.075	106	7.07	72.60

Hydrometer Analysis

Elapsed time (t) min	Actual Hydrometer reading	Temp	Effective depth (L) (cm)	Composite Hydrometer correction c	Corrected Hydromete reading (R)	Particle size D (mm)	Percent finer P	Percent finer with respect to the total mass P*(1500-106)/1500
0.5	1.019	22ºc	6.50	-0.003	1.0160	0.046	51.39	47.76
1	1.017	22ºc	8.06	-0.003	1.0141	0.040	45.29	42.09
2	1.015	22ºc	8.18	-0.003	1.0121	0.028	38.87	36.12
4	1.012	22ºc	10.19	-0.003	1.0085	0.022	27.30	25.37
8	1.009	22ºc	11.44	-0.003	1.0060	0.017	19.27	17.91
16	1.007	22ºc	13.28	-0.003	1.0040	0.013	12.85	11.94
30	1.006	22ºc	12.95	-0.003	1.0033	0.009	10.60	9.85
60	1.005	22ºc	13.70	-0.003	1.0019	0.007	6.10	5.67
120	1.004	22ºc	13.95	-0.003	1.0014	0.005	4.51	4.19
240	1.004	22ºc	14.20	-0.003	1.0009	0.003	2.89	2.69
480	1.004	22ºc	14.33	-0.003	1.0007	0.002	2.25	2.09
1440	1.004	22ºc	14.33	-0.003	1.0007	0.001	2.25	2.09

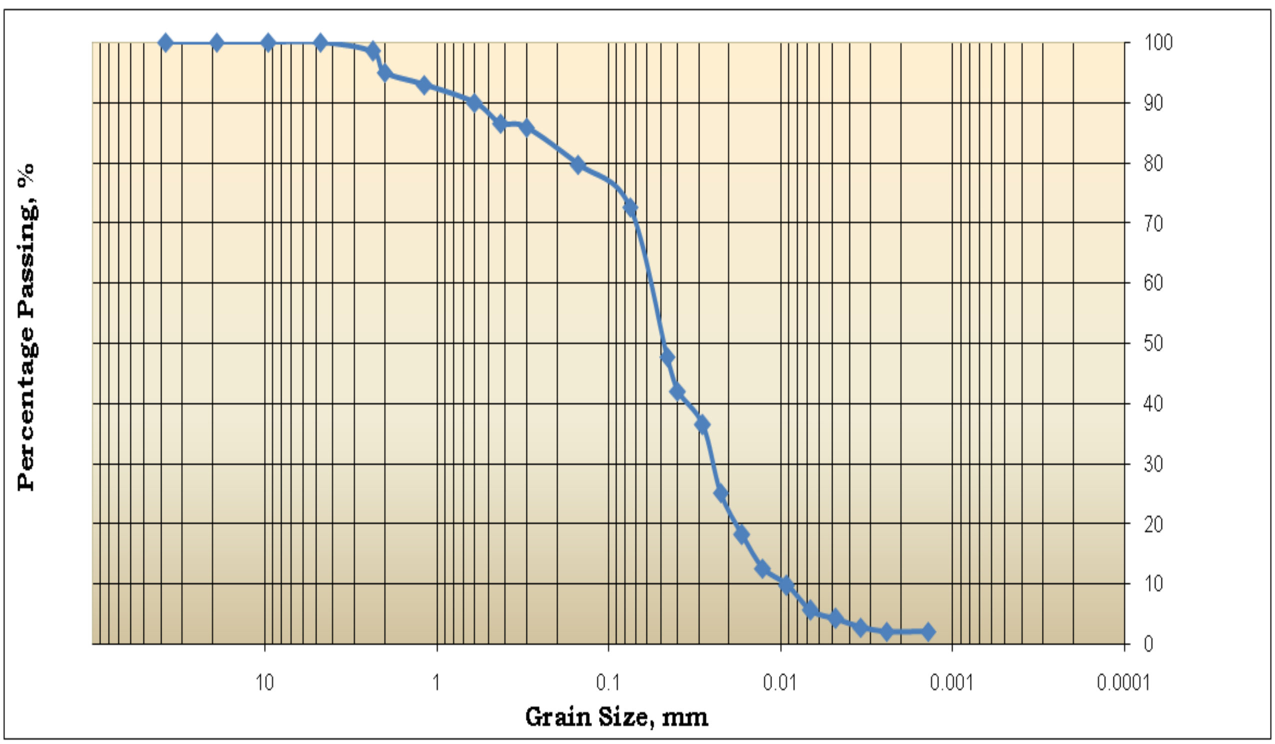

Figure A-1 Grain Size Distribution Curve

Table A-2 Results obtained from Gradation test

Project:	M.Sc. Thesis Work
Location:	Awassa Town
Test Pit No.:	2
Depth:	2.50 meter
Test Method:	ASTM D 421
Specific Gravity:	2.51
Test Temperature Degree:	22
Initial Dry Mass	1500g

Sieve analysis

Sieve Size (mm)	Weight retained g	Percentage retained (%)	Percentage passing %
37.5	0	0	100
19	0	0.00	100.00
9.5	0	0.00	100.00
4.75	0	0.00	100.00
2.36	20	1.33	98.67
2.00	22	1.47	97.20
1.18	29	1.93	96.73
0.6	39	2.60	94.13
0.425	46	3.07	91.07
0.3	56	3.73	90.40
0.15	98	6.53	83.87
0.075	103	6.87	77.00

Hydrometer analysis

Elapsed time (t) min	Actual Hydrometer reading	Temp	Effective depth (L) (cm)	Composite Hydrometer correction c	Corrected Hydromete reading (R)	Particle size D (mm)	Percent finer P	Percent finer with respect to the total mass P*(1500-103)/1500
0.5	1.0223	22ºc	10.50	-0.003	1.0193	0.058	61.99	57.73
1	1.0220	22ºc	10.50	-0.003	1.0190	0.045	60.90	56.72
2	1.0216	22ºc	10.70	-0.003	1.0186	0.032	59.74	55.64
4	1.0159	22ºc	12.10	-0.003	1.0129	0.024	41.37	38.53
8	1.0123	22ºc	13.10	-0.003	1.0093	0.018	29.87	27.82
16	1.0094	22ºc	13.90	-0.003	1.0064	0.013	20.69	19.26
30	1.0080	22ºc	14.20	-0.003	1.0050	0.010	16.09	14.99
60	1.0059	22ºc	14.70	-0.003	1.0029	0.007	9.22	8.59
120	1.0052	22ºc	15.00	-0.003	1.0022	0.005	6.91	6.43
240	1.0044	22ºc	15.10	-0.003	1.0014	0.004	4.60	4.28
480	1.0041	22ºc	15.20	-0.003	1.0011	0.002	3.45	3.21
1440	1.0041	22ºc	15.20	-0.003	1.0011	0.001	3.45	3.21

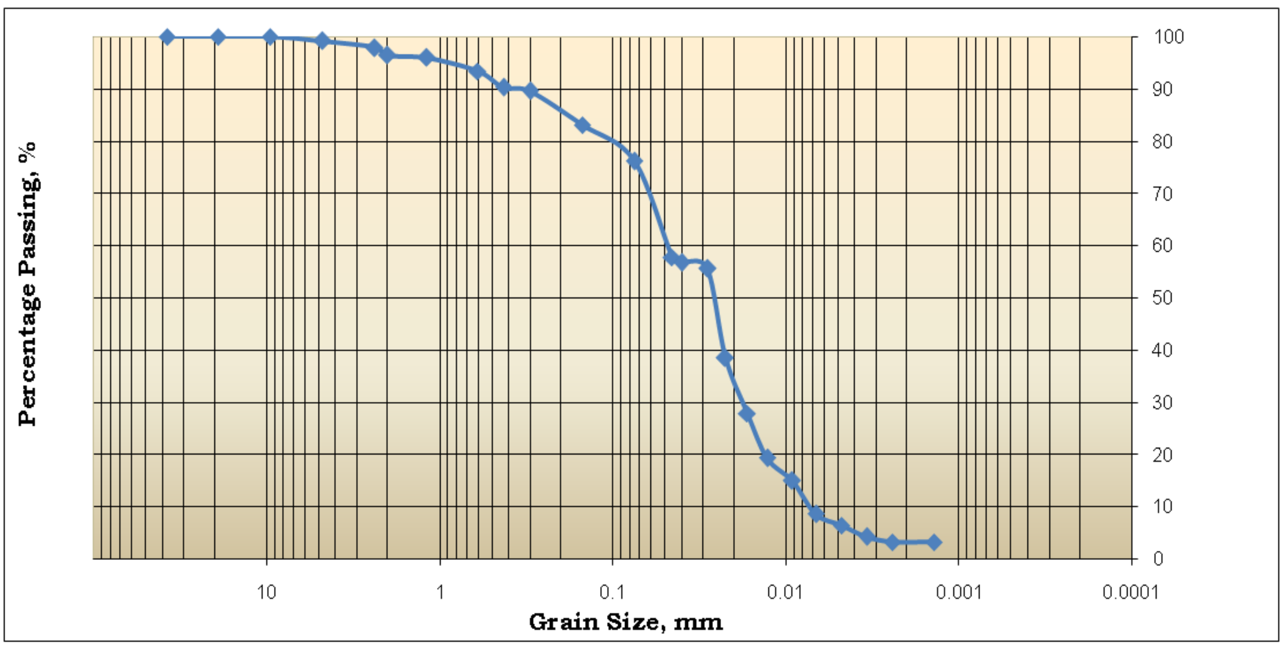

Figure A-2 Grain size curve for pit 2.

Table A-3 Results obtained from Gradation test

Project:	M.Sc. Thesis Work
Location:	Awassa Town
Test Pit No.:	3
Depth:	2.50 meter
Test Method:	ASTM D 421
Specific Gravity:	2.48
Test Temperature Degree:	22
Initial Dry Mass	1500g

Sieve analysis

Sieve Size (mm)	Weight retained g	Percentage retained (%)	Percentage passing %
37.5	0	0	100
19	0	0.00	100.00
9.5	0	0.00	100.00
4.75	0	0.00	100.00
2.36	16	1.07	98.93
2.00	18	1.20	97.73
1.18	25	1.67	96.07
0.6	35	2.33	93.73
0.425	42	2.80	90.93
0.3	52	3.47	87.47
0.15	94	6.27	81.20
0.075	99	6.60	74.60

Hydrometer analysis

Elapsed time (t) min	Actual Hydrometer reading	Temp	Effective depth (L) (cm)	Composite Hydrometer correction c	Corrected Hydrometer reading (R)	Particle size D (mm)	Percent finer P	Percent finer with respect to the total mass P*(1500-99)/1500
0.5	1.0230	22ºc	11.78	-0.003	1.0200	0.068	64.24	60.00
1	1.0210	22ºc	12.05	-0.003	1.0180	0.049	57.82	54.00
2	1.0152	22ºc	12.25	-0.003	1.0122	0.035	39.13	36.55
4	1.0114	22ºc	13.35	-0.003	1.0084	0.026	27.09	25.30
8	1.0091	22ºc	13.89	-0.003	1.0061	0.019	19.56	18.27
16	1.0072	22ºc	14.38	-0.003	1.0042	0.013	13.55	12.66
30	1.0063	22ºc	14.67	-0.003	1.0033	0.010	10.54	9.84
60	1.0049	22ºc	15.00	-0.003	1.0019	0.007	6.10	5.70
120	1.0044	22ºc	15.15	-0.003	1.0014	0.005	4.50	4.20
240	1.0039	22ºc	15.20	-0.003	1.0009	0.004	2.89	2.70
480	1.0037	22ºc	15.35	-0.003	1.0007	0.003	2.25	2.10
1440	1.0037	22ºc	15.35	-0.003	1.0007	0.001	2.25	2.10

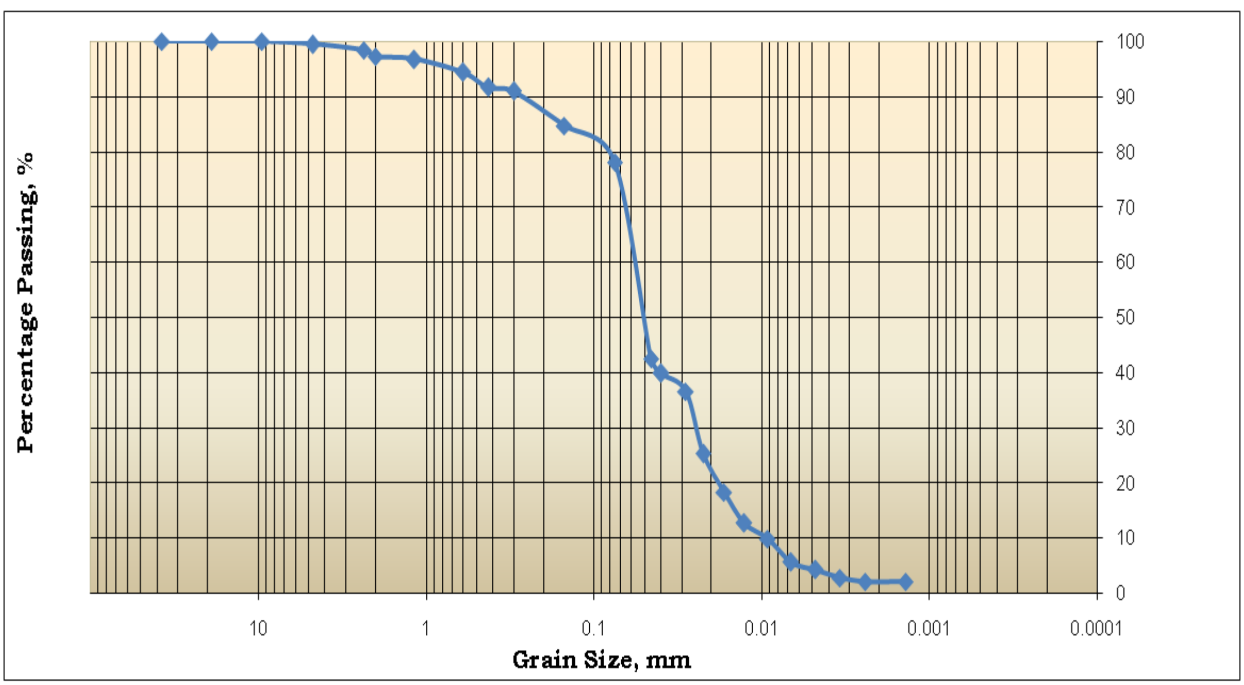

Figure A-3 Grain size curve for pit 3

Table A-4 Results obtained from Gradation test

Project: M.Sc. Thesis Work
Location: Awassa Town
Test Pit No.: 4
Depth: 2.50 meter
Test Method: ASTM D 421
Specific Gravity: 2.48
Test Temperature Degree: 22

Initial Dry Mass 1500gm

Sieve analysis

Sieve Size (mm)	Weight retained g	Percentage retained (%)	Percentage passing %
37.5	0	0	100
19	0	0.00	100.00
9.5	0	0.00	100.00
4.75	0	0.00	100.00
2.36	29	1.93	98.07
2.00	31	2.07	96.00
1.18	38	2.53	93.47
0.6	50	3.33	90.13
0.425	57	3.80	86.33
0.3	67	4.47	81.87
0.15	99	6.60	75.27
0.075	112	7.47	67.80

Hydrometer analysis

Elapsed time (t) min	Actual Hydrometer reading	Temp	Effective depth (L) (cm)	Composite Hydrometer correction c	Corrected Hydrometer reading (R)	Particle size D (mm)	Percent finer P	Percent finer with respect to the total mass P*(1500-112)/1500
0.5	1.0222	22ºc	10.45	-0.003	1.0192	0.064	61.67	57.07
1	1.0219	22ºc	10.50	-0.003	1.0189	0.046	60.71	56.17
2	1.0215	22ºc	10.65	-0.003	1.0185	0.032	59.39	54.96
4	1.0158	22ºc	12.14	-0.003	1.0128	0.025	41.08	38.01
8	1.0123	22ºc	12.95	-0.003	1.0093	0.018	29.71	27.49
16	1.0094	22ºc	13.88	-0.003	1.0064	0.013	20.55	19.02
30	1.0078	22ºc	14.23	-0.003	1.0048	0.010	15.42	14.27
60	1.0059	22ºc	14.70	-0.003	1.0029	0.007	9.15	8.47
120	1.0051	22ºc	15.00	-0.003	1.0021	0.005	6.84	6.33
240	1.0044	22ºc	15.10	-0.003	1.0014	0.004	4.56	4.22
480	1.0041	22ºc	15.20	-0.003	1.0011	0.003	3.44	3.18
1440	1.0041	22ºc	15.20	-0.003	1.0011	0.001	3.44	3.18

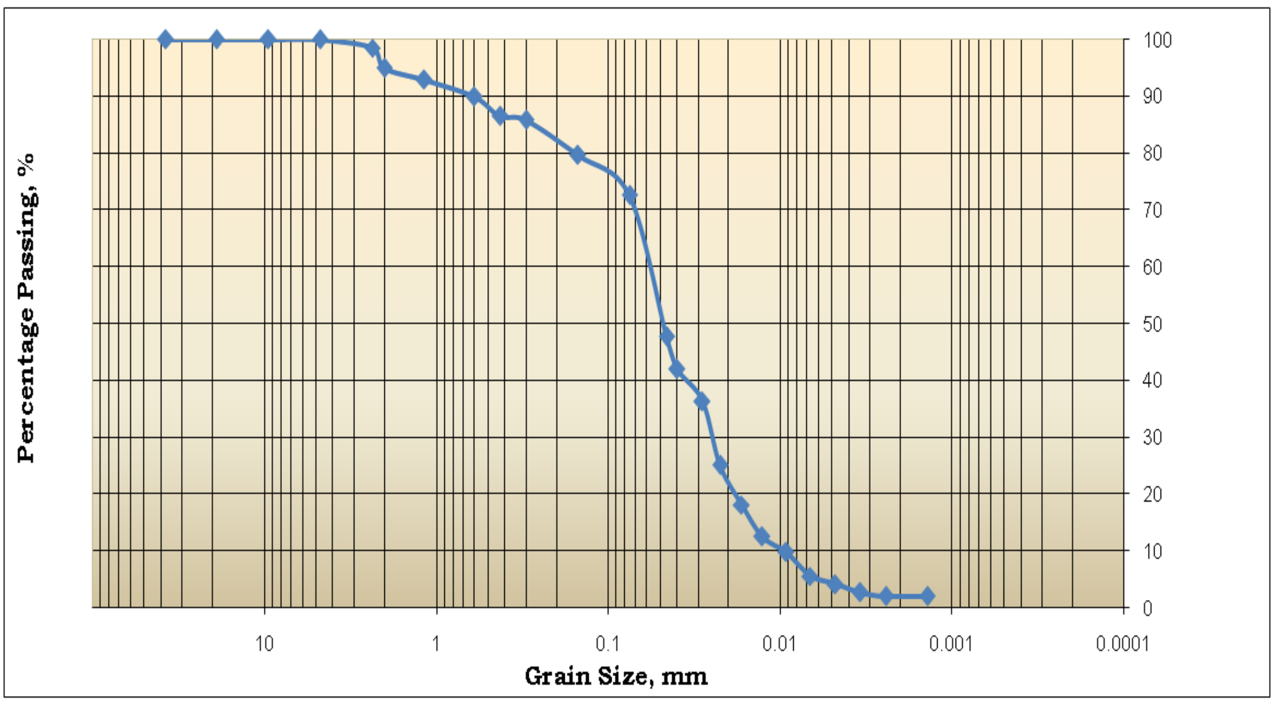

Figure A-4 Grain size curve for pit 4.

Table A-5 Results obtained from plastic and liquid limit test

Project : M.ScThesis Work
Test Pit No. 1
Depth : 2.50 meter
Test Method: BS - 1377 -2 : 1990

	Liquid Limit				**Plastic Limit**	
Trial No						
Number of blows						
Can Number						
Mass of can, g		None Plastic				
Mass of can + Wet soil, g						
Mass of container + Dry soil, g						
Mass of water, g						
Mass of dry soil, g						
Moisture content, %						

Table A-6 Results obtained from plastic and liquid limit test

Project : M.ScThesis Work
Test Pit No. 2
Depth : 2.50 meter
Test Method : BS - 1377 -2 : 1990

	Liquid Limit				**Plastic Limit**	
Trial No	1	2	3	4	1	2
Number of blows	21.00	26.00	29.00	31.00	----	----
Can Number	A108	A17	A2	16.00	33.00	D8
Mass of can, g	16.00	15.98	11.49	14.11	14.12	15.50
Mass of can + Wet soil, g	58.98	63.75	57.94	56.57	31.80	31.17

Mass of container + Dry soil, g	44.97	49.87	48.61	48.67	27.49	27.35
Mass of water, g	14.01	13.88	9.33	7.90	4.31	3.83
Mass of dry soil, g	28.97	33.89	37.12	34.56	13.37	11.85
Moisture content, %	48.36	40.95	25.13	22.86	32.27	32.29

Liquid Limit, %= 38-29 Plastic Limit, %= 32-28 PI, %= 6-02

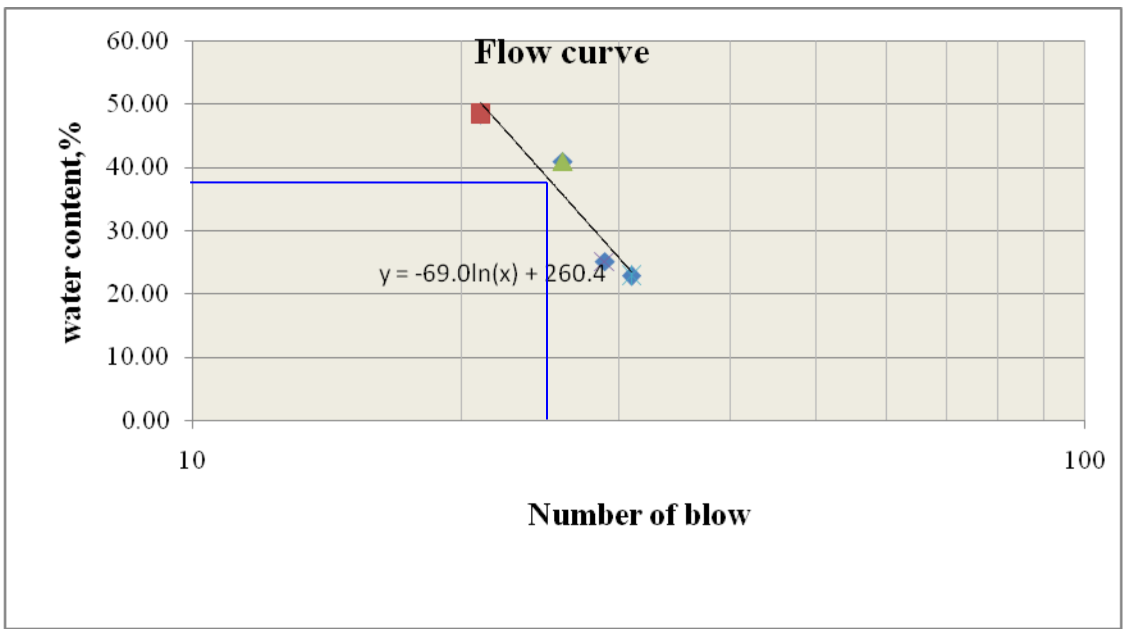

Figure A-6 Flow Curve for soil from pit 2.

Table A-7 Results obtained from plastic and liquid limit test

Project :	M.ScThesis Work
Test Pit No.	3
Depth :	2.50 meter
Test Method :	BS - 1377 -2 : 1990

	Liquid Limit				**Plastic Limit**	
Trial No						
Number of blows						
Can Number						
Mass of can, g		None Plastic				
Mass of can + Wet soil, g						
Mass of container + Dry soil, g						
Mass of water, g						
Mass of dry soil, g						
Moisture content, %						

Table A-8 Results obtained from plastic and liquid limit test

Project :	M.ScThesis Work
Test Pit No.	4
Depth :	2.50 meter
Test Method:	BS - 1377 -2: 1990

	Liquid Limit				**Plastic Limit**	
Trial No						
Number of blows						
Can Number						
Mass of can, g		None Plastic				
Mass of can + Wet soil, g						
Mass of container + Dry soil, g						
Mass of water, g						
Mass of dry soil, g						
Moisture content, %						

Table A-9 Parameters used to draw the Mohr circles for saturated soils (results obtained from the consolidated undrained test)

Sample	σ_3	Excess pore water Δu_w(at failure)	$(\sigma_1 - \sigma_3)$	σ_1	$\sigma'_1 = \sigma_1 - \Delta u_w$	$\sigma'_3 = \sigma_3 - \Delta u_w$
S-1	100	49.47	111.60	211.60	162.13	50.53
S-2	200	91.24	202.48	402.48	311.24	108.76
S-3	300	138.94	294.15	594.15	455.21	161.06

Table A-10 Parameters used to draw the Mohr circles for unsaturated soils (results obtained from the consolidated undrained test)

Sample	σ_3	*MS*	Excess pore air Δu_a	Excess pore water Δu_w	*MS (at failure)*	$(\sigma_1 - \sigma_3)$	σ_1	$\sigma'_1 = \sigma_1 - \Delta u_a$	$\sigma'_3 = \sigma_3 - \Delta u_a$
S-4	200	30	81.95	95.85	13.90	245.59	445.59	363.64	118.05
S-5	200	60	72.78	74.88	33.09	290.00	490.00	417.22	127.22
S-6	200	100	54.63	103.56	52.40	349.60	549.60	494.97	145.37
S-7	200	120	47.65	62.14	83.66	399.36	599.36	551.71	152.35
S-8	200	150	46.98	42.51	137.45	443.14	643.14	596.16	153.02

Appendix-B

Secondary data

Table-B Summary of laboratory test results report obtained from Construction Design Share Company.

Location	Project Name	Depth of sampling (m)	Specific Gravity	Atterberg limits			Description (Soil type)
				LL	PL	PI	
Hawassa Town	B + G + 7 Building (W/ro Tigest Kassa)	5.0	2.45	39	33	6	Grey sandy silt with little clay
"'	"'	13.3	2.45	0	0	NP	Grey sandy silt with little clay
"'	"'	3.0	2.45	42	30	12	Light grey sandy silt with some clay
"'	"'	8.4	2.45	40	35	5	Dark grey silty sand with little clay
"'	"'	12.2	2.45	34	30	4	Grey sandy silt with little clay
"'	Haile and Alem International Plc	4.5	2.46	0	0	NP	Greyish sandy silt with little gravel
"'	"'	3.0	2.45	0	0	NP	Greyish sandy silt
"'	"'	4.5	2.45	0	0	NP	Greyish sandy silt with little gravel
"'	"'	3.5	2.47	0	0	NP	Greyish sandy silt with some gravel
"'	B + G + 6 Building Hawassa Hotel and Resort(Progress International Hotel Plc)	4.5	-	-	-	-	-
"'	"'	5.5	-	0	0	NP	-
"'	"'	2.5	2.45	-	-	-	Grey sandy silt with little clay
"'	"'	8.5	2.46	-	-	-	Grey sandy silt
"'	"'	8.5	2.45	-	-	-	Red to brown sandy silt
"'	"'	2.5	-	0	0	NP	
"'	"'	8.0	2.45	-	-	-	Light grey silty sand
"'	"'	12.5	2.45	-	-	-	Red to brown sandy silt
"'	"'	2.5	2.43	38	30	7	Light grey sandy silt
"'	"'	11.1	-	0	0	NP	-
"'	"'	2.5	-	0	0	NP	-
"'	"'	10.5	-	0	0	NP	-
"'	"'	14.3	-	0	0	NP	-
"'	"'	2.5	2.44	0	0	NP	Red to brown sandy silt
"'	"'	14.0	2.46	-	-	-	Red to brown silty sand

Appendix-C

Pictures

Figure C-1 Compacting the soil in a mold

Figure C-2 Inserting the sampler in to the mold

Figure C-3 Extruding the soil with the sampler from the mold.

Figure C-4 Extruding the Sample from the sampler

Figure C -5 Sample after extracted from the sampler

Figure C - 6 Sample being mounted on the triaxial cell

Figure C -7 Sample mounted on the trixial cell.